STRUCTURES MANUAL

2008

NEVADA DEPARTMENT OF TRANSPORTATION
STRUCTURES DIVISION

NDOT
STRUCTURES MANUAL

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Structures Division

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FOREWORD

The *NDOT Structures Manual* has been developed to provide bridge designers with NDOT's standard structural design policies and practices. Designers should attempt to meet all of the criteria presented in the *Manual*, while fulfilling NDOT's mission of providing a safe and efficient transportation system for the State. Designers must consider economic impacts, aesthetics, and the social and cultural resources of the project area and request exceptions to the *Manual* criteria when conditions warrant. Because it is impossible to address every issue that bridge designers will encounter, sound engineering judgment must be exercised when conditions arise that are not specifically covered in the *Manual*.

The *NDOT Structures Manual* has been prepared based on the 4th Edition of the AASHTO *LRFD Bridge Design Specifications*.

ACKNOWLEDGEMENTS

The *NDOT Structures Manual* was developed by the NDOT Structures Division with assistance from the consulting firm of Roy Jorgensen Associates, Inc., Professor Dennis Mertz of the University of Delaware, and the consulting firm of CH2M Hill, Inc.

REVISION PROCESS

The *NDOT Structures Manual* is intended to provide current structural design policies and procedures for use in developing NDOT projects. To ensure that the *Manual* remains up-to-date and appropriately reflects changes in NDOT's needs and requirements, its contents will be updated on an ongoing basis. It is the responsibility of the *Manual* holder to keep the *Manual* updated.

The NDOT Structures Division will be responsible for evaluating changes in the structural design literature (e.g., updates to the *LRFD Specifications*, the issuance of new research publications, revisions to Federal regulations) and will ensure that those changes are appropriately addressed through the issuance of revisions to the *Manual*. It is important that users of the *Manual* inform NDOT of any inconsistencies, errors, need for clarification, or new ideas to support the goal of providing the best and most up-to-date information practical. Comments and proposed revisions may be forwarded to the Chief Structures Engineer using the Revision Proposal Form.

NDOT Structures Manual

Revision Proposal Form

To propose a revision to the *NDOT Structures Manual*, complete and return this Revision Proposal Form to:

Chief Structures Engineer
Nevada Department of Transportation
1263 S. Stewart Street
Carson City, Nevada 89712
Fax: 775.888.7405
E-mail: info@dot.state.nv.us (include "Structures Manual" in subject line)

Identification

Date Submitted: ______________________________

Submitted By (name, agency/firm): ______________________________

Contact Information (phone #, e-mail): ______________________________

Description of Proposed Revision (attach additional sheets as necessary)

Applicable *Manual* Section Number(s): ______________________________

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Justification for Revision: ______________________________

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COMPUTER SOFTWARE

8.1 OVERVIEW

8.1.1 Introduction

The Structures Division uses a variety of computer programs for structural analysis and design, which can provide significant benefits. These include the capability of quickly analyzing several alternative designs (i.e., simulation capabilities), of reducing the probability of mathematical errors, and for saving time by avoiding laborious hand calculations. However, the user of any computer program must consider the following:

1. Judgment and experience are critical to the proper interpretation of the computer outputs.

2. The user should, after the computer run, recheck inputs for accuracy.

3. The user should carefully check all outputs to ensure that answers are reasonable and logical and that there are no obvious errors. The check should include an equilibrium check in structural applications, for example, verifying that the sum of the applied loads equals the sum of the reactions. In addition, simple free-body diagrams can also be checked by cutting the structure at a section where a free body can easily be taken.

4. Hand calculations based on simplifying assumptions may also be used to verify computer results.

5. The user should be familiar with the applications and limitations of each program.

8.1.2 Chapter Content

Chapter 8 presents the majority of the commercially available computer programs used by the Structures Division. The Chapter provides a brief description of the program's application and function.

8.1.3 Coordination with *Structures Manual*

Throughout Part II of the *NDOT Structures Manual*, the text identifies, where applicable, any computer software that may be used for a specific structural element. For example, Section 17.1.4.3 discusses the use of StrainWedge, LPile Plus and COM624P to analyze the lateral soil-structure interaction in the design of foundations.

8.2 COMMERCIALLY AVAILABLE PROGRAMS

8.2.1 SAP2000

8.2.1.1 Description

SAP2000 is an integrated, general purpose structural analysis program with bridge-specific templates. The program and its element library are capable of simple line-girder, two-dimensional grid or three-dimensional finite-element type analyses. Bridge designers can use SAP2000 bridge templates for generating bridge models, automated bridge live load analysis and design, bridge base isolation, bridge construction sequence analysis, large deformation cable-supported bridge analysis and pushover analysis.

SAP2000 is available from Computers & Structures, Inc.

8.2.1.2 Inputs

The bridge geometry is defined through:

- quick definition of a variety of bridge cross sections including concrete box girder sections, concrete tee-beam sections and steel composite bridge sections;

- variable section parameters along the length of the bridge;

- automatic model updates with changes in parameters; and

- auto application of loads to the bridge structure.

8.2.1.3 Outputs

SAP2000's bridge-specific output includes:

- AASHTO LRFD design checks,
- influence lines and surfaces,
- forces and stresses along and across the bridge,
- displacement plots, and
- animated stress plots.

In general, graphical and tabulated outputs are available.

8.2.2 GT Strudl

GT STRUDL performs general purpose structural analysis and design, plus structural data base processing, for a broad range of structural problems. GT STRUDL integrates graphical modeling, automatic mesh and data generation, finite-element analysis, structural frame design, interactive graphics and structural data base management into a menu-driven information processing system.

GT STRUDL contains a library of member and finite-element types consisting of seven member types (constant or variable cross-section), over 100 conventional, isoperimetric and hybrid formulation finite-element types, and many special transition elements.

8.2.3 WinSEISAB

8.2.3.1 Description

WinSEISAB was specifically developed to perform the seismic analysis of bridges. The overall objective is to provide the practicing bridge engineer with a usable design tool and mechanism for implementing the latest seismic design methodologies into the bridge engineering profession.

WinSEISAB is available from Imbsen Software Systems.

8.2.3.2 Input

Horizontal alignments composed of a combination of tangent and curved segments are described using alignment data taken directly from roadway plans. WinSEISAB has generating capabilities that will, with a minimum amount of input data, automatically provide a model consistent with the model currently being used to conduct dynamic analyses. The central theme underlying the development of WinSEISAB is to provide the bridge designer with an effective means of user-program communication using a problem-oriented language developed specifically for the bridge engineer. User input data is thoroughly checked for syntax and consistency prior to conducting an analysis, and numerous default values are assumed for the data not entered by the user.

8.2.3.3 Output

WinSEISAB can be used to analyze simply supported or continuous deck, girder-type bridges with no practical limitation on the number of spans or the number of columns at a pier. In addition, seismic restrainer units may be placed between adjacent structural segments.

8.2.4 RESPONSE 2000

Response 2000 is a sectional analysis program that calculates the resistance and ductility of a reinforced-concrete cross section subjected to shear, moment and axial load. All three loads are considered simultaneously to determine the full load-deformation response using the modified compression field theory (MCFT).

Response 2000 can predict the shear resistance for sections that cannot easily be modeled, such as circular columns and tapered web beams.

8.2.5 WinBDS

8.2.5.1 Description

WinBDS analyzes or designs orthogonal bridge frames and is applicable to the majority of concrete bridges being designed today. However, the design methodology employed is the load factor design (LFD) methodology of the AASHTO *Standard Specifications for Highway Bridges*. Prestressed concrete analysis and design capabilities include:

- continuous prestress,
- simple span prestress,

- segmental prestress,
- 100 prestress cable paths per frame,
- 3D cable friction losses,
- primary and secondary effects,
- cross-section analysis,
- ultimate capacity check,
- AASHTO *Standard Specifications'* shear requirements,
- tendon elongations, and
- calculation of AASHTO *Standard Specifications'* long-term losses.

8.2.5.2 Input

Cross sections may by specified by using a predefined, standard box girder superstructure shape or by adding or subtracting geometric parts using predefined part codes. Live loads can be automatically generated for the AASHTO HS20-44 live-load model and the P13 permit truck. By providing the number of live loads, lanes may be applied to the structure by describing the axle loads, distance between axles and number of live load lanes. Uniform lane loads can be defined with moment and shear riders. Other automatically generated live loads include the Cooper Loading for railroad bridges and a user-defined vehicle of up to 97 axles.

8.2.5.3 Output

Output results are organized in the same order as the input data. All input data is echoed in the output reports. Output reports include section properties, dead load and additional fixed load results, live load force envelopes, prestress results including required P-Jack, mild steel requirements and shear design stirrup requirements. Additional reports are generated for the Rating Subsystem and Prestressed, Precast Girder Subsystem.

A graphical display of moment and shear envelopes is available after an analysis has been performed. Additional plots are available for deflections, top and bottom fiber stresses, prestress force, prestress cable path and influence lines. These plots can be displayed for all spans in the bridge or for an individual span or for any sequential list of spans. Where applicable, the plots include the different trials that are available, such as dead load, added dead load, live load (both negative and positive envelopes), prestress, etc. These loads can be plotted individually or combined into one load case.

8.2.6 BRASS-GIRDER(LRFD)™ and BRASS-GIRDER(LFD)™

8.2.6.1 Description

BRASS-GIRDER(LRFD)™ and BRASS-GIRDER(LFD)™ are comprehensive programs for the analysis of highway bridge girders. BRASS-GIRDER(LRFD)™ uses the finite element theory of analysis and the current AASHTO *LRFD Specifications*. The BRASS-GIRDER(LFD)™ program is primarily used by the Division for structure load rating.

The BRASS family of programs is available from the Wyoming Department of Transportation.

8.2.6.2 Inputs

BRASS-GIRDER(LRFD)™ uses Windows-based Graphical User Interfaces (GUI) for data input. System input is free format consisting of commands grouped logically to define the bridge structure, loads to be applied and the output desired.

Girder types may be simple span, continuous, hinged or cantilevered. Girders may be constructed of steel, reinforced concrete, or prestressed concrete (pre- and post-tensioned). Composite steel and composite prestressed concrete girders may be included. BRASS can analyze variable-depth girders, such as tapered or parabolic. The user may specify (by name) predefined cross sections that are stored in the cross section library. The library contains nearly all AISC rolled wide flange shapes and most AASHTO standard shapes for prestressed concrete I-beams. Using a library utility program, the user may modify the geometry of the existing sections, add new sections or delete existing sections.

Stage construction may be modeled by respective cycles of the system for girder configuration and load application. Cycles are automatic if desired. The dead load of structure members is automatically calculated if desired. Additional distributed loads and point loads may be applied in groups, and each group assigned to a specific construction stage. Distributed loads may be uniform or tapered and divided into sections to model sequential slab pours. Loads due to prestressing are calculated and applied internally. Live loads may be moving trucks or uniform lane loads, which include the HL-93 vehicle described in the AASHTO *LRFD Specifications*. Impact may be user defined, as specified by AASHTO, or the user may reduce impact to model reduced speed limits.

8.2.6.3 Outputs

The program computes moments, shears, axial forces, deflections and rotations caused by dead loads, live loads, settlements and temperature changes. These actions are used by various subroutines to analyze user-specified sections of the girder.

8.2.7 CONSPAN™

8.2.7.1 Description

CONSPAN™ is a comprehensive program for the design and analysis of prestressed, precast concrete beams using either the AASHTO LFD or LRFD design methodologies. The program accommodates simple spans and multiple simple spans made continuous for live loads. CONSPAN™ incorporates both LFD and LRFD AASHTO Specifications into one interface.

Simple-span static analysis is performed for dead loads resisted by the precast sections. Continuous static analysis is performed for dead loads acting upon the composite structure. A continuous moving load analysis is performed for the live load.

CONSPAN™ is available from LEAP Software, Inc.

8.2.7.2 Input

CONSPAN™ simplifies the entry of project data with a system of tab screens, dialog boxes, graphical button, menus and wizards. Designs are completed with either the CONSPAN™ automated features or the user-specified input.

8.2.7.3 Output

CONSPAN™ presents analysis results in a variety of easy-to-view formats, from a one-page design summary to comprehensive project reports. Analysis results and graphical sketches can be exported to spreadsheets and .dxf formats.

For individual beam designs, various design parameters such as distribution factors, impact/dynamic allowance factors, and allowable stresses are established. The strand and debonding/shielding patterns can be automatically generated by CONSPAN™ or specified by the user. Debonding constraints limiting the number of debonded strands can also be user-specified. Service load stress envelopes, generated by combining the results of the analysis, are checked against allowable limits. Factored positive moments and shears are checked against the ultimate strength capacity of the effective section. Mild reinforcement in the deck, at the piers, is computed for factored negative and positive moments. Many other code criteria (e.g., cracking moments, horizontal shear, stresses at limit states) are also automatically checked.

8.2.8 CONBOX™

8.2.8.1 Description

CONBOX™ is specifically developed for the analysis and design of post-tensioned and cast-in-place reinforced concrete box girder and slab bridges constructed on falsework. By incorporating both AASHTO Standard and LRFD Specifications in one interface, CONBOX™ makes the transition to LRFD simple and efficient. The program accommodates span hinges and a variety of column shapes and fixity conditions.

CONBOX™ is available from LEAP Software, Inc.

8.2.8.2 Input

CONBOX™ requires that users specify bridge layout information such as alignment, span lengths and cross-sections; pier, hinge and abutment locations; and superstructure-to-substructure connectivity. CONBOX™ can import BDS input files to simplify workflow.

8.2.8.3 Output

CONBOX™ computes ultimate moment calculations based on AASHTO equations or strain compatibility. The capacity/demand ratio and factor of safety are reported at each checkpoint for stresses, ultimate moment and shear length. Top and bottom flange stresses are checked, and the required shear and moment reinforcement capacity is calculated. CONBOX™ also calculates the approximate level of post-tensioning force required to satisfy stress and the minimum compressive strength necessary to satisfy AASHTO compressive stress checks. Detailed shear and moment calculations are also automatically computed.

8.2.9 MDX™

8.2.9.1 Description

MDX™ (Curved & Straight Steel Bridge Design & Rating) designs and rates steel I-shaped or box girder bridges for compliance with AASHTO Specifications — the 17th Edition of the AASHTO *Standard Specifications* for Allowable Stress Design (ASD) and Load Factor Design (LFD), and the 4th Edition of the *LRFD Bridge Design Specifications*. Analysis methods include line-girder and system analysis using grid or plate-and-eccentric-beam, finite-element models for up to 20 spans and 60 girders.

MDX™ is available from MDX Software, Inc.

8.2.9.2 Input

Input features include:

- geometry generation feature for laying out parallel/concentric girder systems, including those with variable horizontal curvature and skewed supports;
- nodal coordinate input feature for accommodating complex girder system framing plans and roadway layouts;
- standard trucks and user-defined trucks (including rail loading); and
- lane loading on influence surfaces or wheel load distribution on influence lines.

8.2.9.3 Output

Output features include:

- camber data and live load deflections,
- incremental stress and deflection tables from slab pour sequence analysis,
- performance ratio output, and
- girder rating according to ASD, LFD or LRFD.

8.2.10 BRIDGE DESIGNER II

BRIDGE DESIGNER II (BD2) analyzes and designs concrete segmental bridges. BD2 uses basic matrix structural analysis formulation, combined with time-dependent material properties, to provide a time simulation of a structure under construction. During this process, all stress conditions are checked at every construction step and in service. The composite section capability is directly applicable to precast/prestressed concrete bridges using continuous spliced girders or bulb-tees. Because of the age difference in slabs and girders due to the chosen construction sequence, these structures often exhibit specific behavior due to stress redistributions in composite state (i.e., differential creep and shrinkage).

BD2 is an appropriate tool to evaluate the time-dependent behavior of concrete structures because it builds upon a time-dependent stiffness solution and provides a flexible environment for construction simulation. In addition, it provides a solution for creep and shrinkage in composite structures built in stages, incorporating the concept of self-equilibrating stresses while assuming that the composite sections remain in a plane due to full shear transfer at the connection girder-slab. BD2 also provides accurate stress combinations in the composite sections under live loads (HL-93) and non-linear thermal gradients applied to the composite sections.

8.2.11 WinRECOL

8.2.11.1 Description

WinRECOL analyzes, designs or checks a reinforced concrete column. The program has the option to choose one of the following design specifications:

- CLFD = Caltrans Bridge Design Specification LFD, September 2004, with Revisions
- ALFD = AASHTO Bridge Design Specification LFD, Sixteenth Edition, 1996, 1998 Interims and Division I-A Seismic Design Article
- CLRFD = AASHTO LRFD Bridge Design Specification, Third Edition, 2004, 2006 Interims with Caltrans Amendments, (Blue Sheets v0.06)
- ALRFD = AASHTO LRFD Bridge Design Specification, Third Edition, 2004, 2006 Interims

WinRECOL is based upon the theory of ultimate strength and uses standard or arbitrarily described cross sections with predefined or arbitrary rebar patterns to perform three different solution types.

8.2.11.2 Input

Column shapes may be specified by using predefined, standard geometric shapes commonly used for bridges or by specifying coordinates for an arbitrarily shaped symmetrical cross section. Voids may also be included when coordinates are used to describe the cross section.

Rebar patterns are specified by using predefined rebar configurations commonly used for reinforced concrete columns or by specifying the coordinates for each bar. Coordinates for the rebar pattern are generated automatically from a minimum amount of input data.

8.2.11.3 Output

Solution types in the program include: 1) “Analyze,” 2) “Design,” and 3) “Check.” “Analyze” is used to compute the coordinates of an interaction surface in the first quadrant for planes in 15° increments. “Design” is used to determine the amount of reinforcing steel area required for the selected Design Specification (AASHTO LFD/LRFD or Caltrans LFD/LRFD). “Check” is used to check the adequacy of the designed column for the Design Specification selected and to perform a plastic hinging analysis.

Output in the check mode includes column spiral reinforcement required for column confinement, minimum reinforcement and shear due to plastic hinging.

A graphical display of the column cross section, plus moment interaction diagrams at 15° intervals, can be displayed after an analysis or check has been performed.

8.2.12 StrainWedge

StrainWedge characterizes the behavior of single piles, pile groups and pile caps embedded in uniform or layered soils when subjected to lateral loads. The program is based upon the concepts of the strain wedge model. The characterization of lateral load behavior associated with single piles and pile groups embedded in uniform or layered soils can be accomplished using the strain wedge model. The strain wedge model for laterally loaded pile behavior is a predictive method that relates the stress-strain behavior of soil in the developing three-dimensional passive wedge in front of the pile (denoted as the strain wedge) under lateral load to the one-dimensional, beam-on-elastic foundation parameters commonly employed in “p-y” curve analyses. The strain wedge model has been developed to analyze individual piles, or a square or rectangular pile group with its associated pile cap, under lateral loading. The strain wedge model can account for both free- and fixed-head conditions and uniform or layered soil conditions. The strain wedge model also has the ability to predict the working pile head load of a single pile and/or that of each pile in a group, the working load associated with a pile cap, the pile head deflection, the maximum bending moment for a single pile or each pile in a group, and p-y curves for a single pile.

Based on the strain wedge model, the computer program DFSAP provides a direct assessment of the three-dimensional/rotational spring stiffnesses of an isolated short, intermediate or long pile/shaft or similar stiffness of a pile/shaft group with our without a cap. Accordingly, the bridge engineer will be able to assess the various springs of the foundation stiffness matrix with disregard for the complexity of the soil profile, the type and arrangement of piles/shafts and nonlinear material behavior. Soil liquefaction and the associated induced pore water pressures are considered in the assessed foundation stiffness. Lateral spreading of the soil is an important phenomenon and is also handled by the DFSAP program.

8.2.13 LPILE Plus

8.2.13.1 Description

LPILE Plus is a special purpose program based on rational procedures for analyzing a pile under lateral loading. Soil behavior is modeled with p-y curves internally generated by the computer program following published recommendations for various types of soils; alternatively, the user can manually introduce other p-y curves. Special procedures are programmed for developing p-y curves for layered soils and for rocks.

A single, user-friendly interface written for the Microsoft Windows© environment is provided for the preparation of input and analytical run and for the graphical observation of data contained in the output file. The program has been written in 32-bit programming codes for compatibility with the latest versions of the Microsoft Windows operating system. The program produces plain-text input and output files that may be observed and/or edited for their inclusion in project reports.

8.2.13.2 Inputs

Several types of pile-head boundary conditions may be selected, and the properties of the pile can also vary as a function of depth.

8.2.13.3 Outputs

The program computes deflection, shear, bending moment and soil response with respect to depth in nonlinear soils. Components of the stiffness matrix at the pile head may be computed internally by the program to assist users in superstructure analysis. Several pile lengths may be automatically checked by the program to help the user produce a design with an optimum pile penetration. LPILE Plus has capabilities to compute the ultimate-moment capacity of a pile section and can provide design information for rebar arrangement. The user may optionally ask the program to generate and consider nonlinear values of flexural stiffness (EI) that are generated internally based on specified pile dimensions, material properties and cracked/uncracked concrete behavior.

8.2.14 COM624P

8.2.14.1 Description

Laterally Loaded Pile Analysis, Version 2.0. COM624P uses the p-y curve method to analyze pile foundations subjected to lateral loads. The program determines the pile deflection, rotation, bending moment and shearing forces using an iterative process that considers the nonlinear response of the foundation soils. This program uses the MS DOS operating system.

8.2.14.2 Input

The program's menu-driven input includes the characterization of the piles and soil, and the applied loads.

8.2.14.3 Output

COM624P determines stresses and distortions of piles or drilled shafts under lateral loads and reports them in graphic output.

8.2.15 BRASS-CULVERT

BRASS-CULVERT analyzes and designs reinforced concrete box culverts. The program has an easy-to-use windows GUI input and produces GUI plot results and a text output file. The program is part of the BRASS family of programs available from the Wyoming Department of Transportation.

8.2.16 Retain Pro

8.2.16.1 Description

Retain Pro designs and checks cantilevered or restrained retaining walls in accordance with AASHTO, ACI, IBC and other standards.

8.2.16.2 Input

Retain Pro takes the designer through a series of design tabs where design criteria, loads, and wall and footing dimensions are entered. There is a visual input screen where previous inputs can be seen or edited.

8.2.16.3 Output

Retain Pro calculates the wall stability ratios, soil pressures, wall and footing dimensions, and required reinforcing steel. The program also features graphics showing the final wall configuration, wall loading, and shear and moment diagrams.

Chapter 9

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Chapter 10

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GENERAL

10.1 BASIC APPROACH

The basic approach for Part II of the *NDOT Structures Manual* (the *Manual*) is as follows:

1. Application. The *Manual* is an application-oriented document.

2. Theory. The *Manual* is not a structural design theory resource or a research document. The *Manual* provides background information for NDOT's bridge design criteria and application.

3. Example Problems. Where beneficial to explain the intended application, the *Manual* provides example problems demonstrating the proper procedure for selected bridge design applications. These design examples illustrate the specific structural design criteria, practices and policies used by NDOT for the indicated applications.

4. Details. Where beneficial, the *Manual* provides design details for various structural design elements.

5. Coordination with AASHTO *LRFD Bridge Design Specifications*. Part II of the *Manual* is basically a Supplement to the AASHTO *LRFD Bridge Design Specifications* (*LRFD Specifications*) that:

    - in general, does not duplicate information in the *LRFD Specifications*, unless necessary for clarity;
    - elaborates on specific articles of the *LRFD Specifications*;
    - presents interpretative information, where required;
    - modifies sections from the *LRFD Specifications* where NDOT has adopted a different practice;
    - indicates NDOT's preference where the *LRFD Specifications* presents multiple options; and
    - indicates bridge design applications presented in the *LRFD Specifications* but which are not typically used in Nevada.

    In addition, the *Manual* discusses, for selected applications, the original intent in the development of the *LRFD Specifications* to assist the bridge designer in the proper application of the *LRFD Specifications*.

6. Audience. The primary audience for Part II of the *Manual* is the structural design engineer, either experienced or a recent civil engineering graduate. The *Manual* also serves as a resource document for other NDOT Divisions and other transportation agencies.

10.2 NDOT MANUAL APPLICATION

10.2.1 <u>Project Responsibility</u>

The *Manual* applies to all structural design projects under the responsibility of NDOT, including projects designed by:

- NDOT personnel,
- consultants retained by NDOT,
- contractors retained by NDOT for design/build projects,
- local agencies where the project is funded with Federal money, and
- consultants retained by other agencies or private interests for work within NDOT right-of-way (e.g., for permits).

10.2.2 <u>Hierarchy of Priority</u>

Where conflicts are observed in those publications and documents used by NDOT, the following hierarchy of priority shall be used to determine the appropriate application:

- Structural Design Memoranda,
- the *Manual*,
- *LRFD Specifications*, and
- all other publications (see Section 10.4).

10.2.3 <u>Structural Design Exceptions</u>

Section 10.2.3 discusses NDOT's procedures for identifying, justifying and processing exceptions to the structural design criteria in the *Manual* and *LRFD Specifications*.

10.2.3.1 NDOT Intent

The intent of NDOT is that all design criteria in this *Manual* and the *LRFD Specifications* shall be met. However, recognizing that this may not always be practical, NDOT has established a process to evaluate and approve exceptions to its structural design criteria.

10.2.3.2 Procedures

Formal, written approvals for exceptions are only required where the criteria or policies in either the *Manual* or the *LRFD Specifications* are presented in one of the following contexts (or the like):

- "shall,"
- "mandatory," or
- "required."

In addition, at many locations in Part II of the *Manual*, the text specifically states that any proposed exceptions to the indicated structural design criteria must be approved by the Chief Structures Engineer.

Where the bridge designer proposes a design element that does not meet the requirements of the *Manual* or *LRFD Specifications* in the above context, the following procedure will apply:

1. Documentation. The bridge designer will prepare the justification for the exception at the earliest possible stage of the project, which may include:

   - site constraints,
   - design or detailing considerations,
   - construction costs,
   - construction considerations,
   - product availability,
   - environmental impacts, and/or
   - right-of-way impacts.

   The bridge designer will document any proposed exceptions from NDOT's structural design criteria in the Bridge Type Selection Report. See Section 2.6 for information on the Bridge Type Selection Report.

2. Approval. All proposed exceptions must be approved, in writing, by the Chief Structures Engineer. Prepare a Memorandum to the Chief Structures Engineer, which must include:

   - identification of the requesting designer/public entity;

   - project description and project identification number;

   - structure description and structure number;

   - identification of the applicable section of the *NDOT Structures Manual*, NDOT Structural Design Memoranda and/or AASHTO *LRFD Bridge Design Specifications*; and

   - justification for the request.

10.2.4 Scope of Work Definitions

The appropriate application of the structural design criteria in this *Manual* will depend, in part, on the scope of the proposed structural work. For rehabilitations and widenings, compliance with this *Manual* refers to current NDOT practices and policies, not those that were in effect at the time of original construction of existing structures.

10.2.4.1 New Bridge

This scope of work is defined as a new bridge at a new location. The designer shall make every effort to meet the criteria presented in this *Manual* for all new bridge projects.

10.2.4.2 Bridge Replacement

This scope of work is where the existing bridge requires complete replacement of the superstructure, substructure and foundation due to structural or functional deficiencies of the existing structure. The horizontal and vertical alignment of the existing approaching roadway is usually maintained. In general, the designer shall make every effort to meet the criteria presented in this *Manual* for the structural design of bridge replacement projects.

10.2.4.3 Bridge Widening

It may be necessary to widen an existing bridge for a variety of reasons where the existing superstructure and substructure are considered structurally sound. Reasons for bridge widening may include:

1. The existing bridge may provide an inadequate roadway width.

2. The project may include adding travel lanes to a highway segment to increase the traffic-carrying capacity of the facility.

3. A bridge may be widened to add an auxiliary lane across the structure (e.g., increasing the length of an acceleration lane for a freeway entrance, adding a truck-climbing lane, adding a weaving segment at the interior of a cloverleaf interchange).

In general, the designer shall make every effort to meet the criteria presented in this *Manual* for the structural design of the widened portion(s) of bridge widening projects. A determination must be made for whether the existing structure should be strengthened to the same load-carrying capacity as the widening and which AASHTO design standard should be used. For guidance on bridge widening, see Section 22.10.

10.2.4.4 Major Bridge Rehabilitation

The need for major bridge rehabilitation may be based on any number of deficiencies. These may include the following:

- deterioration of structural elements;
- insufficient load-carrying capacity;
- deck replacement or rehabilitation;
- inadequate seismic resistance;
- safety hazard (e.g., substandard bridge rail, substandard guardrail-to-bridge-rail transition); and/or
- geometric deficiencies (e.g., narrow bridge width, inadequate horizontal alignment).

Major bridge rehabilitation may be performed where it is found to be more cost-effective than replacement. This will be determined on a case-by-case basis. As practical, the designer shall meet the criteria presented in this *Manual* for the structural design of bridge rehabilitation projects. For guidance on bridge rehabilitation, see Chapter 22.

10.2.4.5 Minor Bridge Rehabilitation

Minor rehabilitation work will generally be the types of activities listed below and/or those items listed as Safety Work in Section 10.2.4.6. It is not, however, limited to these activities:

- expansion joints,
- deck patching and/or sealing,
- deck waterproofing overlays,
- spot painting of structural steel,
- structural steel fatigue repairs,
- drains and drainage systems,
- grade adjustments, and/or
- concrete coatings.

10.2.4.6 Safety Work

Safety work may be performed as part of a 3R roadway improvement, but it can be performed as a "stand-alone" bridge project to correct a specific safety problem. Safety work may include:

1. Bridge Rail. Substandard bridge rail, substandard guardrail-to-bridge-rail transition, etc., may require upgrading to meet current NDOT criteria. See Section 16.5.1.

2. Anti-Skid Treatment for Decks. If an existing bridge within the limits of a roadway project has low skid numbers, consider deck grinding or placement of a bridge deck overlay, especially if there is a history of wet-weather crashes.

10.3 QUALIFYING WORDS

Many qualifying words are used in structural design and in this *Manual*. For consistency and uniformity in the application of various design criteria, the following definitions apply:

1. Shall, require, will, must. A mandatory condition. Designers are obligated to adhere to the criteria and applications presented in this context or to perform the evaluation indicated. In particular, the use of the word "shall" bears a special meaning. Where used, the designer must meet the criteria or request a structural design exception. See Section 10.2.3 for the exception procedure.

2. Should, recommend. An advisory condition. Designers are strongly encouraged to follow the criteria and guidance presented in this context, unless there is reasonable justification not to do so.

3. May, could, can, suggest, consider. A permissive condition. Designers are allowed to apply individual judgment and discretion to the criteria when presented in this context. The decision will be based on a case-by-case assessment.

4. Desirable, preferred. An indication that the designer should make every reasonable effort to meet the criteria and that the designer should only use a "lesser" design after due consideration of the "better" design.

5. Ideal. Indicating a standard of perfection (e.g., ideal conditions).

6. Minimum, maximum, upper, lower (limits). Representative of generally accepted limits within the structural design community, but not necessarily suggesting that these limits are inviolable. However, where the criteria presented in the "shall" context cannot be met, the designer must seek an exception.

7. Practical, feasible, cost-effective, reasonable. Advising the designer that the decision to apply the design criteria should be based on a subjective analysis of the anticipated benefits and costs associated with the impacts of the decision. No formal analysis (e.g., cost-effectiveness analysis) is intended, unless otherwise stated.

8. Possible. Indicating that which can be accomplished. Because of its rather restrictive implication, this word is rarely used in this *Manual* for the application of structural design criteria.

9. Significant, major (impact). Indicating that the consequences from a given action are obvious to most observers and, in many cases, can be readily measured.

10. Insignificant, minor (impact). Indicating that the consequences from a given action are relatively small and not an important factor in the decision-making for structural design.

11. Standard. A structural design criteria or compilation of criteria that has gained consensus or unanimous acceptance within the structural design community.

12. Guideline or Guide. Indicating a design value that establishes an approximate threshold that should be met if considered practical.

13. Criteria. A term typically used to apply to design values, usually with no suggestion on the criticality of the design value.

14. Typical. Indicating a design practice that is most often used in application and that is likely to be the "best" treatment at a given site.

15. Acceptable. Design criteria which may not meet desirable values, but yet is considered to be reasonable and safe for design purposes.

16. NDOT Practice. A statement that NDOT is presenting its preferred or typical structural design treatment with the expectation that the designer will make every reasonable effort to meet NDOT practice. Exceptions are considered on a case-by-case basis.

17. NDOT Policy. Indicating NDOT practice that NDOT has adopted that the designer is expected to follow, unless otherwise justified. However, formal structural design exceptions are only required where the statement is presented in the "shall" (or a similar) context.

10.4 STRUCTURAL DESIGN LITERATURE (National)

Section 10.4 discusses the major national publications available in the structural design literature. It provides 1) a brief discussion on each publication, and 2) the status and application of the publication by NDOT. Section 10.4 is not all inclusive of the structural design literature; however, it does represent a hierarchy of importance. In all cases, designers must ensure that they are using the latest edition of the publication, including all interim revisions to date.

10.4.1 LRFD Bridge Design Specifications

10.4.1.1 Description

10.4.1.1.1 *General*

The AASHTO *LRFD Bridge Design Specifications* serves as the national standard for use by bridge engineers or for the development of a transportation agency's own structural specifications. The *LRFD Specifications* establishes minimum requirements, consistent with current nationwide practices that apply to common highway bridges and other structures such as retaining walls and culverts; long-span or unique structures may require design provisions in addition to those presented in the *LRFD Specifications*. Interim revisions are issued annually and, periodically, AASHTO publishes a completely updated edition, historically at four-year intervals.

10.4.1.1.2 *LRFD Methodology*

The *LRFD Specifications* presents a load-and-resistance-factor design (LRFD) methodology for the structural design of bridges, which replaces the load factor design (LFD) and service load design (SLD) methodologies of the previous AASHTO *Standard Specifications for Highway Bridges* (*Standard Specifications*). The *LRFD Specifications* applies live-load factors that are lower than the traditional AASHTO load factors but balances this reduction with an increase in vehicular live load that more accurately models actual loads on our nation's highways. Basically, the LRFD methodology requires that bridge components be designed to satisfy four sets of limit states: Strength, Service, Fatigue-and-Fracture and Extreme-Event. Through the use of statistical analyses, the provisions of the *LRFD Specifications* reflect a uniform level of safety for all structural elements, components and systems.

10.4.1.1.3 *Status*

For Federally funded projects, FHWA and the State DOTs have established a goal that the *LRFD Specifications* be used on all new bridge designs after September 2007 for bridges and after September 2010 for culverts, retaining walls and other standard structures. The *LRFD Specifications* reflects a fundamentally different approach to design theory than the *Standard Specifications.* The information in the *LRFD Specifications* supersedes, partially or completely, several AASHTO structural design publications. However, although superseded, some of these publications contain background information or other presentations that may have utility to a bridge designer. The *LRFD Specifications* supersedes the following publications:

1. *Standard Specifications for Alternate Load Factor Design Procedures for Steel Beam Bridges Using Braced Compact Sections*. This publication provides information on the inelastic design of compact steel members (resistance beyond first yield), historically

known as autostress. An Appendix to the *LRFD Specifications* contains an updated inelastic design process for compact steel sections.

2. *Guide Specifications for Strength Design of Truss Bridges*. This document provides provisions for the design of steel trusses using the Load Factor Design (LFD) methodology. Herein, the load combination for long-span bridges (i.e., the Strength IV load combination of the *LRFD Specifications*) first appeared.

3. *Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members*. This document provides recommended requirements for identifying, fabricating, welding and testing of fracture critical, non-redundant steel bridge members whose failure would be expected to cause a bridge to collapse. This document includes specifications on welding requirements that are in addition to those in the ANSI/AASHTO/AWS *Bridge Welding Code*. This document also discusses the need for proper identification of fracture critical members on plans, and it contains useful information addressing background, example problems, etc., that are not included in the *LRFD Specifications*.

4. *Guide Specifications — Thermal Effects in Concrete Bridge Superstructures*. This publication provides guidance on the thermal effects in concrete superstructures with special attention to the thermal gradient through the depth of the superstructure. These provisions have been incorporated into the *LRFD Specifications*.

5. *Guide Specifications for Fatigue Design of Steel Bridges*. This publication provides an alternative procedure to that of the AASHTO *Standard Specifications for Highway Bridges* wherein the actual number of cycles are used for fatigue design. Such a procedure has now been adopted in the *LRFD Specifications*.

6. *Guide Specifications for Design and Construction of Segmental Concrete Bridges*. This document provides details on the design and construction of segmental concrete bridges. The high points have subsequently been included in the *LRFD Bridge Design Specifications* and the *LRFD Bridge Construction Specifications*.

7. *Guide Specification and Commentary for Vessel Collision Design of Highway Bridges*. This publication is a comprehensive document that includes information relative to designing bridges to resist damage from vessel collisions. To the extent feasible, it is based on probabilistic principles. The *LRFD Specifications* contains only the load section of this document. The *Guide Specification and Commentary for Vessel Collision Design of Highway Bridges* contains considerably more information.

10.4.1.1.4 *Significant Features*

A few significant features of the *LRFD Specifications* are:

1. <u>Commentary</u>. The *LRFD Specifications* are supplemented with a comprehensive commentary placed immediately adjacent to the *LRFD Specifications* provisions in a parallel column.

2. <u>Live Load</u>. The vehicular live load is designated HL-93. This live-load model retains a truck configuration similar to the HS-20 design truck and a tandem slightly heavier than the traditional military loading, but the model has been modified to include simultaneously applied lane loading over full or partial span lengths to produce extreme force effects.

3. <u>Load Factors</u>. Maximum and minimum load factors have been introduced for permanent loads that must be used in combination with factored transient loads to produce extreme force effects. The minimum load factors are most significant for substructure design.

4. <u>Fatigue</u>. Fatigue loading consists of a single truck with axle weights and spacings that are the same as an HS-20 truck with a constant 30-ft spacing between the 32-kip axles that can be located anywhere on the bridge deck to produce the maximum stress range.

5. <u>Load Combinations</u>. In addition to regular load combinations, two design trucks within a single lane are used for negative moments and internal pier reactions in combination with the lane load; the distance between the rear and front axles of the trucks cannot be less than 50 ft; and the combined force effect is reduced by 10%.

6. <u>Bridge Decks</u>. The *LRFD Specifications* includes two methods for the design of concrete bridge decks:

   - the traditional bending method; and
   - an empirical deck design, which allows for reduced deck reinforcement based upon assumed internal arching.

   NDOT mandates the exclusive use of the traditional bending method.

7. <u>Live-Load Distribution</u>. The *LRFD Specifications* allows for relatively easy and more precise estimates of live-load distribution by tabulated equations.

8. <u>Deflection</u>. The *LRFD Specifications* allows the optional use of deflection criteria. NDOT policy is that the deflection criteria shall be used on all projects.

9. <u>Compact Steel Sections</u>. The *LRFD Specifications* allows for the more frequent use of compact steel sections.

10. <u>Shear</u>. The method of shear design in concrete has been revised; modified compression field theory and strut-and-tie models are used.

11. <u>Deck Joints</u>. The *LRFD Specifications* recognizes the detrimental effect of salt-laden water seeping through deck joints and promotes the objective of reducing the number of such joints to an absolute minimum.

10.4.1.2 Department Application

10.4.1.2.1 *State Highway System*

NDOT has adopted the use of the *LRFD Specifications* as the mandatory document for the structural design of highway bridges and other structures on the State highway system. Exceptions to this policy are indicated in Section 10.4.2.2.

Part II of the *Manual* presents NDOT's specific applications of the *LRFD Specifications* to structural design, which modify, replace, clarify or delete information from the *LRFD Specifications* for NDOT's application.

10.4.1.2.2 *Off State Highway System*

For highway bridges and other structures not on the State highway system, NDOT's policy is:

1. Federal Funds. For off State highway system projects funded with Federal funds, NDOT policy on the use of the *LRFD Specifications* is identical to projects on the State highway system.

2. Locally Funded Projects. For projects funded with 100% local money, NDOT encourages (but does not require) the use of the *LRFD Specifications* and the *Manual*.

10.4.2 Standard Specifications for Highway Bridges

10.4.2.1 Description

The AASHTO *Standard Specifications for Highway Bridges* (*Standard Specifications*) was first published in the late 1920s with annual interim revisions and, until the adoption of the *LRFD Specifications*, served as the national standard for the design of highway bridges. The final version of the *Standard Specifications* is based on the Service Load Design (SLD) and Load Factor Design (LFD) methodologies. AASHTO maintained the AASHTO *Standard Specifications* through 2000, and published the final comprehensive 17th edition in 2002.

10.4.2.2 Department Application

NDOT only allows the use of the *Standard Specifications* for specific applications, as follows:

- existing elements for bridge widening and bridge rehabilitation projects (Note: Seismic retrofit must be considered independently);
- structural elements for which no LRFD specifications are available (e.g., pedestrian bridges, soundwalls, signs, signals, lighting); and
- other applications as approved by the Chief Structures Engineer.

The minimum highway live load for strength considerations in the application of the *Standard Specifications* shall be HS-20. An HS-25 live-load shall be used on the National Highway System (NHS) and may be considered for other bridges that carry a significant number of trucks. The HS-25 live-load model is defined as 1.25 times the HS-20 live loading as provided in the *Standard Specifications*. In addition, the P Loads discussed in Section 12.3.2.7 shall also be considered for overload provisions on bridges.

10.4.3 Guide Specifications for Seismic Isolation Design

10.4.3.1 Description

AASHTO published the *Guide Specifications for Seismic Isolation Design* as a supplement to the *Standard Specifications for Seismic Design of Highway Bridges*. The *Guide Specifications for Seismic Isolation Design* presents specifications for the design of bearings to seismically isolate the superstructure from the substructure of highway bridges.

10.4.3.2 Department Application

The AASHTO *Guide Specifications for Seismic Isolation Design* should be used, where applicable, in conjunction with the *LRFD Specifications*.

10.4.4 Guide Specifications for Horizontally Curved Highway Bridges

10.4.4.1 Description

The AASHTO *Guide Specifications for Horizontally Curved Highway Bridges* presents specifications and methodologies for the design of steel I-girder and steel box girder bridges that are on a horizontal curve. This document is applicable to simple and continuous spans and to composite or non-composite structures of moderate length employing either rolled or fabricated sections. The design methodology is based on both the service load and load factor design methodologies and, therefore, is not compatible with the *LRFD Specifications*.

10.4.4.2 Department Application

A 2005 interim change to the *LRFD Specifications* integrates horizontally curved girders, both I-shaped and box girders, in common equations for both straight and curved girders. Therefore, NDOT only allows the use of the *Guide Specifications for Horizontally Curved Highway Bridges* for the same applications as for the *Standard Specifications.*

10.4.5 ANSI/AASHTO/AWS Bridge Welding Code D1.5M/D1.5

10.4.5.1 Description

The *Bridge Welding Code* presents current criteria for the welding of structural steel in bridges. The *Code* superseded the 1981 AASHTO *Standard Specifications for Welding of Structural Steel Highway Bridges* and supplements the *Structural Welding Code*, AWS D1.1.

For the first time, with the 2002 edition, the *Code* includes a commentary on selected sections.

10.4.5.2 Department Application

NDOT has adopted the mandatory use of the *Bridge Welding Code D1.5* for the design and construction of structural steel highway bridges. However, for items not specifically addressed in D1.5, such as welding on existing structures or welding on reinforcing steel, refer to the current edition of ANSI/AWS D1.1 and ANSI/AWS D1.4.

10.4.6 Manual on Subsurface Investigations

10.4.6.1 Description

The AASHTO *Manual on Subsurface Investigations* discusses many of the techniques used in the highway industry for subsurface geotechnical investigations. The objective is to describe accepted procedural and technical methods to determine the geotechnical properties of soils and rock strata that will support the highway facility. The range of topics includes data requirements, field reconnaissance, evaluation of geotechnical data, subsurface water impacts, equipment and laboratory techniques.

10.4.6.2 Department Application

NDOT recommends that this publication be used for all subsurface investigations, which is primarily the responsibility of the NDOT Materials Division.

10.4.7 <u>Guide Specifications for Design of Pedestrian Bridges</u>

10.4.7.1 Description

The AASHTO *Guide Specifications for Design of Pedestrian Bridges* applies to bridges intended to carry primarily pedestrian traffic and/or bicycle traffic. This document is not based upon the LRFD design methodology, but is based upon the service load design (SLD) and load factor design (LFD) methodologies.

10.4.7.2 Department Application

The AASHTO *Guide Specifications for Design of Pedestrian Bridges* shall be used by the designer for the design of pedestrian bridges in conjunction with the *Standard Specifications*. The publication shall not be used in conjunction with the *LRFD Specifications*.

10.4.8 <u>Guide Specifications for Distribution of Loads for Highway Bridges</u>

10.4.8.1 Description

The AASHTO *Guide Specifications for Distribution of Loads for Highway Bridges* provides more refined live-load distribution factors than the traditional S-over factors of the *Standard Specifications*. Although the refined equations appear similar, they are not the same as those provided in the *LRFD Specifications* and shall not be used with the *LRFD Specifications*.

10.4.8.2 Application

The AASHTO *Guide Specifications for Distribution of Loads for Highway Bridges* only applies to non-LRFD applications. Therefore, this document is only used when reverting back to the *Standard Specifications* for design.

10.4.9 <u>Guide Design Specifications for Bridge Temporary Works</u>

10.4.9.1 Description

The AASHTO *Guide Design Specifications for Bridge Temporary Works* has been developed for use by State agencies to include in their existing construction Standard Specifications for falsework, formwork and related temporary construction used to construct highway bridge structures.

10.4.9.2 Application

The AASHTO *Guide Design Specifications for Bridge Temporary Works* should be used by the bridge designer, where applicable.

10.4.10 Guide Specifications for Structural Design of Sound Barriers

10.4.10.1 Description

The AASHTO *Guide Specifications for Structural Design of Sound Barriers* provides criteria for the structural design of sound barriers to promote the uniform preparation of plans and specifications. The publication allows the design of masonry sound barriers in addition to concrete, wood, steel, synthetics and composites and aluminum.

10.4.10.2 Department Application

The AASHTO *Guide Specifications for Structural Design of Sound Barriers* shall be used for all sound barrier designs.

10.4.11 Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals

10.4.11.1 Description

The AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals* presents structural design criteria for the supports of various roadside appurtenances. The publication presents specific criteria and methodologies for evaluating dead load, live load, ice load and wind load. This document also introduces the concept of infinite fatigue life for these structures, many of which are non-redundant. This document also includes criteria for several types of materials used for structural supports such as steel, aluminum, concrete and wood.

10.4.11.2 Department Application

NDOT has adopted the mandatory use of the AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals*. The NDOT Traffic Engineering Section is the primary user for these structures, but the Structures Division is responsible for their design.

10.4.12 AISC LRFD Manual of Steel Construction

10.4.12.1 Description

The *LRFD Manual of Steel Construction*, published by the American Institute of Steel Construction (AISC), provides dimensions, properties and general design guidance for structural steel for various applications. The *Manual* contains AISC criteria for steel buildings. However, the properties of the rolled structural shapes are useful for designing bridge structures.

10.4.12.2 Department Application

Designers may use the AISC *LRFD Manual of Steel Construction* to the extent it does not conflict with the *LRFD Specifications*.

10.4.13 AREMA Manual for Railway Engineering

10.4.13.1 Description

The AREMA *Manual for Railway Engineering*, published by the American Railway Engineering and Maintenance-of-Way Association (AREMA), provides detailed structural specifications for the design of railroad bridges. The AREMA specifications have approximately the same status for railroad bridges as the *LRFD Specifications* have for highway bridges; i.e., the structural design of railroad bridges shall meet the AREMA requirements.

10.4.13.2 Department Application

Occasionally, NDOT is responsible for the structural design of railroad bridges over highways. The specifications of the AREMA *Manual* must be met, except as modified by railroad companies operating in Nevada. In addition, the AREMA *Manual* contains the AREMA requirements for the geometric design of railroad tracks passing beneath a highway bridge. As appropriate, these criteria have been incorporated into Chapter 21 of the *NDOT Structures Manual*.

10.4.14 Other Structural Design Publications

The structural design literature contains many other publications which may, on a case-by-case basis, be useful. The following briefly describes several other structural design publications:

1. AF&PA *National Design Specification (NDS) for Wood Construction.* This document, published by the American Forest and Paper Association (AF&PA), provides the reference design values (i.e., nominal resistance and stiffness) for wood products that are tabularized in the *LRFD Specifications*. The *LRFD Specifications* refers to the *NDS* for reference design values for lumber grades not included in the *LRFD* tables. The *NDS* publishes reference values for allowable stress design and provides format conversion factors for the use of these values with the LRFD methodology.

2. AITC *Timber Construction Manual*. This document, published by the American Institute of Timber Construction (AITC), provides comprehensive criteria for the design of timber structures, including bridges. This document contains information for both sawn and laminated timber. The designer should use the AITC *Timber Construction Manual* to supplement the AASHTO publications on the design of timber bridges.

3. *American Concrete Institute (ACI) — Analysis and Design of Reinforced Concrete Bridge Structures*. This publication contains information on various concrete bridge types, loads, load factors, service and ultimate load design, prestressed concrete, substructure and superstructure elements, precast concrete, reinforcing details and metric conversion.

4. American Concrete Institute (ACI) 318-05 *Building Code Requirements for Structural Concrete*. This document addresses the proper design and construction of buildings of

structural concrete. The Code has been written such that it may be adopted by reference in a general building code; earlier editions have been widely used in this manner. A Commentary discusses some of the considerations of the Committee in developing the Code with emphasis on the explanation of new or revised provisions that may be unfamiliar to Code users. Even though this document is intended for building design, bridge designers find it useful because it provides more detail on aspects of concrete design that are less typical in highway bridges.

5. *Concrete Reinforcing Steel Institute (CRSI) Handbook*. This publication meets the ACI Building Code Requirements for Reinforced Concrete. Among other information, it provides values for both design axial load strength and design moment strength for tied columns with square, rectangular or round cross sections, and it provides pile cap designs.

6. *CRSI Manual of Standard Practice*. This publication explains generally accepted industry practices for estimating, detailing, fabricating and placing reinforcing bars and bar supports.

7. *International Building Code*. This document, published by the International Conference of Building Officials (ICBO), provides criteria for the design of buildings throughout the United States and abroad. It is intended to be used directly by an agency or to be used in the development of an agency's own building codes. Buildings for which NDOT is responsible for their design (e.g., at rest areas) shall be designed based on the *International Building Code*. This design is typically the responsibility of the NDOT Architectural Section.

8. *National Steel Bridge Alliance (NSBA) Highway Structures Design Handbook*. This document addresses many aspects of structural steel materials, fabrication, economy and design. Recently updated with LRFD examples in both US customary units and SI units, the general computational procedure is helpful to designers using the *LRFD Specifications*.

9. NCHRP 343 *Manuals for Design of Bridge Foundations*. This publication was produced during the development of the LRFD provisions for foundation design. The publication provides valuable additional information on the application of the *LRFD Specifications* to foundations.

10. PCA *Notes on ACI 318-02 Building Code Requirements for Structural Concrete with Design Applications*. The primary purpose of the PCA Notes is to assist the engineer in the proper application of the ACI 318-02 design standard, which is the predecessor to ACI 318-05. Each chapter of the publication starts with a description of the latest Code changes. Emphasis is placed on "how-to-use" the Code. Numerous design examples illustrate the application of the Code provisions.

11. *Post-Tensioning Institute (PTI) Post-Tensioning Manual*. This publication discusses the application of post-tensioning to many types of concrete structures, including concrete bridges. This publication also discusses types of post-tensioning systems, specifications, the analysis and design of post-tensioned structures and their construction.

12. *Prestressed/Precast Concrete Institute (PCI) Bridge Design Manual*. This two-volume, comprehensive design manual includes both preliminary and final design information for standard girders and precast, prestressed concrete products and systems used for transportation structures. This document contains background, strategies for economy,

fabrication techniques, evaluation of loads, load tables, design theory and numerous complete design examples. This publication is intended to explain and amplify the application of both the *Standard Specifications* and *LRFD Specifications*.

13. *Prestressed/Precast Concrete Institute (PCI) Design Handbook*. This publication includes information on the analysis and design of precast and/or prestressed concrete products in addition to a discussion on handling, connections and tolerances for prestressed products. It contains general design information, specifications, and standard practices.

14. *PTI — Post-Tensioned Box Girder Bridges*. This publication contains information on economics, design parameters, analysis and detailing, installation, prestressing steel specifications, post-tensioning tendons, systems and sources.

15. *United States Navy — Design Manual for Soil Mechanics, Foundations and Earth Structures*. This is a comprehensive document covering embankments, exploration and sampling, spread footings, deep foundations, pressure distributions, buried substructures, special problems, seepage and drainage analysis, settlement analysis, soil classifications, stabilization, field tests and measurements, retaining walls, etc. Note that the loading sections of this document are superseded by the *LRFD Specifications*.

10.5 NDOT DOCUMENTS

NDOT has other publications in addition to the *NDOT Structures Manual* that may apply to a bridge design project. This Section briefly discusses other relevant NDOT publications that may have a significant impact on a bridge design project.

10.5.1 NDOT Project Design Development Manual

The Roadway Design Division is responsible for the *NDOT Project Design Development Manual*. This document provides guidance to the road designer on design criteria and project development. The *NDOT Project Design Development Manual* discusses:

- NDOT organization and the responsibilities of the various Divisions, Offices, Sections, etc., within NDOT;
- road design functions including design controls, design exceptions, access control, geometric criteria, roadside safety, drainage, work zones, cost estimating, contract administration, agreements, CADD, specifications, etc.; and
- project development from planning (scoping) to project letting.

10.5.2 NDOT Geotechnical Design Manual

The Geotechnical Design Section is responsible for the *NDOT Geotechnical Design Manual*. This document presents NDOT's criteria for geotechnical investigations and designs performed by NDOT. The *NDOT Geotechnical Design Manual* discusses:

- reconnaissance surveys;
- field investigations (e.g., subsurface);
- pavement section support (e.g., pavement subgrade, subgrade drainage, erosion control);
- embankments/slopes (e.g., settlement, slope stability);
- foundations for structures (e.g., geotechnical properties);
- retaining walls (e.g., external stability); and
- geotechnical involvement in construction.

10.5.3 NDOT Standard Plans for Road and Bridge Construction

The Specifications Section within the Roadway Design Division is responsible for the *NDOT Standard Plans for Road and Bridge Construction*. The *NDOT Standard Plans* provides details on various design treatments that are consistent from project to project (e.g., guardrail, drainage details). They provide information on how to layout and/or construct the design element.

10.5.4 NDOT Drainage Manual

The Hydraulics Section within the Roadway Design Division is responsible for the *NDOT Drainage Manual*, which presents design criteria on the following topics:

- hydraulic surveys;
- hydrologic methods used in Nevada;
- hydraulic design of culverts, open channels, bridge waterway openings and closed drainage systems; and
- erosion control.

10.5.5 NDOT Standard Specifications for Road and Bridge Construction

The Specifications Section within the Roadway Design Division is also responsible for the *NDOT Standard Specifications for Road and Bridge Construction*. The *Standard Specifications* presents the work methods and materials approved by NDOT for the construction of road, traffic and bridge projects. The publication is divided into three Divisions. Division I “General Requirements” includes information on the administration of a construction project. Division II “Construction Details” addresses the items of work. The sections in Division II are grouped into general work areas such as excavation, plant mix surfacing, concrete structures and guardrail. Each section can have up to five subsections that include:

1. Description. This subsection identifies the type of work addressed in the section.
2. Materials. This subsection identifies the materials that must be used in each item of work. The descriptions in this section are generally short; more extensive materials descriptions are provided in Division III.
3. Construction. This subsection describes how each item of work must be built. The description can be either a method specification, in which the contractor is told how to complete the work, or a performance specification, in which the contractor is provided the end result and is responsible for how the work will be completed.
4. Measurement. This subsection describes how each item of work is measured for payment.
5. Basis of Payment. This subsection describes how each item of work will be paid for under the contract.

Division III “Materials Details” provides a more detailed description of the materials. Each section can have up to three subsections that include:

1. Scope. This subsection describes the general materials addressed in the section.
2. Requirements. This subsection provides general requirements for the materials in this section.
3. Physical Properties and Tests. This subsection provides the specific requirements for each item of work described in this section.

10.5.6 NDOT Construction Manual

The Construction Division is responsible for the *NDOT Construction Manual*. This document is intended for use by construction personnel in the administration of construction contracts, especially the application of the *NDOT Standard Specifications for Road and Bridge Construction*. As such, the *NDOT Construction Manual* addresses each of the items listed for the *NDOT Standard Specifications for Road and Bridge Construction*, but not within the context of a contractual document.

Chapter 11

PRELIMINARY DESIGN

NDOT STRUCTURES MANUAL

September 2008

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Chapter 11
PRELIMINARY DESIGN

This Chapter addresses the preliminary phase of bridge design. It provides guidance to bridge designers in determining the most appropriate overall structure type to meet the structural, geometric, hydraulic, environmental, economical and right-of-way characteristics of the site.

11.1 INTRODUCTION

11.1.1 Objectives of Preliminary Design

Bridge design is accomplished in two equally important phases — preliminary design and final design. Decisions made during preliminary design may significantly impact structure performance, functionality and long-term maintenance.

The preliminary design phase concludes with the selection of a structure type and development of the Structure Front Sheet. The bridge designer must compile the back-up information to support the proposed construction materials, span configuration, superstructure type, abutment type, pier type and location, foundation type, structure dimensions, roadway features, pedestrian features, etc.

The final design process is generally well understood. The preliminary design phase varies from project-to-project. This Chapter assists engineers in preparing preliminary designs for common highway bridges.

Preliminary design includes evaluating many bridge features, which includes the elements of the bridge (foundations, abutments, piers, girders, bearings, expansion joints), materials (concrete, steel) and geometrics (clearances, structure depth, structure width, span lengths). The designer must evaluate each feature to identify the most appropriate selection. High-cost features or those with a "fatal flaw" should be eliminated early in the evaluation process. However, some features may have more than one acceptable solution. This results in the need for an alternatives analysis; see Section 11.8. The superstructure type selection generally drives the need for the alternatives analysis.

11.1.2 Chapter Presentation

In general, this Chapter has been organized to present the decision-making process in preliminary design from the location of the bridge to the structure-type selection for the site.

11.2 BRIDGE LOCATION

Establishing the location of a bridge is an interactive process among NDOT units responsible for roadway, bridge, hydraulics, geotechnical, right-of-way and environment. In addition, District Offices, local governments and the public are involved in determining bridge location. Bridges are an integral part of the transportation system and should be located considering economics, ease of construction, and the minimization of environmental impacts to optimize service to the traveling public. This Section summarizes the significant factors that determine the location of a bridge.

11.2.1 Roadway Design

11.2.1.1 General

Often the considerations for bridge design differ from those for roadway design because the design life of the bridge is 3 to 4 times the design life of the roadway. Roadway design factors which impact bridge location and structure type selection include:

- horizontal alignment (e.g., tangent, curve, superelevation, skew);
- vertical clearances and alignment (e.g., longitudinal gradient, vertical curves);
- traffic volumes;
- roadway width;
- presence of medians, sidewalks and bike lanes; and
- clear zones through underpasses.

The roadway designer establishes the roadway alignment. Ideally, bridges would be located where they are on tangent alignment with no skew, width changes or superelevation transitions. However, project constraints seldom allow this. Bridges are usually located where they fit into the transportation system irrespective of the effect on bridge design and construction. Although bridges can be designed to accommodate almost any given geometry, the bridge designer must work closely with the roadway designer to minimize the adverse effect of some of the following roadway design issues to minimize costs.

Section 11.9 discusses roadway design elements and criteria specifically as they pertain to the roadway design portion of a bridge. Section 11.2.1 discusses roadway design issues specifically as they pertain to bridge location considerations.

11.2.1.2 Horizontal Alignment

Many bridges are constructed on horizontal curves. This complicates the design, geometry and construction of bridges and reduces the number of bridge types that are considered. The analysis of horizontally curved bridges with small radii of curvature requires a refined analysis. Hand-calculation methods are available but are accurate only for horizontally curved bridges of large radii of curvature, and they should be used only as a check of the refined analysis. See Chapter 13 for guidance on acceptable methods of analysis. In general, cast-in-place concrete and structural steel are best suited for horizontally curved bridges.

11.2.1.3 Skew

Skews of less than approximately 30° are acceptable for most bridge types and result in moderate detailing challenges. Some structure types with skews more than 30° may require a refined analysis. All structures with skews of more than 60° should be analyzed by refined methods. Bridges having a high skew may also have long-term functionality problems such as uplifting of girders in the acute corners and/or the bridge bearings translating sideways. Alternatives to these highly skewed bridges should be considered. See Chapter 13 for guidance on acceptable methods of analysis for bridges of varying skew.

11.2.1.4 Vertical Curvature

Vertical curvature is not typically considered in the structural analysis of bridges. The geometry, however, is more difficult, and vertical curvature is reflected in design because the calculated camber for steel and precast concrete girders must be included. All bridges with significant vertical curvature require perpendicular placement of the deck-finishing machine. See Section 11.2.1.7.

11.2.1.5 Variable Width

Most bridges will have a constant width along their entire length. However, ramps and roadway approaches sometimes will extend onto or through a bridge. This can create complex detailing and design challenges. The transitions in bridge width can be either linear or curved. The easiest transitions to detail and design are those that have a linear change in width across the entire length of bridge where the bridge is on a tangent alignment. Detailing and design can become very complex when the bridge is on a horizontal curve with a linear or curved width transition.

11.2.1.6 Superelevation Transition

Superelevation transitions do not create additional structural analysis; however, the geometry is more difficult. Most bridges can be easily constructed with the transitions on a bridge if the transition is constant over the bridge's entire length. Superelevation transitions on only one side of a crown section should be avoided. The deck-finishing machine requires constant adjustment to match the changing roadway crown. All bridges with superelevation transitions require perpendicular placement of the deck-finishing machine.

11.2.1.7 Deck-Finishing Equipment

Deck-finishing machines are used to place, consolidate and finish concrete for bridge decks. Limitations on their use need to be considered in the preliminary design. A deck-finishing machine can finish a deck up to 120 ft in width if the machine is placed perpendicular to the girders. Skews will reduce this width. The roadway geometry, width, width variation, crown, and crown breaks can dictate the deck finishing. Closure pours can be used when multiple longitudinal deck pours are needed. Closure pours must be shown on the contract plans. Locate closure pours where the anticipated traffic wheel lines are away from the longitudinal joints.

11.2.2 Hydraulics

11.2.2.1 General

The Hydraulics Section will prepare a Hydraulics Report or provide preliminary hydraulic recommendations in coordination with the Structures Division's structure-type selection. The critical hydraulic factors may include:

- channel bottom elevation and width;
- water surface elevation for the design-year flood;
- skew angle and side slopes of channel;
- required low-chord elevation;
- bridge scour potential; and
- freeboard required for the passage of debris.

Bridges crossing streams and rivers should be located such that the effects of scour and river meander are minimized. Most river systems in Nevada have the potential for significant scour and meander. Scour is a function of the stream flow, size of bridge opening, pier and abutment locations and widths and soil type. The Geotechnical and Hydraulics Sections will provide preliminary information to determine the potential for scour at each proposed site. Meanders can cause a significant cost increase to the project. Spur dikes and other heavy riverbank armoring are sometimes needed to control a river's meander. The Hydraulics Section should provide preliminary information on the potential for river meander and the cost for its mitigation. Costs for scour and meander mitigations must be included in the evaluation of alternatives if there are differences between the alternatives.

11.2.2.2 Division of Responsibilities

The Hydraulics Section is responsible for hydrologic and hydraulic analyses for both roadway drainage appurtenances and bridge waterway openings. The Hydraulics Section will perform the following for the design of bridge waterway openings for new bridges:

- the hydrologic analysis to calculate the design flow rates based on the drainage basin characteristics;

- the hydraulic analysis to determine the necessary dimensions of the bridge waterway opening to pass the design flood, to meet the backwater allowances and to satisfy any regulatory floodplain requirements; and

- the hydraulic scour analysis to assist in determining the recommended foundation design for the new bridge.

Based on the hydraulic analysis, the Hydraulics Section will provide the following to the Structures Division for new bridges:

- the water surface elevation for the design-year flood and 100-year flood;

- a suggested low-chord elevation;

- the necessary bridge waterway opening dimensions, skew angle, bottom of channel elevation and channel centerline station;

- the results of its hydraulic scour analysis;
- any necessary channel and abutment protection measures; and
- the recommended deck drainage design.

The Hydraulics Section is also responsible for determining that the bridge design is consistent with regulations promulgated by the Federal Emergency Management Agency (e.g., development within regulatory floodplains).

11.2.2.3 Hydraulic Definitions

The following presents selected hydraulic definitions which have an application to bridge design:

1. Auxiliary Waterway Openings. Relief openings provided for streams in floodplains through the roadway embankment in addition to the primary bridge waterway opening.

2. Bridge Backwater Effect. The incremental increase in water surface elevation upstream of a highway facility.

3. Base Flood. The flood having a 1% chance of being exceeded in any given year (i.e., the 100-year event).

4. Base Floodplain. The area subject to flooding by the base flood.

5. Bridge Waterway Opening. The opening provided in the roadway embankment intended to pass the stream flow under the design conditions.

6. Design Flood Frequency. The flood frequency selected for determining the necessary size of the bridge waterway opening.

7. Flood Frequency. The number of times a flood of a given magnitude can be expected to occur on average over a long period of time.

8. Freeboard. The clearance between the water surface elevation based on the design flood and the low chord of the superstructure.

9. Maximum Allowable Backwater. The maximum amount of backwater that is acceptable to NDOT for a proposed facility based on State and Federal laws and on NDOT policies.

10. 100-Year Flood Frequency. A flood volume (or discharge) level that has a 1% chance of being equaled or exceeded in any given year.

11. Overtopping Flood. That flood event that will overtop the elevation of the bridge or roadway approaches.

12. Peak Discharge (or Peak Flow). The maximum rate of water flow passing a given point during or after a rainfall event or snow melt. The peak discharge for a 100-year flood is expressed as Q_{100}.

13. Recurrence Interval (Return Period). For a given discharge, the number of years between occurrences of that discharge. For example, the recurrence interval for a 100-year flood discharge is 100 years.

14. Regulated Floodway. The floodplain area that is reserved in an open manner by Federal, State or local requirements (i.e., unconfined or unobstructed either horizontally or vertically) to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount as established by the Federal Emergency Management Agency (FEMA) for administering the National Flood Insurance Program (NFIP).

15. River Stage. The water surface elevation above some elevation datum.

16. Scour. The action at a bridge foundation in which the movement of the water erodes the channel soil that surrounds the foundation. There are several types of scour:

    a. Contraction. A constriction of the channel (i.e., the flow area) that may be caused, for example, by bridge piers.

    b. Local. Removal of material from around piers, abutments, embankments, etc., due to high local velocities or flow disturbances such as eddies and vortices.

    c. Long-Term. Aggradation and degradation of the stream bed.

17. Thalweg. The path of deepest flow.

18. Standard Flood. The 500-year flood event.

11.2.2.4 Hydraulic Design Criteria

The following summarizes NDOT's basic hydraulic criteria used for the design of bridge waterway openings:

1. Design Flood Frequency. The minimum design flood frequency is based on the roadway classification and ranges from the 10-year event to the 100-year event. The design flood frequency is increased to the 100-year event if necessary to mitigate adverse flood impacts. A 50-year event is typically used for bridges on roadways classified as interstate or principal arterial.

2. Maximum Allowable Backwater. On FEMA-delineated floodways, no backwater may be introduced by the structure. On FEMA-delineated floodplains, 1 ft of maximum backwater may be introduced. For all sites, the maximum allowable backwater shall be limited to an amount that will not result in unreasonable damage to upstream property or to the highway. The Hydraulics Section will determine the allowable backwater for each site.

3. Freeboard. Where practical, a minimum clearance of 2 ft should be provided between the design water surface elevation and the low chord of the bridge to allow for passage of debris. Where this is not practical, the clearance should be established by the bridge and hydraulic designers based on the type of stream and level of protection desired. For example, 6 in may be adequate on small streams that normally do not transport debris. Urban bridges with grade limitations may not provide any freeboard. On bridge replacement projects, efforts should be made to at least match pre-existing low-chord elevations.

4. Scour. The bridge foundation must not fail or be damaged for the scour design event of the 100-year flood. The overtopping flood is used as the design event if less than the

100-year flood. Lesser flood events should be checked if there are indications that less frequent events may produce deeper scour than the 100-year or overtopping flood. The bridge foundation must also be checked using estimated total scour for the lesser of the 500-year or overtopping event. The foundation must not fail while maintaining a minimum geotechnical factor of safety of 1.0 under the appropriate flood scour conditions.

11.2.2.5 Environmental Considerations

See Section 11.2.5 for more discussion.

Many aspects of the environmental assessment with respect to the site are also related to the hydraulic design of a stream crossing. These include the effects on the aquatic life in the stream; effects on other developments, such as a domestic or irrigation water supply intake; and the effects on floodplains.

The evaluation of the typical hydraulic engineering aspects of bridge design are interrelated with environmental impacts. These include the effects of the crossing on velocities, water surface profiles, flow distribution, scour, bank stability, sediment transport, aggradation and degradation of the channel, and the supply of sediment to the stream or water body.

The environmental process for stream crossing projects may also precipitate the need for several State and Federal permits and approvals that are water related. The bridge designer should consider the future requirements for these permits and approvals in the preliminary bridge design stage.

11.2.2.6 Stream Types

The three basic types of streams are braided, straight and meandering.

Hydraulic analysis of braided streams is extremely difficult because of the inherent instability and unpredictable behavior of such streams. Constricting a braided channel into one channel or placing roadway fill between subchannels may change sediment transport capacity at some locations. Where practical, an alternative crossing site at a reach of stream that is not braided should be selected.

A straight reach of stream channel in an otherwise meandering stream may be viewed as a transient condition. Aerial photographs and topographic maps should be examined for evidence of past locations of the channel and of tendencies for meanders to form in the straight reach.

For meandering streams, the outside bank of a bend (i.e., the bank with the longer radius of curvature) presents the greatest hazard to highway facilities because the stream attacks that bank. The design of crossings at bends is complex because it is difficult to predict flood flow distribution. The stream is usually deeper at that bank, velocities are higher, and the water surface is superelevated. The location of a structure in the overbank area may encourage a cutoff and, if the bend system is moving, approach fills and abutments will be subjected to attack as the bend moves downstream.

11.2.2.7 Bridge vs Culvert

In some cases, the waterway opening for a highway-stream crossing can be accommodated by either a culvert or a bridge. Estimates of costs and risks associated with each will indicate which structural alternative should be selected. Figure 11.2-A lists many of the advantages and disadvantages of bridges and culverts. The selection will be a collaborative effort by the bridge designer, Hydraulics Section and the Environmental Services Division. Construction and Maintenance may also become involved.

11.2.2.8 Abutments

The principal hydraulic concerns for abutments are orientation and protection from scour-related failure. Concerns for scour are usually resolved by protective and preventive measures that are identified by the Hydraulics Section. Orientation is usually the same as for adjacent piers.

11.2.2.9 Piers

11.2.2.9.1 *Coordination*

The location of piers in waterways is an interactive process among the Structures Division, Geotechnical Section and the Hydraulics Section. Initially, the Hydraulics Section will determine the required channel geometry to meet the hydraulic criteria (e.g., maximum backwater for 100-year flood). The bridge designer will determine the number and length of spans, types of piers and low-chord elevation. The Hydraulics Section will evaluate the bridge design proposal to determine if it meets the hydraulic requirements of the waterway opening. For example, meeting the hydraulic criteria may require that span lengths be increased. Next, the bridge designer and Geotechnical Section will evaluate potential foundation designs for the pier and provide preliminary design information to the Hydraulics Section for scour analysis. If the resulting foundation design is judged to be too costly, the bridge designer will evaluate reducing the number of piers or eliminating piers altogether based on overall structure costs, environmental impacts, constructibility, etc.

The highway profile (i.e., vertical alignment and bridge end elevations) is an additional highway design element in the iterative process to identify the number and location of piers in waterways. The profile can have a significant impact on the overall bridge opening and floodplain flow conditions. The Roadway Design Division may prefer, for example, to lower the highway profile due to significant right-of-way impacts which, all other factors being equal, reduces the hydraulic capacity of the waterway opening and increases the frequency of overtopping.

Ultimately, all of these factors (i.e., structural, hydraulic, geotechnical, roadway, environmental, costs) must be evaluated to identify the optimum number and location of piers.

11.2.2.9.2 *Costs*

Economy of construction is usually a significant consideration in the determination of spans, pier locations and orientation, and substructure and superstructure design. Construction costs are always a factor in the structural design of a bridge to ensure the use of economically available structural materials, but the cost of construction is only one element of the total economic cost of a stream crossing system. There are hydraulic considerations, maintenance costs and risks of future costs to repair flood damages that should also be factored into the decision on the number of piers and their location, orientation and type.

Bridges	
Advantages	*Disadvantages*
Less susceptible to clogging with drift, ice and debris. Waterway increases with rising water surface until water begins to submerge superstructure. Scour increases waterway opening. Flowline is flexible. Minimal impact on aquatic environment and wetlands. Widening does not usually affect hydraulic capacity. Capacity increases with stage.	Requires more structural maintenance than culverts. Abutment fill slopes susceptible to erosion and scour damage. Piers and abutments susceptible to failure from scour. Susceptible to ice and frost formation on deck. Bridge railing and parapets hazardous as compared to recovery areas. Deck drainage may require frequent maintenance cleanout. Buoyancy, drag and impact forces are hazards to bridges. Susceptible to damage from stream meander migration.
Culverts	
Advantages	*Disadvantages*
More roadside recovery area can be provided. Grade raises and widening projects sometimes can be accommodated by extending culvert ends. Requires less structural maintenance than bridges. Frost and ice usually do not form before other areas experience the same problems. Capacity increases with stage. Capacity can sometimes be increased by installing improved inlets. Usually easier and quicker to build than bridges. Scour is localized, more predictable and easier to control. Storage can be utilized to reduce peak discharge. Avoids deep bridge foundations.	Multiple barrel culverts, whose width is considerably wider than the natural approach channel, may silt in and may require periodic cleanout. No increase in waterway as stage rises above soffit. May clog with drift, debris or ice. Possible barrier to fish passage. Susceptible to erosion of fill slopes and scour at outlets. Susceptible to abrasion and corrosion damage. Extension may reduce hydraulic capacity. Inlets of flexible culverts susceptible to failure by buoyancy. Rigid culverts susceptible to separation at joints. Susceptible to failure by piping and/or infiltration.

BRIDGE vs CULVERT
(Hydraulics)

Figure 11.2-A

11.2.2.9.3 *Design Factors*

The number of piers in any channel should be limited to a practical minimum, and piers in the channel of small streams should be avoided, if practical. The cost of construction of a pier in the water increases with increasing water depth. Piers properly oriented with the flow do not contribute significantly to bridge backwater, but they do contribute to general scour. In some cases, severe scour can develop immediately downstream of bridges because of eddy currents and because piers occupy a significant area in the channel. Lateral and vertical scour also occur at some locations.

Piers should be aligned with flow direction at flood stage to minimize the opportunity for drift to be caught in piling or columns, to reduce the contraction effect of piers in the waterway, to minimize debris forces and the possibility of debris dams forming at the bridge, and to minimize backwater and local scour. Pier orientation is difficult where flow direction changes with stage or time. Circular piers, or some variation thereof, may be the best alternative if orientation at other than flood stage is critical.

Piers located on a bank or in the stream channel near the bank are likely to cause lateral scouring of the bank. Piers located near the stream bank in the floodplain are vulnerable because they can cause bank scour. They are also vulnerable to failure from undermining by meander migration.

Pier shape is also a factor in local scour. A solid pier (e.g., a pier wall) will not collect as much debris as a pier bent or a multi-column pier. Rounding or tapering the leading edges of piers helps to decrease the accumulation of debris and reduces local scour at the pier.

11.2.2.10 Foundations

The foundation is usually the element of a bridge that is most vulnerable to attack by floods. Examination of individual boring logs and plots of the profiles of various subsurface materials are important to the prediction of potential scour depths and to the estimation of the bearing capacity of the soils.

Driven piles or drilled shafts usually depend upon the surrounding material for skin friction and lateral stability. In some cases, they can be extended to rock or other dense material for load-carrying capacity through tip resistance. Tip elevations for piling or drilled shafts should be based on estimates of potential scour depths and bearing to avoid losing lateral support and load-carrying capacity during floods.

The bridge designer must consider the potential scour and the possibility of channel shifts in designing foundations for bridges on floodplains and spans approaching the stream channel. The thalweg (i.e., the line or path connecting the lowest flow points along the channel bed) should not be considered to be in a fixed location when establishing founding elevations. The history of a stream and a study of how active it has been can be useful in making decisions on pile and drilled shaft tip elevations.

11.2.2.11 Bridge Deck Drainage

See Section 16.4.

11.2.3 Geotechnical

Foundations can be either shallow (spread footing) or deep (driven piles and drilled shafts). Shallow foundations are significantly less expensive than deep foundations. The Geotechnical Section provides preliminary foundation information for a proposed site. Bridges located at different sites have different foundation requirements. These differences must be included in an alternative's cost. In addition, the structure type can be influenced by the supporting soils. Heavy bridges such as cast-in-place concrete box girders can require a substantially larger foundation compared to a structural steel bridge. See Section 11.7.

11.2.4 Right-of-Way

Right-of-way and utilities have a significant influence on most projects. Their cost can be as high as the cost of construction; therefore, the location of a bridge must be carefully considered. In addition, right-of-way acquisition and utility relocations can require a significant timeframe to complete. The Right-of-Way Division can provide preliminary estimates on the number of properties and utilities for each bridge location. The Division will provide estimates on cost, number of properties and utilities encountered, possible difficult acquisitions, and approximate time frames. In addition to property acquisition, most projects require temporary and permanent easements for construction staging areas, access, future maintenance and actual construction. These must also be considered when evaluating alternatives.

The designer should consider the following right-of-way factors when selecting the structure type:

1. Expensive Right-of-Way. If right-of-way will be expensive, this may lead to the use of retaining walls and other measures to reduce right-of-way impacts.

2. Structure Depth. The available right-of-way at the bridge site may affect the vertical alignment of the structure which may, in turn, affect the acceptable structure depth to meet the vertical clearance requirements. The depth of the superstructure is a significant issue in urban areas. Right-of-way acquisition costs are high, and roadway profiles cannot usually be raised due to access rights on approaches. All costs including approach costs, right-of-way acquisition, easements, etc., for each alternative must be included in an alternatives analysis.

3. Detours. For bridge widening projects, if right-of-way is not available for detours, it may be necessary to maintain traffic across the existing bridge during widening.

Any bridge design must be consistent with NDOT utility accommodation policies. Section 16.5.4 discusses utility attachments to bridges.

11.2.5 Environmental

11.2.5.1 General

The evaluation of potential environmental impacts can have a significant impact on bridge location, structure-type selection and configuration, especially for highway bridges over streams. In general, any bridge project should, within reason, attempt to minimize the environmental impacts, especially in sensitive areas (e.g., wetlands, endangered species habitat). The Environmental Services Division is responsible for identifying all environmental resources within the proposed project limits and for evaluating the potential project impacts on these resources.

In addition, the Environmental Services Division is responsible for ensuring that the State and Federal requirements for public involvement are met.

Cultural resources, endangered species (including plants and habitat), and other environmental concerns must be identified at each bridge location. Almost all current NDOT right-of-way has been cleared for cultural resources and endangered species. Although projects within NDOT right-of-way may not need extensive evaluation, these projects could still have environmental issues. The proposed location of the bridge can be of no consequence, require some form of mitigation, or be so significant that the bridge must be moved to another location. See the *NDOT Environmental Services Manual* for a detailed discussion on environmental considerations and permits.

The following Sections discuss specific environmental impacts and actions that may be required for a bridge project.

11.2.5.2 Environmental Class of Action

For every NDOT project, the Environmental Services Division will determine the Environmental Class of Action. Based on the results of the evaluation of project impacts and the nature and scope of the proposed project, the Division will determine the level of NEPA compliance processing for the project. This will be one of the following:

1. Categorical Exclusion. A Categorical Exclusion (CE) is issued for categories of projects that do not individually or cumulatively have a significant effect on the environment and, therefore, do not require the preparation of an EA or EIS.

2. Environmental Assessment. An Environmental Assessment (EA) is prepared for projects for which the significance of the environmental impact is not clearly established.

3. Environmental Impact Statement. An Environmental Impact Statement (EIS) is prepared for projects where it is known that the action will have a significant effect on the environment. Mitigation measures are typically required to be incorporated into the proposed improvements if environmental impacts are deemed significant.

11.2.5.3 Permits/Approvals

A proposed bridge project may precipitate the need for one or more environmental permits or approvals. Except for floodplains and permits with the Tahoe Regional Planning Agency (TRPA) (which are the responsibility of the Hydraulics Section), the Environmental Services Division is responsible for coordinating with the applicable Federal or State agency and acquiring the permit or approval. This will require considerable coordination with the Structures Division. The following sections briefly discuss these permits/approvals.

11.2.5.3.1 *US Army Corps of Engineers Section 404 Permit*

The Section 404 Permit is required for the discharge of dredge or fill material into any waters of the United States, including wetlands. The purpose of Section 404 is to restore and maintain the chemical, physical and biological integrity of the Nation’s waters through the prevention, reduction and elimination of pollution.

11.2.5.3.2 *Section 401 Water Quality Certification*

Pursuant to Section 401 of the Clean Water Act, the Section 401 Water Quality Certification is issued by the Nevada Division of Environmental Protection (NDEP) based on regulations issued by the US Environmental Protection Agency. The purpose of the Section 401 Certification is to restore and maintain the chemical, physical and biological integrity of the Nation's waters through prevention, reduction and elimination of pollution. A Section 401 Certification (or waiver of Certification) is required in conjunction with any Federal permit (e.g., a Section 404 Permit) to conduct any activity that may result in any discharge into waters of the United States.

11.2.5.3.3 *Section 402 NPDES Permit*

Pursuant to Section 402 of the Clean Water Act, the Section 402 National Pollutant Discharge Elimination System (NPDES) Permit is issued by NDEP based on regulations issued by the US Environmental Protection Agency. The purpose of the Section 402 Permit is to restore and maintain the chemical, physical and biological integrity of the Nation's waters through prevention, reduction and elimination of pollution.

11.2.5.3.4 *US Coast Guard Section 9 Permit*

Pursuant to Section 9 of the Rivers and Harbors Act of 1899, the Section 9 Permit is issued by the US Coast Guard. The purpose of the Section 9 Permit is to protect and preserve the navigable waterways of the United States against any degradation in water quality. The Permit is required for structures or work (other than bridges or causeways) affecting a navigable waterway (tidal or non-tidal). Examples of work include dredging, channelization and filling. The Colorado River in Southern Nevada is the only river in the state that requires a Section 9 Permit.

11.2.5.3.5 *Floodplains Encroachment*

Pursuant to Executive Order 11988 "Floodplain Management," NDOT must seek approval from FEMA for any Federally funded/regulated project that produces a significant floodplain encroachment. If a project will have a significant floodplain encroachment, the project will require either an EA or EIS. A proposed action that includes a significant floodplain encroachment will not be approved unless FHWA finds (pursuant to 23 CFR650A) that the proposed action is the only practical alternative.

In addition, NDOT must secure a TRPA Permit when working in the Tahoe basin.

11.2.5.4 Historic Bridges

Based on Section 106 of the National Historic Preservation Act of 1966 (as amended), NDOT must consider the effects of the project on properties included in or eligible for inclusion in the National Register of Historic Places (NRHP). Where such properties will be affected, the Advisory Council on Historic Preservation (ACHP) must be afforded a reasonable opportunity to comment on the undertaking. NDOT must implement special efforts to minimize harm to any property on or eligible for the NRHP that may be adversely affected by the proposed project. The mitigation is accomplished through written agreements among NDOT, the ACHP and the Nevada State Historic Preservation Officer (SHPO). This applies not only to historic bridges, but to other historic properties within and adjacent to NDOT right-of-way.

11.2.5.5 Hazardous Waste

The Hazardous Waste Section within the Environmental Services Division is responsible for identifying and evaluating hazardous waste sites and for determining the needed mitigation measures. Three specific types of hazardous waste that may require treatment for a bridge project include:

1. Paint Removal. Removal of paint from steel bridges that may contain heavy metals or from concrete bridges that may contain asbestos.

2. Fine Surface Finish. This type of concrete finish may contain asbestos or heavy metals.

3. Timber Removal. Salvaging or disposing of timber, from an existing bridge, that may contain creosote or other wood preservative.

4. Plates. Asbestos blast plates on railroad overpasses.

11.2.5.6 Construction

For information on construction-related environmental impacts, reference the following NDOT publications:

- *Planning and Design Guide*; and

- *Construction Site Best Management Practices*, which discusses temporary stream crossings and clear water diversions.

11.2.5.7 Other Environmental Impacts

Occasionally, a proposed bridge project may precipitate other environmental impacts. These include Section 4(f), Section 6(f), Section 106 (other than historic bridges) and threatened and endangered species. Contact the Environmental Services Division for more information.

11.2.6 District

The District Office must be consulted when selecting a bridge location. District input is typically via a review of the Type and Size Report (TSR) and/or Front Sheet. District maintenance and construction personnel can assist with local knowledge of the area including potential political issues, usage of the roadway, possible detours, effect of falsework on the transportation system and other issues. The District Office must also approve non-standard temporary vertical clearances on State Highways.

11.2.7 Local Governments

Local agencies are involved in most projects. Coordination is usually with the local agency's engineering staff, planning staff and/or consultant. Presentations to the local government's elected officials are necessary on some projects. The local government input should be considered but, in most cases for projects on the State Highway System, the final decision is made by NDOT. However, some projects include local funding or replacement of a locally owned bridge. In these cases, local input and approval is necessary. Most local governments have limited funding, and enhancements beyond the minimum requirements allowed in the Nevada Bridge Program cannot usually be included. Local agencies must also approve non-standard temporary vertical clearances on their roads.

11.2.8 General Public

Input from the general public will be sought at informational meetings and public hearings. The information collected at these meetings should be considered, but the final decision is made by NDOT. Formal public hearings completed under NEPA have greater weight than an informational meeting. The Environmental Services Division should be contacted concerning the comments received at public hearings to discuss the relevance of the comments and the required action to be taken.

11.3 SPAN LENGTH AND CONFIGURATION

11.3.1 General

The total required length of a bridge is, in most cases, fairly easy to determine. Determining the optimum number of spans is more difficult. This depends upon the:

- roadway profiles;
- vertical clearances;
- construction requirements (e.g., river diversions, falsework openings);
- environmental factors;
- depth of structure;
- allowable locations of piers;
- foundation conditions;
- waterway opening requirements;
- safety of underpassing traffic;
- navigational requirements; and
- flood debris considerations.

Initially, the bridge designer should consider using a single span. This is usually ideal for most moderate length bridges. Spans up to 225 ft are achievable using cast-in-place, post-tensioned box girders or structural steel plate girders. However, for these span lengths, a fairly deep structure is required, which increases approach roadway costs. The additional approach roadway costs must be included in an alternatives analysis. See Section 11.8.

11.3.2 Waterway Crossings

Abutments for bridges crossing streams and rivers are usually placed at the banks of the river so that the bridge does not affect flow. In addition, abutments can also be placed a sufficient distance back from the edge of the banks (outside the "ordinary high-water elevation") to keep excavations, backfill and riprap out of the river eliminating the need for a US Army Corps of Engineers Section 404 permit. Generally, Section 404 permits are easy to obtain, but some river systems have construction windows, endangered species and water quality requirements that greatly restrict construction activities below the "ordinary high-water elevation."

Piers are expensive, time consuming to construct and reduce the hydraulic opening. Their use should be minimized. Each pier is assumed to collect debris during a flood, which further reduces the hydraulic opening and increases scour. However, more supports allow for a shallower superstructure depth. Streams and rivers almost always require deep foundations. A bridge with foundations that remain out of the water greatly reduces foundation costs and can in many cases be the least cost alternative. Access into a stream or river (usually through adjacent property), pile drilling and driving equipment logistics, river diversions, settling basin requirements, environmental restrictions and risk of flooding greatly increase the cost of placing a support or multiple supports in a stream or riverbed.

See Section 11.2.2 for a further discussion on hydraulic considerations for bridge design.

11.3.3 Highway Crossings

Highway bridges over other highways should have their abutments set based on the anticipated future width requirements of the highway beneath the bridge. The number of lanes and shoulder widths are based on 20-year traffic projections. However, the *LRFD Specifications* provides a 75-year design life for bridges. Traffic projections to 75 years are highly speculative; however, some provision for future widening beyond 20 years should be considered. Open abutments can accommodate the future construction of a retaining wall to increase the width of roadway under the bridge. Locate the abutment a sufficient distance back to allow for the placement of a conventional retaining wall. The excavation for a future retaining wall should not influence the active pressure of the abutment spread footing. If the retaining wall influences the abutment spread footing, a tieback (ground anchor) retaining wall will be required. Section 11.9.6.1 discusses this further.

Bridges in rural areas generally do not need consideration for widening beyond 20 years, and additional span length should not be included. However, in urban areas, the roadway under a bridge may be widened several times over the life of the bridge. The Planning Division and Roadway Design Division may provide an estimate of the potential maximum build out in a community. The number of piers should be minimized with consideration given to clear spanning. No pier should be placed in the area of potential widening between the abutment and roadway. Most highway-over-highway bridges can accommodate a pier in the median because medians are also used for barrier rail, lighting and sign supports. Consistency of structure type along a corridor should also be considered.

11.3.4 Railroad Crossings

The rationale for the location of abutments for highway bridges over railroads is similar to highway-over-highway bridges. Pier locations, however, are different. Generally, the railroad requires that its tracks and maintenance roads be clear spanned. Typically, three-span and single-span bridges are built over railroad facilities. See Section 21.1 for more information.

11.3.5 Urban Bridges and Structure Depth

The depth of the superstructure is a significant issue in urban areas. Right-of-way acquisition costs are high, and roadway profiles cannot usually be raised due to access rights on approaches. All costs, including approach roadway costs, right-of-way acquisition, easements, etc., for each alternative must be included in an alternatives analysis.

11.3.6 Cantilever End Spans

Bridges with cantilever end spans are structurally efficient, have shallow superstructure depths, and require only a small retaining wall to support the approach fill. However, they have demonstrated poor performance and should be avoided. The end spans move up and down with seasonal temperature changes and may have a permanent movement upward due to long-term superstructure creep and shrinkage. This creates a “bump” at certain times of the year that can be a hazard. Long-term maintenance problems often develop due to the need for overlays and expansion joint reconstruction.

11.4 GENERAL DESIGN CONSIDERATIONS

As discussed in this Section, the bridge designer must evaluate certain general design factors in the selection of the structure type and size.

11.4.1 <u>Definition of Terms</u>

11.4.1.1 Substructure vs Foundation

Foundations include the supporting rock or soil and those bridge elements that are in direct contact with, and transmit loads to, the supporting rock or soil. In the *NDOT Structures Manual*, this definition will be used. Typically, foundations include piles, drilled shafts, spread footings and pile caps.

11.4.1.2 Substructure vs Superstructure

The *NDOT Structures Manual* will refer to the substructure as any component or element (not including the foundation) supporting the bearings. The superstructure then consists of the bearings and all of the components and elements resting upon them. For those supports without bearings (e.g., integral abutments, fixed columns to integral caps), the distinction between superstructure and substructure can be blurred.

11.4.1.3 Concrete Slab vs Bridge Deck

A concrete slab (whether cast-in-place/precast or conventionally reinforced/post-tensioned) refers to a superstructure consisting solely of a concrete slab constructed without any supporting girders. The *LRFD Specifications* refers to this as a slab superstructure. A bridge deck refers to a concrete slab supported on longitudinal or transverse supporting components (e.g., girders, beams).

11.4.2 <u>Live-Load Deflection Criteria</u>

Reference: LRFD Articles 2.5.2.6.2 and 2.5.2.6.3

11.4.2.1 General

The *LRFD Specifications* states that the traditional live-load deflection criteria is optional for bridges both with and without sidewalks because static live-load deflection is not a good measure of dynamic excitation. Nonetheless, in the absence of a better criterion and because of durability concerns, NDOT has determined that it is appropriate to limit live-load deflections. The live-load deflection criteria of the *LRFD Specifications* are calibrated to yield comparable results for the HL-93 notional live-load model as the provisions of the *Standard Specifications for Highway Bridges* with the HS20-44 live-load model. Therefore, NDOT mandates the optional live-load deflection check.

11.4.2.2 Criteria

The bridge designer shall limit the live-load deflections to the span-length-based criteria of Section 11.5.1.4 and considering the presence or the absence of pedestrian traffic. The minimum superstructure depth limits of Section 11.5.1.4 shall also be met. Section 11.5.1.4 provides limits based upon provisions of the *LRFD Specifications* for NDOT-specific superstructures types.

11.4.3 <u>Continuous vs Simple Spans</u>

In general, continuous structures provide superior structural performance when compared to bridges with simple spans and joints, and their use is strongly recommended for multi-span bridges. However, in rare cases, it may be appropriate to use simple spans (e.g., widenings of existing simple-span bridges), where high differential settlements are anticipated, and for longer spans or other geometric constraints. Back-to-back multiple simple spans should be avoided, if possible.

11.4.4 <u>Jointless Bridges</u>

NDOT prefers to use, where possible, integral or dozer abutments. When such abutments are used and when estimated total abutment movements are less than ½ in, the approach slab may be tied directly to the abutment (no joint). In this case, provision for bridge movement shall be made at the roadway end of approach slabs. For greater movements with integral or dozer abutments and for all other abutment types, an expansion joint shall be used at the bridge end of approach slabs.

11.4.5 <u>Slab-on-Girder Bridges</u>

The following discussion on slab-on-girder bridges does not apply to cast-in-place, post-tensioned concrete box girders.

11.4.5.1 Composite Action

Reference: LRFD Articles 4.5.2.2 and 9.4.1

Composite action enhances the stiffness and economy of girder bridges by using the bridge deck as an integral part of the girder cross section. NDOT policy is that all bridge decks and their supporting members shall be made fully composite throughout the entire span of the bridge, in both positive and negative moment regions. Thus, the shear connectors and other connections between decks and their supporting members shall be designed to develop full composite action. In other words, the shear connections must be able to resist the horizontal interface shear at the nominal resistance of the section.

The stiffness characteristics of composite girders shall be based upon full participation of the effective width of the concrete deck in the positive moment regions. Composite concrete bridge decks shall be considered uncracked throughout the span for the determination of moments and shears for Service and Strength limit states in structural analysis.

11.4.5.2 Number of Girders

Because of concerns for redundancy, new bridges shall have a minimum of four girders per span. An exception is for narrow bridges on low-volume roads where a minimum of three girders may be used.

The cost of a girder bridge increases with the number of girders in the cross section. Conversely, structure redundancy increases with the number of girders. The basic objective is to identify a girder spacing and corresponding number of girders that optimizes the design of the superstructure by providing sufficient redundancy with minimal cost. In addition, the designer should consider the structural implications on maintaining traffic across the bridge during future operations to redeck or widen the bridge.

11.4.5.3 Interior vs Exterior Girders

Reference: LRFD Article 4.6.2.2.1

To simplify future bridge widenings and for economy of fabrication, all girders within a span should be designed identically to the governing condition, either interior or exterior girder. This also eliminates the possibility of misplacement during construction.

11.4.6 Seismic Requirements

Reference: LRFD Articles 3.10, 4.7.4, 5.10.11, A10.1-3 and 11.6.5

The bridge designer shall incorporate the seismic requirements of the *LRFD Specifications* with the selection of a superstructure, substructure or foundation type. The seismic demand of the bridge and the flexibility/stiffness of the bridge are coexistent. Therefore, the structure-type selection must satisfy the seismic performance, ductility requirements, plastic hinge location, etc., as specified in the *LRFD Specifications*.

Ideally, bridges should have a regular configuration so that seismic behavior is predictable and so that plastic hinging is promoted in multiple, readily identifiable and repairable yielding components. Selecting a structural form based solely on gravity-type loading considerations and then adding seismic-resistive elements and details is unlikely to provide the best solution.

Although the *LRFD Specifications* seismic provisions do not discuss preliminary structure-type selection, certain guidelines should be followed. In general, structure type should be selected with the following considerations:

1. Alignment. Straight bridges are preferred because curved bridges can lead to unpredictable seismic response.

2. Substructure Skew. Substructure units should have little or no skew. Skewed supports cause rotational response with increased displacements.

3. Superstructure Weight. Superstructure weight should be minimized.

4. Joints. The deck should have as few expansion joints as practical.

5. Foundations. Shallow foundations should be avoided if the foundation material is susceptible to liquefaction.

6. Substructure Stiffness. Large differences in the stiffness of the substructure units should be avoided. Seismic forces should be uniformly distributed to all substructure units. This can be accomplished by varying the cross section, providing isolation casings, or strategically locating pinned vs fixed column ends.

7. Plastic Hinges. The formation of plastic hinges should be avoided if at all possible. Where their formation is unavoidable, plastic hinges should be forced to develop only in the columns rather than the cap girders, superstructure or foundations, and they should be accessible for inspection and repair after an earthquake.

11.4.7 Approach Slabs

Approach slabs are required on all bridges.

11.4.8 Foundation Considerations

The following applies, in general, to foundation considerations:

1. Grade Adjustment. When considering structure-type selection, the ability to adjust the structure through jacking is an important issue, which is required by LRFD Article 2.5.2.3. Jacking stiffeners or diaphragms may be required. The subgrade may settle differently from the calculated estimates. It is understood that, where superstructures and substructures are integral with each other, this facilitation for adjustment cannot exist.

   The nature of the subgrade should be considered prior to the final selection and design of the superstructure, substructure and foundation to ensure adjustability if needed.

2. Settlement Limits. Experience demonstrates that bridges can accommodate more settlement than traditionally allowed in design due to creep, relaxation and redistribution of force effects. LRFD Article 10.6.2.2.1 mandates that settlement criteria be developed consistent with the function and type of structure, anticipated service life and consequences of unanticipated movements on service performance. Further, in the commentary it suggests that longitudinal angular distortions between adjacent spread footings greater than 0.008 radians in simple spans and 0.004 radians in continuous spans should not be ordinarily permitted.

11.4.9 Aesthetics

Reference: LRFD Article 2.5.5

11.4.9.1 General

Structures should be aesthetically pleasing to the traveling public. The *LRFD Specifications* emphasizes and NDOT encourages the objective of improving the appearance of highway bridges in the State. The Landscape/Aesthetics Section has developed the *NDOT Landscape and Aesthetics Master Plan*, which presents NDOT's policies, procedures and practices for incorporating aesthetic features into NDOT projects. The following discussion presents a brief overview on aesthetic practices for bridges. See the *Master Plan* for more information.

11.4.9.2 Aesthetic Classifications for Structures

The following classifications present a common language for the aesthetics of highway structures, in order from the least sophisticated application to the most sophisticated:

- Standard Structures,
- Accentuated Structures,
- Focal Structures, and
- Landmark Structures.

11.4.9.3 Design Guidelines for Bridges

The bridge designer should adhere to the following design guidelines for aesthetic treatments of bridges:

- Use a consistent bridge design.
- Use simple substructure and support features.
- Use visually light bridge barrier railings.
- Consider fill embankments and approach barriers as part of the bridge design.
- Use landscape or rock mulch to stabilize embankments.
- Select vandalism-resistant finishes.
- Create a visual design unity among all existing and new structures.
- Integrate landscape and aesthetics at the onset of project planning.
- Use a uniform, consistent color palette for all highway structures, and ensure that accent colors highlight structural aspects.

11.4.10 Construction

11.4.10.1 General

The *LRFD Specifications* requires that, unless there is a single obvious method, at least one sequence of construction should be indicated in the contract documents. If an alternative sequence is allowed, the contractor should prove that stresses that accumulate in the structure during construction will remain within acceptable limits.

11.4.10.2 Access and Time Restrictions

Water-crossing bridges will typically have construction restrictions associated with their construction. These must be considered during structure-type evaluation.

The time period that the contractor will be allowed to work within the waterway may be restricted by regulations administered by various agencies. Depending on the time limitations, a bridge with fewer piers or faster pier construction may be more advantageous even if more expensive.

11.4.10.3 Staged Construction

Occasionally, due to the proximity of existing structures or a congested work area, it may be necessary to build a structure in multiple stages. The arrangement and sequencing of each stage of construction is unique to each project, and the bridge designer must consider the requirements for adequate construction clearances and the requirements of the traveling public. If staged construction is required, then a staging sequence and controlling lane/construction dimensions must be shown in the contract documents.

11.4.10.4 Construction Costs

Initial construction cost is one factor in the selection of the structure type, but not the only factor. Future expenditures during the service life of the bridge should also be considered. The initial costs depend on a variety of factors including:

- type of structure,
- economy of design,
- market conditions,
- experience of local contractors,
- vicinity of fabrication shops, and
- local availability of structural materials and labor.

These factors may change rapidly, and the designer may have no control over them. It may be advisable to prepare competitive plans (i.e., for both concrete and steel superstructures) occasionally even for small-span structures. A review of Post-Construction Reports on completed bridges may avoid future errors.

11.4.10.5 Falsework

Temporary falsework is an expensive construction item. If the bridge is over a waterway and/or will have a high finished elevation, the cost of the falsework may become prohibitive, and the designer should consider other structural systems.

The following will apply to the use of falsework:

1. Railroads. Each railroad company has its own requirements for falsework over its facilities. Depending on the railroad company and the type and amount of railroad traffic, the railroad company may prohibit the use of falsework. The railroad company should be contacted early in project development to determine if falsework may be used and its minimum clearance requirements. See Chapter 21 for more information.

2. Environmental. Some sites may be very environmentally sensitive, and the use of falsework may be prohibited.

3. Hydraulics. For falsework over a waterway, the Hydraulics Section will provide the minimum falsework opening dimensions.

4. Traffic Impacts. Constructing falsework over traffic poses a number of risks. Installing and removing falsework requires extended lane closures or expensive traffic cross-overs. Vehicular impacts to falsework can pose a hazard to the traveling public and construction works. Impact girders that protect the falsework can be constructed on low-volume roads with low posted speed limits. However, the impact girder itself may also

be a hazard. Increasing the vertical clearance to the falsework and using an over-height detection system are more positive methods to reduce risk. See Section 11.9.6.3.

5. Traffic Capacity. The volume and composition of traffic will impact the falsework opening, which must provide sufficient capacity to accommodate the traffic flow.

Figure 11.4-A presents a schematic of a typical falsework opening.

11.4.10.6 Drainage

Refer to Section 3.3.10 of the *NDOT Drainage Manual* for drainage considerations during construction.

11.4.11 Maintenance and Durability

The structure-type selection will, over the life of the structure, have a major impact on maintenance costs. Based on type of material, the following is the approximate order of desirability from a maintenance perspective:

- prestressed concrete,
- reinforced concrete,
- unpainted weathering steel, and
- painted structural steel.

The following maintenance considerations apply:

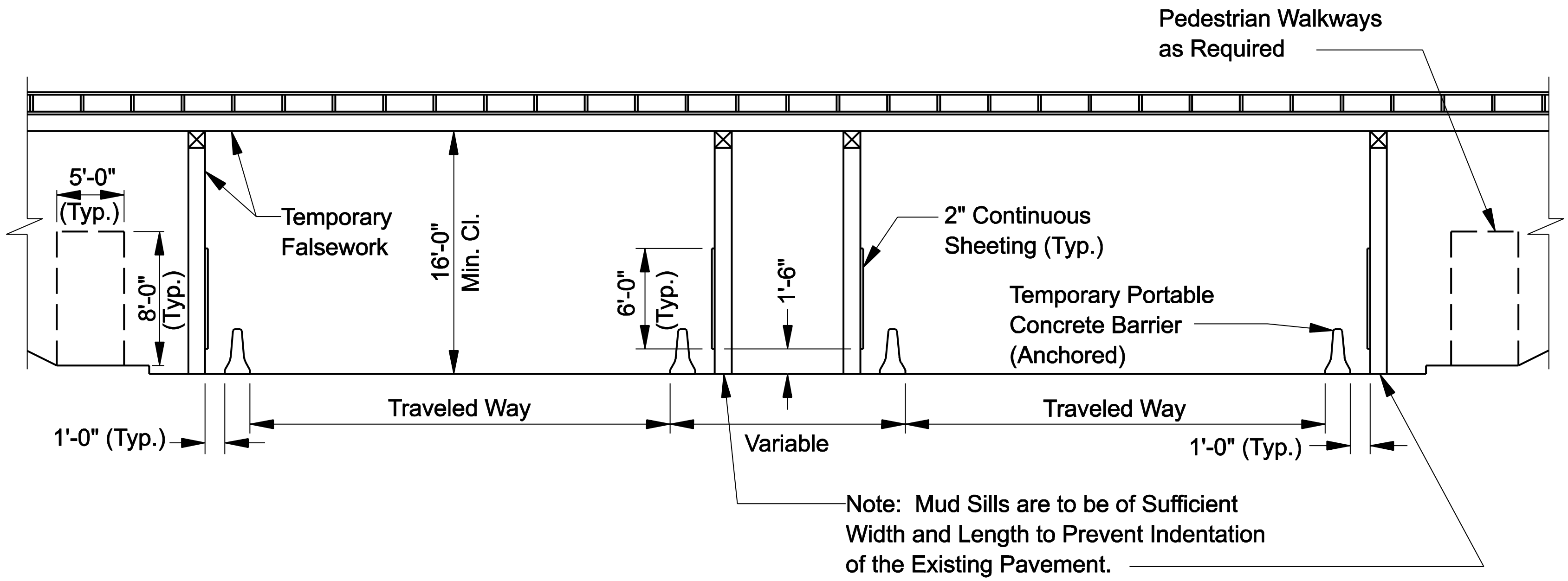
1. Deck Joints. Open, or inadequately sealed, deck joints have been identified as the foremost reason for corrosion of structural elements by permitting the percolation of salt-laden water through the deck. To address this, the *LRFD Specifications* promotes jointless bridges with integral abutments, continuous decks and improvements in drainage. If an in-span hinge must be used, consider using a second joint seal below the one at deck level consisting of a neoprene sheet trough.

2. Paint. The potential environmental issues associated with removing paint from steel structures makes the use of weathering steel preferable to painted steel from a maintenance perspective. However, in general, NDOT discourages the use of unpainted weathering steel because of aesthetic considerations. If weathering steel is used, the bridge designer must address the staining problem that can result from the use of weathering steel.

3. Drainage. Avoid elaborate plumbing systems where a closed system is used for bridge deck drainage. See Section 16.4.

4. Bridge Inspection. In addition to the maintenance needs of the structure, the bridge designer should consider the bridge inspection logistics including access.

5. Structural Details. As another maintenance/inspection consideration, the bridge designer should, as practical, limit the number of structural details (e.g., bearings, expansion joints).



TYPICAL FALSEWORK OPENING

Figure 11.4-A

11.4.12 Future Widening

In general, the bridge designer should consider the possibility of future structure widening. For example, structures supported by single columns or cantilevered piers cannot practically be widened; a separate adjacent structure will be required.

Almost every superstructure type can be widened, but not with the same level of ease. Slabs, slab on girders, and systems consisting of prefabricated elements lend themselves best to widening.

11.5 SUPERSTRUCTURES

This Section discusses those factors that should be considered in the selection of the superstructure type in preliminary design.

11.5.1 General Considerations

11.5.1.1 Introduction

Throughout the nation, many types of superstructures have been developed for the myriad applications and constraints that prevail at bridge sites. However, NDOT, like most other State DOTs, has narrowed its typical selection of superstructure types to a relatively small number based on NDOT's experience, geography, terrain, environmental factors, local costs, local fabricators, the experience of the contracting industry, availability of materials and NDOT preference. This promotes uniformity throughout the State and simplifies the bridge type selection and design process.

11.5.1.2 Span Length Ranges

Figure 11.5-A presents the typical span length ranges for the common and special application NDOT superstructure types. The upper limit of the span ranges relates to the structural capacity of the superstructure type. The lower limit of the range suggests a boundary below which other superstructure types are usually more cost effective; the lower limit does not suggest that the superstructure type cannot be used for shorter spans.

11.5.1.3 Typical Girder Spacings

Figure 11.5-B presents the typical girder spacings for the common and special application NDOT superstructure types. Generally, wider girder spacing results in a lower cost superstructure. However, wider girder spacings may require thicker decks, deeper girders, larger cross frames and higher concrete strengths for concrete girders, and they reduce redundancy.

11.5.1.4 Typical and Minimum Depths

Reference: LRFD Articles 2.5.2.6.2 and 2.5.2.6.3

Figure 11.5-C presents the typical and minimum depths for the common and special application NDOT superstructure types.

11.5.1.5 Substructure/Foundation Type Considerations

The selection of the foundation type typically occurs after the superstructure type selection. See Section 11.7. Therefore, the designer must anticipate the nature of the foundation characteristics in selecting the type of superstructure. The bridge designer should consider the following:

Structure Type		Span Length Ranges	
		Simple	Continuous
Common Superstructure Types			
Cast-in-Place, Post-Tensioned Concrete Box Girder		100′ - 225′	100′ - 250′
Composite Steel	Plate I-Girders	90′ - 250′	100′ - 400′
	Rolled Beams	30′ - 90′	
Composite Steel Tub Girders		120′ - 250′	120′ - 400′
Precast, Prestressed Concrete I-Girders		30′ - 80′	Up to 180′ spliced
Special Application Superstructure Types			
Cast-in-Place, Conventionally Reinforced Concrete Slab		Up to 40′; up to 60′ for voided slabs	
Cast-in-Place, Post-Tensioned Concrete Slab		40′-65′	
Precast, Post-Tensioned Concrete U-Girders		Spliced Lengths: 100′-230′	Spliced Lengths: 100′-250′

SPAN LENGTH RANGES

Figure 11.5-A

Structure Type		Typical Girder Spacing
Common Superstructure Types		
Cast-in-Place, Post-Tensioned Concrete Box Girder		Two times girder depth; 7′-10′ preferred
Composite Steel	Plate I-Girders	8′-14′
	Rolled Beams	6′- 10′
Composite Steel Tub Girders		Web-to-web spacing: 8′-12′
Precast, Prestressed Concrete I-Girders		6′-10′
Special Application Superstructure Types		
Cast-in-Place, Conventionally Reinforced Concrete Slab		N/A
Cast-in-Place, Post-Tensioned Concrete Slab		N/A
Precast, Post-Tensioned Concrete U-Girders		Two times girder depth; 8′-10′ preferred

TYPICAL GIRDER SPACING

Figure 11.5-B

Structure Type		Typical Depths (Relative to Span Length, L)	Minimum Superstructure Depth
Common Superstructure Types			
Cast-in-Place, Post-Tensioned Concrete Box Girder		Simple: 0.045L Continuous: 0.040L	3′-6″
Composite Steel	Plate I-Girders	Simple: 0.045L Continuous: 0.040L	3′
	Rolled Beams		2′
Composite Steel Tub Girders		Simple: 0.045L Continuous: 0.040L	4′
Precast, Prestressed Concrete I-Girders		Simple: 0.050L Continuous: 0.045L	3′-4′
Special Application Superstructure Types			
Cast-in-Place, Conventionally Reinforced Concrete Slab		Simple: 0.045L Continuous: 0.040L	1′-6″
Cast-in-Place, Post-Tensioned Concrete Slab		Simple: 0.030L Continuous: 0.027L	2′
Precast, Post-Tensioned Concrete U-Girders		Simple: 0.045L Continuous: 0.045L	4′

Note: For variable depth members, values may be adjusted to account for the change in the relative stiffness of positive and negative moment sections.

TYPICAL AND MINIMUM DEPTHS

Figure 11.5-C

1. Number of Supports. The expected foundation conditions will partially determine the number of and spacing of the necessary substructure supports. This will have a significant impact on the acceptable span lengths.

2. Dead Load. When foundation conditions are generally poor, the bridge designer should consider the economics of using structural steel over concrete to reduce dead load.

3. Scour. The geologic or historic scour may have a significant impact on the foundation design which may, in turn, have a significant impact on the superstructure type selection.

11.5.2 Cast-in-Place, Post-Tensioned Concrete Box Girder

11.5.2.1 Description

The cast-in-place, post-tensioned concrete box girder has been the most common structure type used in Nevada over the last 25 years. Practical limitations on span length are approximately 225 ft for simple spans and 250 ft for continuous spans. These bridges are torsionally stiff and can be used where there are high skews and tight-radius curves. A variable depth and width can also be easily accommodated. The depth variation can be linear or parabolic and extend either over a partial length or the entire span length.

In general, prestressed bridges do not have the long-term camber creep problems of conventionally reinforced concrete structures. This is due to the prestressing counteracting the dead load. In addition, the girder sections do not crack at service loads. The full cross section is effective in resisting loads and deflections resulting in a reduced structure depth.

The CIP, PT concrete box girder has the lowest construction and maintenance cost of all bridge types commonly used by NDOT. However, it involves the use of falsework and requires the longest time to construct. NDOT policy is to require a minimum of 16 ft of vertical clearance to falsework over traffic. If the temporary vertical clearance is less than 18 ft, protective systems are required. These temporary clearance requirements for falsework result in permanent vertical clearances that are considerably higher than required by NDOT minimum vertical clearances; see Section 11.9.6.2. These higher permanent clearances increase the approach roadway costs that must be included in the alternatives analysis. There is also a risk to the public and construction personnel when falsework spans over traffic. Use of falsework over rivers is risky and should be avoided.

All structures outside of Clark County must be protected from road salts. Epoxy-coated reinforcing steel and low water-cement ratio concrete have historically been the minimum requirements for deck protection. The use of high-performance concrete (low permeability and other factors) will help increase the life of deck slabs. However, decks of post-tensioned concrete box girders cannot be replaced without an adverse redistribution of the post-tensioning force. As such, additional deck protection should be considered for this structure type. The thickness of the top slab should be a minimum of 8 in.

Simple-span bridges typically require higher concrete strengths. Values up to 6500 psi have been used. The inventory and operating ratings of long simple-span bridges are very sensitive to small changes in the top slab concrete strength. A concrete strength of only 90% of the specified 28-day strength may have an adverse impact on the live-load capacity of a post-tensioned concrete box girder. Therefore, consider increasing the required 28-day strength by 5% to 10% over that required by design to account for possible production strength variations.

11.5.2.2 Typical Usage

The cast-in-place, post-tensioned box girder is normally used for simple spans over 100 ft and continuous spans over 130 ft. It may be considered for span lengths less than 100 ft if a shallower structure depth is needed.

11.5.2.3 Advantages/Disadvantages

The advantages of this structure type include low construction cost, low maintenance costs, good aesthetics, low depth-to-span ratio, and easy adaptability to complex geometry. High torsional resistance makes it desirable on horizontal curved alignment.

The disadvantages include longer construction time, need for falsework and complicated formwork, and the inability to replace the top slab.

11.5.2.4 Appearance

The appearance is good from all directions. The system conceals utilities, pipes and conduits.

11.5.2.5 Typical Girder Spacing

Girder spacings are normally from 7 ft to 10 ft for this structure type. Wider girder spacings can be used; however, the amount of post-tensioning for each web becomes crowded and inefficient. Reducing the number of webs in this structure type does not demonstrably reduce costs. The amount of post-tensioning can generally be reduced if more webs are added. Fewer ducts in a web results in an increased eccentricity between the post-tensioning center of gravity and the neutral axis, which increases efficiency.

11.5.3 <u>Composite Steel I-Girders</u>

11.5.3.1 Description

Composite steel I-girders are fairly common in Nevada. The steel girders can be either rolled shapes with spans up to approximately 90 ft or plate girders with spans up to approximately 250 ft. Plate girders can have a constant or variable depth. Abrupt depth changes should be avoided for aesthetic reasons. Changes in structure depth can be accomplished over the length of a span by linearly varying the web depth. Continuous girders can also be deepened at the supports and reduced at the center of the span where vertical clearance is tight. These haunched girders usually have a parabolic variation in depth.

Most structural steel is fabricated out of State, and shipping increases the cost of this structure type. Girder field sections can be easily transported in lengths up to approximately 125 ft. Splices are used to construct single girder lines up to approximately 1000 ft in length. The designer must consider how this structure type will be erected, where the erection crane(s) will be located and how the girders will be delivered to the site.

This is a girder-and-deck type of structure; therefore, the deck can be removed if needed without adversely affecting the steel I-girders. This structure type can also be used with large skews and on horizontal curves. Detailing of steel I-girders is critically important. Poor detailing will greatly increase the cost of the bridge. In addition, structural steel is susceptible to fatigue cracking and brittle fracture, but good detailing practices greatly reduce this potential.

NDOT's standard paint system for steel bridges is an inorganic zinc (zinc in a silicate media) base coat, an epoxy middle coat and urethane top coat. The inorganic zinc is applied to all steel surfaces except the top of top flange. Inorganic zinc is considered a Class B coating for slip resistance and can be included on the faying surfaces of all bolted connections.

See Chapter 15 for a detailed discussion on NDOT's design practices for structural steel superstructures.

11.5.3.2 Typical Usage

NDOT typically limits the use of structural steel superstructures to sites where a cast-in-place, post-tensioned concrete box girder superstructure is not the best choice. Structural steel is used where falsework necessary for cast-in-place, post-tensioned concrete boxes is not allowed, where construction duration is an issue, or where the falsework may cause problems (e.g., low clearances, safety).

Rolled beams are used for spans up to approximately 90 ft. Welded plate girders are used for spans from approximately 90 ft to 400 ft. Typically, simple spans may be used up to approximately a 250-ft span length; continuous spans may be used for spans from 120 ft to 400 ft. If a rolled-beam design is proposed for a new bridge, the contract documents should allow the substitution of a welded plate-girder with equivalent plate dimensions at the contractor's discretion.

Transportation of girders must be considered when identifying field splice locations.

11.5.3.3 Advantages/Disadvantages

When compared to other superstructure types, advantages of composite steel I-girders include fast on-site construction, no falsework, relatively simple details and formwork, good aesthetics, adaptable to complex geometrics, low dead weight, deck can be replaced and long-span capability. The structural characteristics for composite steel I-girders provide low dead load and, therefore, may be suitable when foundation conditions are poor.

The disadvantages of composite steel I-girders include moderate to high construction costs, high maintenance costs and attention to detailing practices. Detailing of steel girders is important. Poor detailing will greatly increase the cost of the bridge and can decrease durability through fatigue cracking. Composite steel I-girder bridges have a higher maintenance cost than concrete bridges.

11.5.3.4 Typical Girder Spacing

Girder spacings for steel I-girders are normally from 8 ft to 14 ft. Deep plate girder sections benefit the most from wide girder spacings. Shallow plate girders and rolled beams do not accommodate wider girder spacings and may require spacings less than 8 ft when at the limit of the depth-to-span ratios. The design of the shallow girder sections can be controlled by deflection requirements.

11.5.4 <u>Composite Steel Tub Girders</u>

11.5.4.1 Description

Composite steel tub girders are typically considered in urban areas where a steel-I girder could be used but enhanced aesthetics are desired. Steel tub girders are plate girders with two webs with a common bottom flange. The webs are usually inclined to improve aesthetics and reduce the width of the bottom flange. Spans are economical up to approximately 250 ft. Steel tub girders can have a variable depth, but this significantly increases the cost of the bridge. They can also be used on very tight-radius curves due to their high torsional stiffness. They do not, however, adapt well to skews or variable widths. Fabrication, transportation and erection of this structure type must be carefully considered. Steel tub girders are difficult to handle in the shop due to their size and weight. They require significant bracing during fabrication and erection. In addition, they are susceptible to thermal movements once erected and require temporary external bracing between boxes.

The paint system for steel tub girders is the same as for steel I-girders.

11.5.4.2 Typical Usage

Composite steel tub girders can be used for simple spans up to 250 ft and for continuous spans from 120 ft to 400 ft. This structure type is used mainly in urban areas.

11.5.4.3 Advantages/Disadvantages

Advantages of composite steel tub girder bridges include fast on-site construction, no falsework requirements, low dead weight, adaptability to tight-radius curves, deck can be replaced, good aesthetics and longer span capability. Higher torsional resistance makes them desirable on a horizontally curved alignment.

Few fabricators are available to construct composite steel tub girders. Disadvantages include the highest construction cost of all common NDOT superstructure types, high maintenance costs, and not readily adaptable to skewed or variable-width bridges. Composite steel tub girders require complicated fabrication, welding and erection. This structure type has higher maintenance cost than concrete bridges.

11.5.4.4 Appearance

This structure type is generally pleasing; it is more attractive than steel or precast concrete I-girders.

11.5.4.5 Typical Girder Spacing

Tub spacing is normally 8 ft to 12 ft. Deep sections benefit the most from wider spacings. Shallow sections do not accommodate wider girder spacings.

11.5.5 Precast, Prestressed Concrete I-Girders

11.5.5.1 Description

Precast, prestressed concrete I-girders have historically been used infrequently by NDOT. Relatively small bridges, common in Nevada, and precast plants located outside of Nevada have contributed to a higher cost for this structure type. Practical limitations on span length are approximately 100 ft for simple spans and 125 ft for continuous spans. Some States have spliced the girders to extend the spans even longer. This is a girder-and-deck type of structure; the deck can be removed if needed without adversely affecting the girders. This structure type is best used on multiple spans with a large number of girders. It does not adapt well to large skews and cannot be used on tight horizontal curves or bridges with a variable width.

See Chapter 14 for a detailed discussion on NDOT's design practices for prestressed, precast concrete girders.

11.5.5.2 Typical Usage

Precast concrete girders are generally only used where falsework is not allowed (e.g., over railroads), where construction duration is an issue, or where falsework may cause considerable problems (e.g., low clearances, safety). Precast, prestressed concrete I-girders are used for spans from 30 ft to 150 ft. The transportation and erection of precast I-girders must be considered in selecting girder lengths.

11.5.5.3 Advantages/Disadvantages

Advantages of this structure type include moderate construction cost on small bridges to fairly low construction cost on large bridges, low maintenance cost, no falsework, deck can be replaced, and moderately fast on-site construction. Its disadvantages include poor aesthetics due to thick bottom flanges with relatively narrow girder spacings, cannot be adapted to complex geometrics, limited span lengths and slightly higher depth-to-span ratios. Precast, prestressed concrete I-girders require careful handling during transportation and erection.

11.5.5.4 Appearance

Straight girders on curved alignment are discouraged, effectively precluding the use of precast I-girders except for relatively large radius curves.

11.5.5.5 Typical Girder Spacing

Girder spacings are normally from 7 ft to 10 ft. Concrete strength and the number of prestressing strands usually control the girder spacing. Concrete strengths of 6000 psi to 7000 psi can be produced in most precast plants. These higher strengths allow longer spans and/or increased girder spacing.

11.5.6 Special Application Superstructure Types

For special applications, it may be appropriate to select a superstructure type other than the common NDOT types. This Section briefly identifies those types (and their potential application) that may be used in special applications.

11.5.6.1 Cast-in-Place, Conventionally Reinforced Concrete Slab

11.5.6.1.1 *Description*

NDOT occasionally uses cast-in-place, conventionally reinforced concrete slabs (CIP concrete slabs) because of their suitability to short spans and low clearances and their adaptability to skewed and curved alignments. It is the simplest among all superstructure systems, it is easy to construct, and structural continuity can be achieved without difficulty.

NDOT practice is to use a constant-depth slab or a slab with haunches in the negative-moment regions.

11.5.6.1.2 *Typical Usage*

The CIP concrete slab is used for bridge spans up to approximately 40 ft; voided slabs are used up to approximately 60 ft.

11.5.6.1.3 *Advantages/Disadvantages*

The advantages of this structure type are low construction costs and low maintenance costs. The details and formwork are the simplest of any superstructure type. Construction time is also fairly short. The disadvantages are that the CIP concrete slab requires falsework and has a high depth/span ratio.

11.5.6.1.4 *Appearance*

Neat and simple, especially for low, short spans.

11.5.6.2 Cast-in-Place, Post-Tensioned Concrete Slab

11.5.6.2.1 *Description*

The basic distinction between the CIP concrete slab and the cast-in-place, post-tensioned concrete slab is the difference in the manner of reinforcement; therefore, most of the information in Section 11.5.6.1 is applicable. This structural system can be used for greater span lengths than the CIP concrete slab at comparable structural depth.

11.5.6.2.2 *Typical Usage*

Consider using this structure type for spans up to 65 ft. It may be the best selection where a very low depth/span ratio is required.

11.5.6.2.3 *Advantages/Disadvantages*

The advantages are low construction costs, low maintenance costs and thin superstructure depth. The disadvantages are long construction time and the requirement for falsework. The CIP, PT concrete slab is more difficult to construct than conventionally reinforced concrete slabs.

11.5.6.2.4 *Appearance*

This is the same as conventionally reinforced concrete slabs.

11.5.6.3 Precast, Post-Tensioned Concrete U-Girders

11.5.6.3.1 *Description*

The precast, post-tensioned U-girder bridge has recently gained in popularity. It provides the look and behavior of cast-in-place, post-tensioned concrete box girder construction but uses precast elements. It is higher in cost than both cast-in-place, post-tensioned concrete and precast, prestressed concrete I-girders. The spans are comprised of segments that are erected on temporary towers, connected together by cast-in-place concrete, and then post-tensioned full length.

Many concrete U-girders have a combination of pretensioning and post-tensioning. The pretensioning carries the girder self weight and construction loads, and the post-tensioning is designed to carry loads in the permanent configuration.

11.5.6.3.2 *Typical Usage*

This structure type can also be used to widen an existing cast-in-place concrete box girder to provide compatible aesthetics and structural performance. In addition, it can be used where clearances restrict the use of falsework.

11.5.6.3.3 *Advantages/Disadvantages*

Advantages include no falsework over traffic, low maintenance costs, good aesthetics and low depth-to-span ratios. Disadvantages include non-standard U-girder sections, moderate to long construction time, higher construction cost, complex construction, cannot be adapted to complex geometrics and decks are difficult to replace.

11.5.6.3.4 *Appearance*

The appearance is good from all directions. Concrete U-girders provide the look of cast-in-place, post-tensioned concrete box girder construction.

11.5.6.3.5 *Typical Girder Spacing*

The typical girder spacing for both cast-in-place, post-tensioned box girders and precast I-girders should be used for U-girders.

11.5.6.4 Long-Span Culverts

Reference: LRFD Section 12

Long-span culverts may be an attractive alternative for small stream and ditch crossings (where they can protect the stream bed), minor highway and street crossings, and pedestrian or wildlife crossings. As discussed in Section 11.2.2.7, hydraulics is one of the significant issues in selecting a culvert or a bridge. Long-span culverts are commonly made of steel or concrete. The most common configurations used are the three-sided concrete or steel culvert, four-sided monolithic precast concrete box culvert, structural plate pipe arch and circular pipe. Spans of 50 ft or less are reasonable; pipe arch spans up to 80 ft are possible.

If a single or multiple specialty installation is being considered, the designer should consult with the manufacturer(s) of the specialty structure for design information (e.g., cost, availability, design).

11.5.7 Superstructure Types Used With Approval

Superstructure types other than the "common" and "special application" types may be used. The bridge designer should investigate the experience of other owners, and the acceptability of these superstructure types may be based upon their successful experiences. The Chief Structures Engineer must provide written approval for the selection of structure types not considered "common" or "special application."

11.5.7.1 Cast-in-Place, Conventionally Reinforced Concrete Box Girders

The basic distinction between cast-in-place (CIP), conventionally reinforced concrete (RC) box girders and cast-in-place, post-tensioned concrete box girders is the difference in the manner of reinforcement; therefore, some of the information in Section 11.5.2 is applicable. However, the disadvantages of CIP RC boxes result in very limited application, if at all in Nevada. This structural system can only be used for shorter span lengths than the CIP, PT concrete boxes at comparable structural depth. CIP RC boxes have low construction and maintenance costs with good aesthetic qualities. However, this superstructure type requires falsework, requires a long construction time and demonstrates long-term creep deflection problems not associated with comparable prestressed superstructure types. CIP RC box girders are limited to shorter span lengths due to inherently high depth-to-span ratios.

11.5.7.2 Timber Bridges

Reference: LRFD Section 8

Timber structures shall be avoided, unless the bridge site is a remote, off State highway system location where a conventional bridge is impractical. The maximum span length is 40 ft, and the typical girder spacing is 4 ft to 6 ft. Depth/span ratios are as follows:

- simple span (timber girder): 1/10
- simple span (glulam girder): 1/12
- continuous span: 1/14

Timber bridge design details and construction are simple, and they can be aesthetically pleasing in the proper setting. Timber bridges are lightweight and require no falsework. Disadvantages include high maintenance costs, limited useful life and barrier rail connection problems.

Glued-laminated timber shall be used (rather than sawn timber) for main load-carrying elements (e.g., girders, caps). Glued-laminated timber deck panels are also available.

11.5.7.3 Segmental Concrete Box Girders

Reference: LRFD Article 5.14.2

Segmental construction is only considered on large projects where the total deck area is approximately 250,000 sq ft or more. However, it can be used in other special applications that dictate the use of this structure type. A significant investment in the casting facilities and erection equipment is required for precast segmental construction. This structure type usually consists of a single-cell box girder built in segments by either the precast or cast-in-place method.

Advantages include no falsework, least effect on traffic of any structure type, low maintenance costs, good aesthetics, fast on-site construction for precast method and low depth-to-span ratios. Disadvantages include the complexity of time-dependent analysis and design, variable construction costs, limited number of qualified contractors, complex construction and construction engineering, lengthy construction time for the cast-in-place method, economical only on large projects, and decks cannot be replaced.

Most segmental concrete bridges have a single-cell superstructure with only two girder webs. The spacing of the web is based on the efficiency of the deck design. They are either cast-in-place or precast construction with longitudinal post-tensioning and transverse post-tensioning in the deck. The longitudinal post-tensioning can be placed internal to the webs as with conventional cast-in-place, post-tensioned construction; fully external to the web; or a combination of internal and external. Internal post-tensioning has the same considerations as cast-in-place, post-tensioned concrete box girders. Externally post-tensioned bridges usually have thinner webs because a wider web is not needed for the post-tensioning. Research at the University of San Diego indicates a combination of internal and external tendons has the lowest seismic resistance of the three and should not be used in moderate-to-high seismic areas outside Clark County.

Precast box segments differ from project to project, but the American Segmental Bridge Institute has established standard sections. Spans are used typically up to approximately 250 ft.

Concrete segmental construction requires that the bridge be designed for a specified method of construction and included in the contract documents with assumed erection loads. The design of the substructure is, in many cases, controlled by the construction of the bridge and not the permanent loads.

The precast method is usually less expensive than the cast-in-place method and is used mainly for shorter spans. Precast segments are match cast in a casting yard, transported to the site, drawn together in the erected position by temporary post-tensioning, the span closed by cast-in-place concrete, and subsequently post-tensioned the full length of span. Erection methods can be by balanced cantilever, temporarily supported by under-slung trusses, or by an overhead gantry.

A traveling form supports cast-in-place segmental construction and is advanced when the poured concrete has reached sufficient strength. Erection methods include balanced cantilever, a gantry crane or a system of stays.

These projects are complex. An experienced contractor and contractor's engineer are necessary. Erection methods and equipment used to erect the segments vary from project to project. The contractor is required to verify the design based on their means and methods of construction.

11.5.7.4 Other Superstructure Types

Superstructures types other than those specified elsewhere in Section 11.5 may be used. These include:

- steel trusses,
- steel tied arches,
- concrete arches, and
- cable-supported concrete or steel bridges.

11.6 SUBSTRUCTURES

11.6.1 Objective

This Section discusses the types of substructure systems used by NDOT and their general characteristics. The bridge designer should use this guidance to select the substructure type that is suitable at the site to economically satisfy the geometric and structural requirements of the bridge and to safely use the strength of the soil or rock to accommodate the anticipated loads. Chapter 18 discusses the detailed design of substructure elements, including detailed figures for each abutment type.

11.6.2 Abutments

Reference: LRFD Article 11.6

11.6.2.1 General

Abutments can be classified as flexible or rigid. Flexible (also known as integral or dozer) abutments transmit earth pressures on the abutments through the superstructure eliminating expansion joints at the end of the superstructure (for total movements of less than ½ in). Rigid (also known as seat) abutments incorporate expansion joints at the end of the bridge to accommodate thermal movements. Flexible abutments must be able to accommodate the movements through elastic behavior of the bridge and the surrounding soil because the deck and girders are integral with the abutment. Flexible abutments are considered pin-ended, expansion bearings in the superstructure analysis. Rigid abutments can be fixed or expansion based upon the choice of bearings.

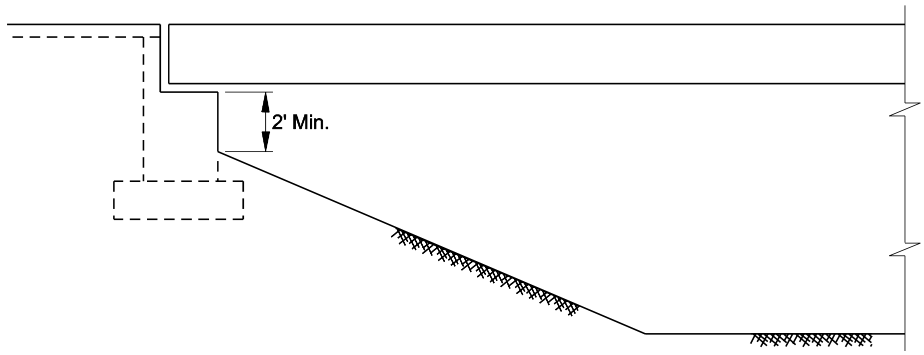
Abutments may be further classified as either open or closed. See Figure 11.6-A for schematics. Open abutments are used for most bridges and are placed at the top of the slope. Slopes are typically 2H:1V. See Figure 11.6-B. This slope is based on stability requirements and erosion control. However, landscaping and aesthetic considerations may result in 3H:1V slopes. Contact the Landscaping/Aesthetics Section for slope recommendations when constructing bridges along corridors that have landscape requirements. In addition, the 2 ft of exposed abutment face may be increased for landscape aesthetics.

Open abutments result in longer spans compared to closed abutments, but the total overall cost is usually less compared to closed abutments because open abutments are typically shorter. In general, open abutments are considered more aesthetic than closed abutments.

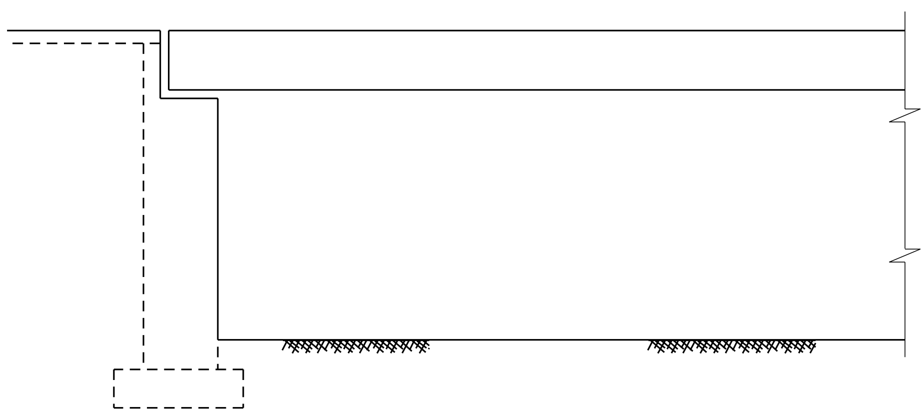
Closed abutments are used when span lengths need to be minimized. There are no fill slopes under the bridge but extensive retaining walls must be used. These retaining walls run either along the approaches to the bridge or parallel to the abutment. Retaining walls along the approaches are preferred from a visual perspective. Closed abutment footings must be placed below the level of the highway running beneath the bridge resulting in tall exposed abutment faces.

11.6.2.2 Basic Types

An abutment may be designed as one of the following basic types:



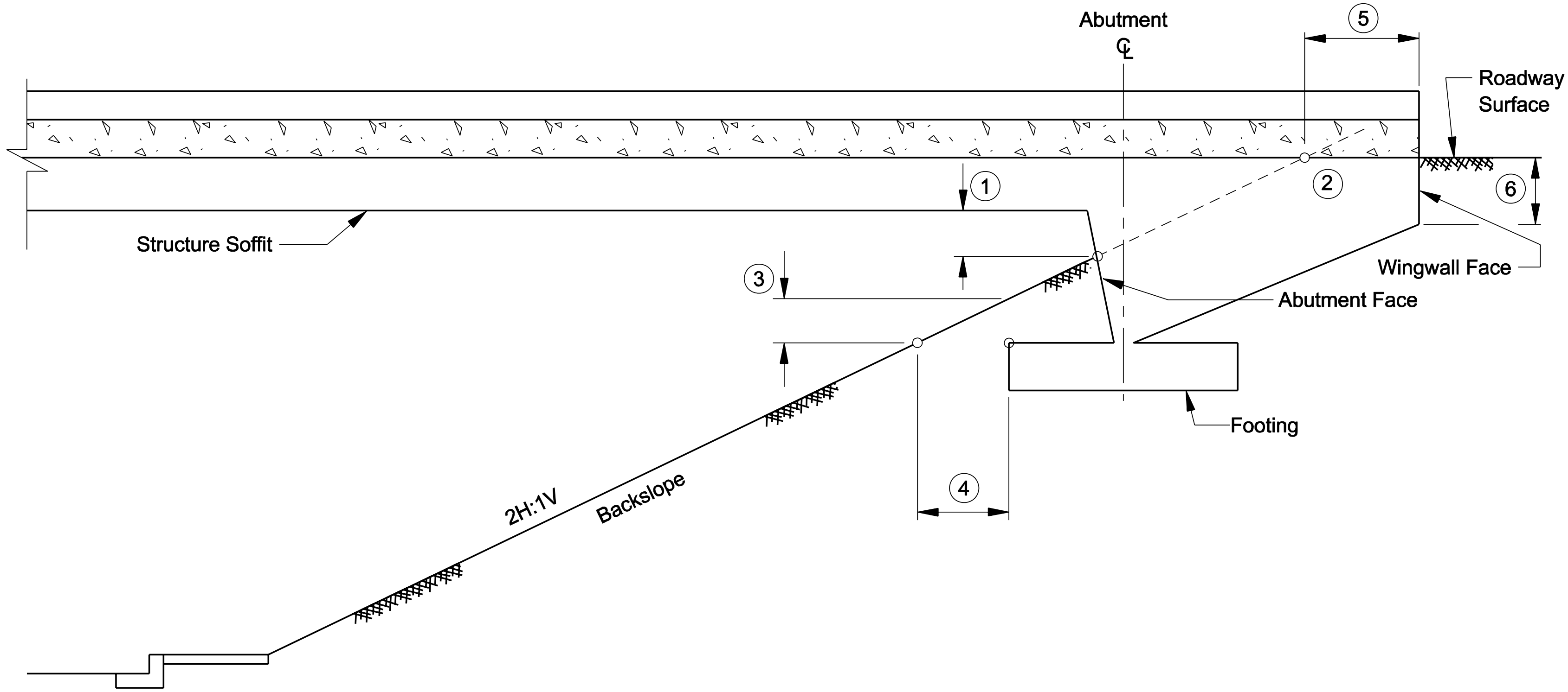
OPEN ABUTMENT



CLOSED ABUTMENT

OPEN vs CLOSED ABUTMENTS

Figure 11.6-A



1. The intersection of the backslope and abutment face should occur approximately 2 ft below the structure soffit.
2. Intersection of backslope and roadway surface.
3. Earth cover on footing. Minimum is 1.5 ft.
4. Distance from top edge of footing to face of slope. Minimum is 4.0 ft.
5. Distance from backslope/roadway surface intersection to end of wingwall. 5.0 ft preferable.
6. Distance from roadway surface to bottom of end of wing. 3.0 ft preferable.

OPEN ABUTMENT SLOPE

Figure 11.6-B

- integral abutments,
- seat abutments, and
- dozer abutments.

Each of these is discussed in the following Sections.

Flexible abutments, either integral or dozer abutments, are generally preferred for bridges of length up to 400 ft where the soil pressure on the two abutments is opposing and approximately balanced. Seat abutments are used where these conditions and other geometric limitations are not met.

11.6.2.3 Integral Abutments

Integral abutments are segregated into the following sub-types:

1. <u>Diaphragm-with-Footing Abutment</u>. This abutment is formed by extending an end diaphragm down to a footing. The end diaphragm is rigidly attached to the superstructure and free to slide on the footing. Stops (shear keys) are placed on the footing to limit the movement of the diaphragm during seismic events. The extension of the end diaphragm down to the footing forms the abutment stemwall. The footing may be either a spread footing or pile supported. If piles are used, two rows of driven piles or small-diameter drilled shafts spaced a minimum of 6 in to each side of the centerline of bearing, or a large-diameter (36 in minimum) drilled shaft is required.

   The diaphragm-with-footing abutment is the most widely used and is generally preferred. It can be used with the restriction that the diaphragm height measured from the top of footing to the bottom of the superstructure does not exceed 9 ft or twice the superstructure depth. A footing pedestal of up to 3 ft in height may be used to comply with this restriction.

2. <u>Diaphragm-with-Pile Abutment</u>. This abutment type is restricted to shorter bridge lengths. The end diaphragm is extended down to the foundation as for diaphragm-with-footing abutments. However, the diaphragm is supported on a single row of piles and is connected to them. The connection to the piles may be rigid where the pile is embedded into the diaphragm or pinned where the diaphragm is connected to a pile cap by a pin connection. In either case, the diaphragm is rigidly connected to the superstructure.

   Diaphragm-with-pile abutments may be used where piles are required and the bridge can be supported on a single row of piles. The piles must be installed accurately without drift and must be capable of accommodating expected thermal movements. Stiff, large-diameter drilled shafts are not suitable for use with this type of abutment. Drilled shafts of any size should not be used on post-tensioned structures due to the movement resulting from prestress shortening.

11.6.2.4 Seat Abutments

Seat abutments consist of a footing, stemwall, seat and backwall. The superstructure is supported by bearings on the abutment seat. The backwall retains the backfill above the abutment seat so that the backfill is not in contact with the superstructure. The approach slab extends over the top of the backwall, and there is an expansion joint between the approach slab and the superstructure deck.

11.6.2.5 Dozer Abutments

A dozer abutment is a hybrid of the conventional seat abutment and an integral abutment. The superstructure is supported by bearings on top of the abutment stem wall. The superstructure end diaphragm extends beyond and below the stem wall. The height of the diaphragm extension may be adjusted as required by design.

Dozer abutments are used in lieu of the diaphragm-with-footing abutment where the stem wall height exceeds the height limit for a diaphragm or where skews exceed 30°. However, dozer abutments are complex to design and detail for high skews.

11.6.2.6 Applicability and Limitations

Integral and dozer abutments are advantageous because they mobilize earth passive pressure to resist and dampen seismic forces and eliminate damage to abutment backwalls from seismic movements. Additionally, both integral and dozer abutments provide the capability to move the expansion joint at the end of the bridge to a point where joint leakage does not promote deterioration of bearings or abutment seats.

Integral and dozer abutments may be used on bridges with flares, skewed support or horizontal curvature only where the effect of these design features is limited. The determination of acceptable limits is based more on experience than theory. Flares and curvature are limited because the soil pressures must be reasonably balanced such that the imbalanced force can be readily accommodated. Structure skew also results in unbalanced soil pressures because the lines of action of the soil pressures on the two abutments does not coincide. Additionally, for integral abutments, the horizontal axis of rotation of a skewed abutment is not parallel to the bending axis of the superstructure girders, which produces torsion in the superstructure. Dozer abutments reduce this effect through the use of bearings similar to a seat abutment.

The same abutment type should be used at both ends of the bridge. Integral and dozer abutments shall not be used where there are expansion hinges in the superstructure. Figure 11.6-C presents the more detailed limits that should be used in selecting an abutment type.

The thermal movement of an abutment should be limited to approximately 1 in of expansion or contraction for integral and dozer abutments. For structures with integral abutments, the wingwalls are cantilevered from the diaphragm wall and are assumed to move with the structure. They are usually oriented parallel to the roadway to avoid resisting passive earth pressures due to the thermal movement. Wingwalls on structures with dozer or seat abutments are supported by a footing or piles. They are usually oriented parallel to the roadway, but may be set at angles if desired, such as at stream crossings. Approach slabs are required on paved roads and preferred on unpaved roads. The approach slab is stationary with a watertight joint to accommodate movement at the end of the structure where seat abutments are used. Where the wingwalls are oriented parallel to the roadway, the approach slab should extend over the wingwalls and support the barrier rail.

11.6.3 <u>Piers</u>

Reference: LRFD Article 11.7

Geometric Parameters	Abutment Type			
	Integral Abutment		Dozer Abutment	Seat Abutment
	Diaphragm with Footing	Diaphragm with Piles		
Maximum Length (1) Concrete Steel	 400 ft 250 ft	 250 ft 150 ft	 400 ft 250 ft	 Unlimited Unlimited
Maximum Flare (2)	20%	10%	40%	Unlimited
Maximum Skew	30°	20°	30°	Unlimited
Maximum Curvature (3)	10°	10°	10°	Unlimited

(1) Values are for cold climate per LRFD Article 3.12.2.1. Values may be increased by 20% in areas of moderate climate.

(2) Adjust diaphragm height to approximately balance soil pressures. Limitation does not apply to simple span bridges with diaphragm abutments.

(3) Central angle of a horizontal curve within the bridge limits or the difference in survey bearings of abutment centerlines.

LIMITS FOR ABUTMENT TYPES

Figure 11.6-C

11.6.3.1 General

Piers consist of a pier cap supported on columns or a pier wall. Although rarely used by NDOT, under certain conditions, the economy of substructures can be enhanced by extending a deep foundation above ground level to the superstructure forming a pile bent.

11.6.3.2 Pier Caps

Pier caps are usually reinforced concrete members that transfer girder loads into columns or pier walls. In all cases, a pier cap shall be used. These can be integral, drop or outrigger caps.

Integral caps are mainly used with cast-in-place concrete girders but also can be used with precast and steel girders. Integral caps used with steel girders can be either steel cross girders or post-tensioned concrete. They should be used only when necessary. Integral steel caps are non-redundant, expensive and require precise fabrication. Integral concrete caps with steel girders are difficult to construct, usually require temporary falsework and do not allow inspection of the top tension flanges after the bridge goes into service.

Outrigger caps are used where a column support must extend beyond the edge of the superstructure. Outrigger caps should not be used unless necessary. They should be simple spans with pin connections at the top of the columns. Pin connections reduce the torsional shear forces in the outrigger cap. Most outrigger caps are integral concrete and post-tensioned to reduce their depth, control cracking and enhance torsional resistance.

11.6.3.3 Columns and Pier Walls

Columns and pier walls are substructure components that support the cap. Either single or multiple columns can be used depending upon the width and skew of the bridge. Columns are used in highway-over-highway construction. NDOT's standard columns have an octagonal or rounded shape with 4 ft outside dimensions. This allows the use of a continuous transverse spiral or welded hoops. Rectangular columns up to 8 ft in width by 4 ft in depth have been used with interlocking transverse spiral reinforcement.

A single-column pier should be considered for narrow bridges over rivers. River meander can change the direction of flow, which has an adverse effect on pier walls. Water hitting a pier at an angle will greatly increase scour. This does not adversely affect a single round column. However, single columns are usually at least 6 ft in diameter.

Pier caps supported on columns for highway-over-highway applications may consist of a minimum of two columns. Where redundancy considerations are an issue, the bridge designer should consider a minimum of three columns.

A pier wall is a continuous wall extending to almost the outside face of bridge. They are typically 2′-6″ wide with tied reinforcement.

Pier walls are also used for bridges over railroads to satisfy AREMA crash wall requirements. Columns with crash walls can be used, but a pier wall eliminates the need for a massive cap.

Pile bents shall not be used where large lateral forces may develop due to collision by vehicles, scour or stream flow intensified by accumulated debris Where used, the piles may be either steel H-piles or pipe piles.

11.6.3.4 General Usage

The following will apply to the selection of a pier type:

1. Water Crossings. If the foundation conditions allow, a drilled-shaft, single-column pier is preferred. Multiple columns are usually preferred to a pier wall. The Hydraulics Section will provide a recommendation to assist in this determination.

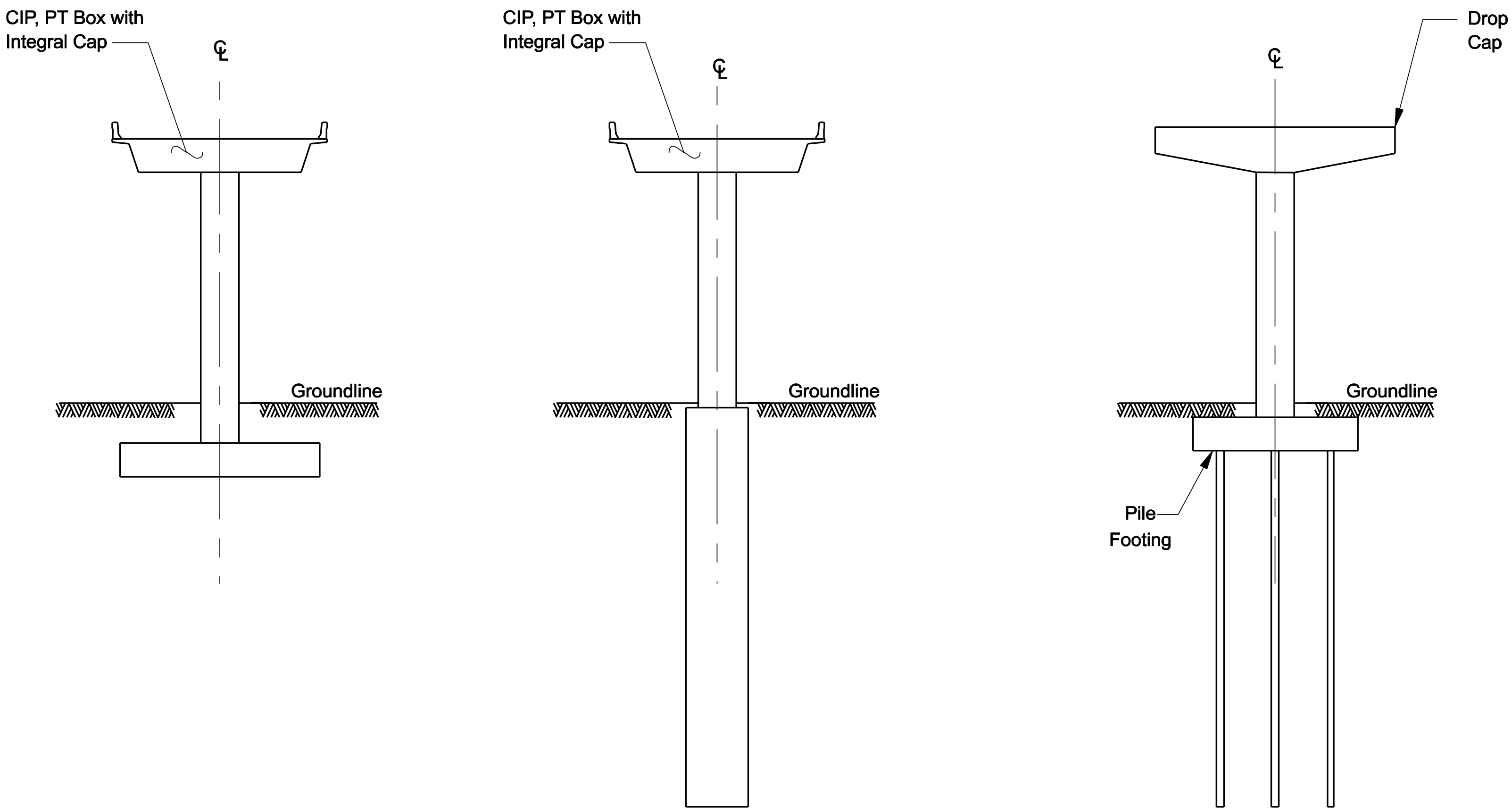
2. Meandering Rivers. For meandering rivers, the most desirable pier type is normally a single pier column. This type should be used, if practical.

3. Railroad Crossings. Use a solid pier wall that satisfies AREMA requirements if the pier is within 25 ft of the track centerline or future track centerline. See Section 21.1.3.4 for more information.

4. Highway Grade Separation. Preferably, use multiple column piers.

11.6.3.5 Schematics

Figure 11.6-D presents schematics of typical pier types used by NDOT in combination with typical NDOT foundations. Note that other pier/foundation combinations may be appropriate that are not shown in Figure 11.6-D.



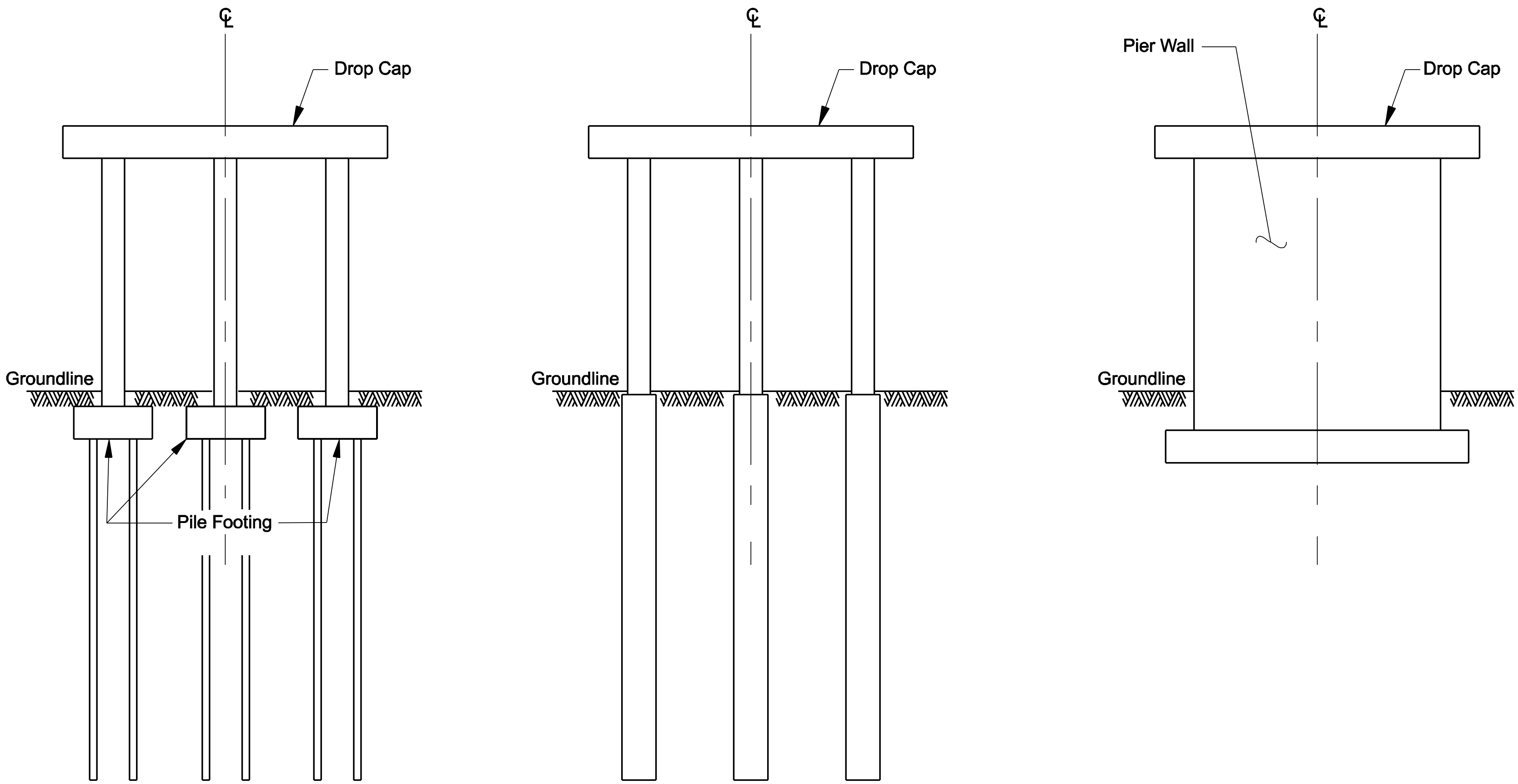
SINGLE-COLUMN PIER WITH SPREAD FOOTING
(a)

SINGLE-COLUMN PIER WITH DRILLED SHAFT
(b)

SINGLE-COLUMN PIER WITH DRIVEN PILES
(c)

TYPICAL PIER/FOUNDATION COMBINATIONS

Figure 11.6-D



MULTI-COLUMN PIER
WITH DRIVEN PILES
(d)

MULTI-COLUMN PIER
WITH DRILLED SHAFTS
(e)

PIER WALL
WITH SPREAD FOOTING
(f)

TYPICAL PIER/FOUNDATION COMBINATIONS
(Continued)

Figure 11.6-D

11.7 FOUNDATIONS

11.7.1 Coordination

11.7.1.1 General

Coordination between the bridge designer and Geotechnical Section for foundation-type selection and design is performed in two phases. During preliminary design, the bridge designer provides the Geotechnical Section with a structure layout (Front Sheet) and preliminary foundation vertical loads. The Geotechnical Section performs the drilling, sampling and testing and then provides the preliminary foundation recommendations. These recommendations will include either spread footings or deep foundations (with recommended pile type).

For waterway crossings, bridge foundations must be designed by an interdisciplinary team of hydraulic, geotechnical and bridge engineers to withstand the effects of estimated total scour, including:

- local scour at piers and abutments,
- contraction scour, and
- long-term aggradation or degradation.

During final design, vertical loads are refined, lateral loads determined and provided to the Geotechnical Section along with scour depths (if a stream crossing). The Geotechnical Section provides final foundation recommendations. This can also include special requirements such as p-y curves for lateral pile design, downdrag potential, preloading requirements and ground modification. The pile depths are based on vertical loads with scour considerations. The bridge designer evaluates the structural requirements of piles and will extend the depth due to lateral loading if needed.

Chapter 17 discusses the detailed design of foundations, including the coordination among the Structures Division, Geotechnical Section and Hydraulics Section.

11.7.1.2 Seismic Analysis

For drilled-shaft/driven-pile-supported bridges that require a rigorous seismic analysis, the Structures Division performs lateral soil-structure interaction analyses using Extreme Event I loadings. If soil liquefaction is anticipated, the Geotechnical Section will provide the Structures Division with foundation downdrag loads due to liquefaction for use in developing the Extreme Event I load combination. The Geotechnical Section will also provide any lateral soil forces that act on the foundation as a result of seismically induced stability movements of earth retaining structures (e.g., embankments, retaining walls) or lateral soil movements attributable to lateral spread. These additional lateral loads should be included in the Extreme Event I load combinations when performing lateral soil-structure interaction. The Geotechnical Section will provide the soil parameters necessary to generate a p-y soil model of the subsurface that accounts for cyclic loadings and any liquefied soil conditions. The Structures Division then performs the lateral soil-structure interaction analysis with computer programs such as STRAINWEDGE or LPILE. The Structures Division uses this information to calibrate the seismic model. The Structures Division performs the seismic analysis in accordance with the *LRFD Specifications.*

If structural members are overstressed or if deflections exceed acceptable limits from any loading combination, then a redesign of the foundation is required. Redesign may include the adjustment of support member spacing or modification of member sizes. When a redesign of the foundation is required, the Structures Division must resubmit the redesign information (new foundation layout, sizes, foundation load combinations, etc.) to the Geotechnical Section. The Geotechnical Section will analyze the new foundation and resubmit the necessary information to the Structures Division.

11.7.2 Types/Usage

The following summarizes NDOT's typical practices for the selection of the type of foundation.

11.7.2.1 Spread Footings

Reference: LRFD Article 10.6

Spread footings are NDOT's preferred foundation type if soils and settlement allow their use. They may also be used beneath retaining walls and sound walls. The use of spread footings requires firm bearing conditions; competent material must be near the ground surface (i.e., a maximum of 15 ft below the ground line). They are not allowed at stream crossings where they may be susceptible to scour.

A spread footing is a shallow foundation consisting of a reinforced concrete slab bearing directly on the founding stratum. A spread footing's geometry is determined by structural requirements and the characteristics of supporting components, such as soil or rock. Their primary role is to distribute the loads transmitted by piers or abutments to suitable soil strata or rock at relatively shallow depths.

Settlement criteria need to be consistent with the function and type of structure, anticipated service life and consequences of unanticipated movements on service performance. Longitudinal angular distortions between adjacent spread footings greater than 0.008 radians in simple spans and 0.004 radians in continuous spans should not ordinarily be permitted.

Ground modification techniques may be used to improve the soil allowing the use of spread footings where they would not otherwise be appropriate as determined by the Geotechnical Section. These techniques are typically used to address differential settlement concerns or to avoid potential liquefaction problems. These techniques include the construction of columns of gravel in the ground called stone columns or compaction grouting through the pressure injection of a slow-flowing water/sand/cement mix into a granular soil.

11.7.2.2 Driven Piles

A driven pile is a long, slender deep foundation element driven into the ground with power hammers.

If underlying soils cannot provide adequate bearing capacity or tolerable settlements for spread footings, driven piles may be used to transfer loads to deeper suitable strata through skin friction and/or point bearing. The selected type of pile is determined by the required bearing capacity, length, soil conditions and economic considerations. NDOT primarily uses steel pipe piles, but will occasionally consider steel H-piles or prestressed concrete piles. See Section17.3 for more information.

11.7.2.3 Drilled Shafts

Reference: LRFD Article 10.8

A drilled shaft (also called a caisson or cast-in-drilled-hole pile) is a long, slender deep-foundation element constructed by excavating a hole with auger equipment and placing concrete, with reinforcing steel, in the excavation. Casing and/or drilling slurry may be necessary to keep the excavation stable.

The bridge designer should use drilled shafts where significant scour is expected, where there are limits on in-stream work or tight construction zones, or where driven piles are not economically viable due to high loads or obstructions to driving. Limitations on vibration or construction noise may also dictate the selection of this foundation type. Drilled shafts can be a more costly foundation alternative.

11.8 ALTERNATIVES ANALYSIS

Many factors enter into the selection of the most suitable bridge type and size. Initial cost is important, but it should not be the only consideration. Durability and long-term maintenance requirements, aesthetics, constructibility, effect on the public and environment, use of falsework, geometric adaptability, quantity of embankment required for the approach roadway, permanent clearances, structural requirements, redundancy and other factors should be included as appropriate in the alternatives analysis.

The documentation for structure-type selection may be as little as several paragraphs of explanation or as detailed as a multi-page report. Cost of the bridge, project controversy, complexity of the site, public involvement and other issues will dictate the effort needed.

Every bridge could in theory have numerous alternatives. However, many alternatives can be eliminated due to their high cost or because they have a fatal flaw such as an incompatibility with a horizontally curved alignment. The analysis should be performed only on viable alternatives. Features such as location, span length, superstructure type, girder material and substructure type tend to dictate the need for an alternatives analysis. At least two viable superstructure types can usually be identified for most proposed bridges. This usually dictates the need for the alternatives analysis.

There are many available strategies to compare bridges in an alternatives analysis. There is no established method that is best for all projects. Use a rating method that supports the features of the project. The weighting of each evaluation factor can be used to provide more emphasis to certain factors if these factors warrant more consideration. Initial cost and effect on the public are evaluation factors that can have a major influence on the selection of a bridge type, but this will vary from project to project.

Foundations can be either shallow or deep. Shallow foundations are significantly less expensive than deep foundations. Bridges located at different sites can have different foundation requirements. These differences must be included in an alternative's cost.

11.9 ROADWAY DESIGN ELEMENTS

11.9.1 Coordination

In general, the roadway design criteria will determine the geometric design of the roadway, and the bridge design will accommodate the roadway design across any structures within the project limits. This will provide full continuity of the roadway section for the entire project. This process will, of course, require communication between the bridge designer and roadway designer to identify and resolve any inconsistencies. This Section provides roadway design information that is directly relevant to determining the structural dimensions for the preliminary bridge design and to provide the bridge designer with some background in roadway design elements.

The Roadway Design Division is involved with all bridge projects, and the Structures Division and Roadway Design Division collaborate on the roadway design features crossing the bridge. Initially, Roadway sets the geometrics, which is based on Section 2.2 of the *NDOT Project Design Development Manual*. The bridge designer will check the proposed geometric design (e.g., clearances, horizontal curves, vertical curves, roadway approach, cross slopes, widths) to identify any modifications that may be warranted to better accommodate structural design considerations. Any proposed modifications are communicated to the Roadway Design Division.

11.9.2 Highway Systems

11.9.2.1 Functional Classification System

The functional classification concept is one of the most important determining factors in highway design. The functional classification system recognizes that the public highway network serves two basic and often conflicting functions — travel mobility and access to property. In the functional classification scheme, the overall objective is that the highway system, when viewed in its entirety, will yield an optimum balance between its access and mobility purposes.

The functional classification system provides the guidelines for determining the geometric design of individual highways and streets. Based on the function of the facility, the roadway designer selects an appropriate design speed, roadway width, roadside safety elements, amenities and other design values.

The following briefly describes the characteristics of the various functional classifications.

11.9.2.1.1 *Arterials*

Arterial highways are characterized by a capacity to quickly move relatively large volumes of traffic and by a restricted function to serve abutting properties. The arterial system typically provides for high travel speeds and the longest trip movements. The arterial functional class is subdivided into principal and minor categories for both rural and urban areas.

Principal arterials provide the highest traffic volumes and the greatest trip lengths. The freeway, which includes Interstate highways, is the highest level of arterial. In rural areas, minor arterials will provide a mix of interstate and interregional travel service. In urban areas, minor arterials may carry local bus routes and provide intra-community connections.

11.9.2.1.2 *Collectors*

Collector routes are characterized by a roughly even distribution between access and mobility functions. Traffic volumes will typically be somewhat lower than those of arterials. In rural areas, collectors serve intra-regional needs and provide connections to the arterial system. In urban areas, collectors act as intermediate links between the arterial system and points of origin and destination.

11.9.2.1.3 *Local Roads and Streets*

All public roads and streets not classified as arterials or collectors are classified as local roads and streets. These facilities are characterized by their many points of direct access to adjacent properties and their relatively minor value in accommodating mobility.

11.9.2.2 Federal-Aid System

11.9.2.2.1 *Background*

The Federal-aid system consists of those routes within Nevada that are eligible for the categorical Federal highway funds. NDOT, working with the local governments and in cooperation with FHWA, has designated the eligible routes. The following briefly describes the components of the Federal-aid system.

11.9.2.2.2 *National Highway System*

The National Highway System (NHS) is a network of principal arterial routes identified as essential for international, interstate, and regional commerce and travel and for national defense. It consists of the Interstate highway system, logical additions to the Interstate system, selected other principal arterials and other facilities that meet the requirements of one of the subsystems within the NHS.

11.9.2.2.3 *Surface Transportation Program*

The Surface Transportation Program (STP) is a flexible funding program that provides Federal-aid funds for:

- highway projects on all functional classes (except facilities functionally classified as "local"),
- bridge projects on any public road (including "local" functional classes),
- transit capital projects, and
- public bus terminals and facilities.

The basic objective of STP is to provide Federal-aid for improvements to facilities not considered to have significant national importance (i.e., facilities not on the NHS) and to minimize the Federal requirements for funding eligibility. The Federal funds allocated to STP are comparable to those funds previously designated for use on the former Federal-aid primary, Federal-aid urban and Federal-aid secondary systems. STP funds are distributed to each State based on its lane-miles of Federal-aid highways, total vehicle-miles traveled on those highways, and estimated contributions to the Highway Trust Fund.

11.9.2.2.4 *Highway Bridge Program*

The Highway Bridge Program (HBP), formerly known as the Highway Bridge Rehabilitation and Replacement Program, provides funds for eligible bridges located on any public road. The HBP is the cornerstone of FHWA's efforts to correct, on a priority basis, deficient bridges throughout the nation. The number of structurally deficient and/or functionally obsolete bridges in Nevada compared to the number nationwide basically determines Nevada's share of HBP funds.

HBP funds available to non-State maintained facilities are based on the provision that no less than 15% of the funds must be used on public roads that are functionally classified as local roads (urban and rural) or rural minor collectors.

HBP funds can be used for total replacement or for rehabilitation. HBP funds can also be used for a nominal amount of roadway approach work to tie the new bridge in with the existing alignment or to tie in with a new gradeline. HBP funds cannot be used for long approach fills, causeways, connecting roadways, interchanges, ramps and other extensive earth structures.

Eligibility for HBP funding is based on a Sufficiency Rating (SR) (0-100). The SR is based on an equation that considers many aspects of a bridge (e.g., structural adequacy, safety, serviceability, functionality, detour length). The following applies:

1. Replacement. Bridges scheduled for replacement require an SR less than 50 and must be classified as structurally deficient or functionally obsolete.

2. Rehabilitation. Bridges scheduled for rehabilitation require an SR less than 80 and must be classified as structurally deficient or functionally obsolete.

3. Exception. If the cost of rehabilitation is greater than replacement, then coordination with FHWA is required to determine if the bridge can be replaced.

4. 10-Year Rule. If a bridge has been rehabilitated or replaced with HBP funds, it is not eligible for additional HBP funds for 10 years.

5. SR $\geq$ 80. If a bridge has an SR greater than or equal to 80, it is not eligible for HBP funds.

Section 22.1.3 and Section 28.2.12.3 discuss the Sufficiency Rating in more detail.

11.9.2.3 Jurisdictional Responsibilities

This Section briefly discusses the jurisdictional responsibility for the public highway system in Nevada.

11.9.2.3.1 *State-Maintained System*

The Nevada State-Maintained System represents those public highways, roads and streets for which NDOT has direct jurisdictional responsibility for all planning, design, construction and maintenance. The State-Maintained System may be identified by the route shield used on the facility, which may be:

- an Interstate Route,
- a US Route, or
- a Nevada State Route.

Frontage roads are also on the State-maintained system. Note that the State-Maintained System is not equivalent to the Federal-aid System, which is based on the functional classification system. The Federal-aid System includes most State-maintained routes and selected higher functional classification facilities not on the State-Maintained System.

11.9.2.3.2 *County/Municipal System*

For all public roads and streets not on the State-Maintained System, either a county or local municipality has jurisdictional responsibility for the facility.

11.9.3 Roadway Definitions

The following defines selected roadway elements that often have an application to the roadway design portion of a bridge:

1. Average Annual Daily Traffic (AADT). The total volume of traffic passing a point or segment of a highway facility, in both directions, for one year, divided by the number of days in the year.

2. Average Daily Traffic (ADT). The total volume of traffic during a given time period, greater than one day and less than one year, divided by the number of days in that time period.

3. Average Daily Truck Traffic (ADTT). The total number of trucks passing a point or segment of a highway facility, in both directions, during a given time period divided by the number of days in that time period.

4. Cross Slope. The slope in the cross section view of the travel lanes, expressed as a percent or ratio, based on the change in horizontal compared to the change in vertical.

5. Design Hourly Volume (DHV). Typically, the 30^{th} highest hourly volume for the future year used for design, expressed in vehicles per hour.

6. Design Speed. The maximum safe speed that can be maintained over a specified section of highway.

7. K-Values for Vertical Curves. The horizontal distance needed to produce a 1% change in longitudinal gradient.

8. Longitudinal Grade. The rate of roadway slope expressed as a percent between two adjacent Vertical Points of Intersection (VPI). Upgrades in the direction of stationing are identified as positive (+). Downgrades are identified as negative (–).

9. Median. On a multilane facility, the area (or distance) between the inside edges of the two traveled ways. Note that the median width includes the two inside (or left) shoulders.

10. Normal Crown (NC). The typical cross section on a tangent section of roadway (i.e., no superelevation).

11. Overpass. A grade separation where a highway passes over an intersecting highway or railroad.

12. Profile Grade Point (Finished Grade). The line at which the profile grade is measured on the pavement.

13. Roadway. The portion of a highway, including shoulders, for vehicular use. A divided highway includes two roadways.

14. Superelevation. The amount of cross slope provided on a horizontal curve to counterbalance, in combination with the side friction, the centrifugal force of a vehicle traversing the curve.

15. Superelevation Transition Length. The distance needed to transition the roadway from a normal crown section to the design superelevation rate. Superelevation transition length is the sum of the tangent runout (TR) and superelevation runoff (L) distances.

16. Traveled Way. The portion of the roadway for the movement of vehicles, exclusive of shoulders and auxiliary lanes.

17. Truck. A heavy vehicle engaged primarily in the transport of goods and materials, or in the delivery of services other than public transportation. For geometric design and capacity analyses, trucks are defined as vehicles with six or more tires.

18. Truck Percentage (T). The percentage of trucks in the total traffic volume on a facility.

19. Twenty-Year ADT. For new construction and reconstruction projects, the projected future traffic volume most often used in project design.

20. Underpass. A grade separation where a highway passes under an intersecting highway or railroad.

11.9.4 Roadway Cross Section (Bridges)

11.9.4.1 Profile Grade Point/Profile Grade Line

The location of the profile grade point (which is in the cross section view) and the profile grade line (which is in the elevation view) on the bridge must match those on the approaching roadway. The profile grade point location varies according to the type of highway and type of median.

The profile grade is located between the dense-grade and open-grade on bituminous pavements and the top of the concrete on concrete pavements. Adjustments to bridge elevations are required to match the riding surface of bituminous pavements. See Section 16.3 for more discussion.

11.9.4.2 Cross Slope

Bridges on tangent sections typically provide a uniform cross slope of 2.0% from the crown line to the concrete barrier rail. On rare occasions, the cross slope of the shoulder (and sometimes one or more of the travel lanes) on the approaching roadway is steeper than 2.0%; therefore, the roadway must be transitioned to a uniform 2.0% slope before it reaches the bridge; this is the responsibility of the roadway designer when designing the roadway approaches.

11.9.4.3 Bridge Roadway Widths

In general, bridge widths should match the approach roadway widths (traveled way plus shoulders plus auxiliary lanes), which are determined by the Roadway Design Division. However, in determining the width for major water crossings, consider the cost of the structure, traffic volumes and potential for future width requirements.

11.9.4.4 Sidewalks

11.9.4.4.1 *Warrants*

The Roadway Design Division determines the warrants for sidewalks on the approach roadway and, if provided, the sidewalks are carried across the bridge. Sidewalk requirements for each side of the bridge will be evaluated individually; i.e., placing a sidewalk on each side will be based on the specific characteristics of that side. However, typical NDOT practice is to place a sidewalk on both sides of the bridge.

11.9.4.4.2 *Cross Section*

The typical sidewalk width is 5′-6″ as measured from the gutter line to the back of the sidewalk; i.e., this width includes the width of curb. The cross slope on the sidewalk is 2% sloped towards the roadway.

11.9.4.4.3 *ADA Requirements*

In general, the Roadway Design Division is responsible for establishing NDOT criteria to comply with the requirements of the *Americans with Disabilities Act*. ADA requirements pertain to sidewalk features such as width, cross slope and longitudinal grade, surface type and texture, curb ramps, etc. The bridge designer should coordinate with the Roadway Design Division to ensure that any sidewalk across a bridge meets the ADA requirements.

11.9.4.5 Bicycle Accommodation

The bicycle is classified as a vehicle according to Nevada law, and bicyclists are granted all of the rights and are subject to all of the duties applicable to the driver of any other vehicle.

A bridge may need to be configured to accommodate bicycle traffic. This must be coordinated with the Statewide Bicycle/Pedestrian Coordinator in the Transportation Planning Division, who will refer to the State Transportation Improvement Program (STIP) to determine the necessary bicycle accommodation for a specific roadway segment. In general, the bicycle accommodation on the approaching roadway will be carried across the bridge. One method of accommodation is to provide a shoulder wide enough to accommodate bicycles. Although a 4-ft wide shoulder may be considered adequate for bicycle traffic on the roadway, the shoulder should be increased by 2 ft to provide a shy distance where barriers are present. Therefore, a 6-ft wide shoulder is considered the minimum shoulder width for bridges that are designed to carry bicycle traffic. In addition, on bridges, a minimum of 4 ft from the edge of the traveled way should be clear of drainage inlets.

If the approaching roadway includes a separate bicycle lane, then the width of the lane will be carried across the bridge. Requests for and accommodation for anticipated future bicycle lanes are only warranted when they are part of NDOT long-range plans (i.e., the STIP).

11.9.5 Alignment at Bridges

11.9.5.1 Horizontal Alignment

The roadway designer will determine the horizontal alignment at the bridge based on NDOT criteria adopted by the Roadway Design Division (e.g., curve radius, superelevation transition). From the perspective of the roadway user, a bridge is an integral part of the roadway system and, ideally, horizontal curves and their transitions will be located irrespective of their impact on bridges. However, practical factors in bridge design and bridge construction warrant consideration in the location of horizontal curves at bridges. The following presents, in order from the most desirable to the least desirable, the application of horizontal curves to bridges:

1. Considering both the complexity of design and construction difficulty, the most desirable treatment is to locate the bridge and its approach slabs on a tangent section; i.e., no portion of a horizontal curve or its superelevation development will be on the bridge or bridge approach slabs.

2. If a horizontal curve is located on a bridge, the superelevation transition should not be located on the bridge or its approach slabs. This will result in a uniform cross slope (i.e., the design superelevation rate) throughout the length of the bridge and bridge approach slabs.

3. If the superelevation transition is located on the bridge or its approach slabs, the road designer should place on the roadway approach that portion of the superelevation development that transitions the roadway cross section from its normal crown to a point where the roadway slopes uniformly; i.e., to a point where the crown has been removed. This will avoid the need to warp the crown on the bridge or the bridge approach slabs.

11.9.5.2 Vertical Alignment

The bridge designer and road designer will coordinate on the vertical alignment of the roadway across a bridge based on the criteria adopted by the Roadway Design Division. The following applies specifically to the vertical alignment at bridges:

1. Minimum Gradient. The minimum longitudinal gradient will be preferably 1% with an absolute minimum of 0.5%.

2. Maximum Grades. The Roadway Design Division has adopted NDOT's maximum grade criteria based on the highway type, design speed and rural/urban location.

3. Vertical Curves. Crest and sag vertical curves will be designed according to the criteria adopted by the Roadway Design Division. If practical, no portion of a bridge should be located in a sag vertical curve. If the bridge is located in a sag vertical curve, the low point of the sag should not be located on the bridge or the bridge approach slab.

11.9.5.3 Skew

Skew is defined as the angle between the end line of the deck and the normal drawn to the longitudinal centerline of the bridge at that point. Typically, the bridge skew is determined by the roadway alignment, and the bridge is designed to accommodate the skew. The impacts of skew on structural design are discussed at their respective locations throughout the *NDOT Structures Manual*. In general, skew angles of more than 30° will affect the design of structural elements.

11.9.6 <u>Highway Grade Separations</u>

For bridges over highways, the geometry of the underpassing roadway will determine the length of the overpassing bridge. See Chapter 21 for railroads underpassing a highway bridge.

11.9.6.1 Roadway Cross Section

The approaching roadway cross section, including any auxiliary lanes, bicycle lanes, sidewalks, etc., should be carried through the underpass. Desirably, also include the clear zone width for each side through the underpass, although this could prove to be prohibitively expensive. In addition, it is important to consider the potential for further development or traffic increases in the vicinity of the underpass that may significantly increase traffic or pedestrian volumes. If appropriate, an allowance for future widening may be provided to allow for sufficient lateral clearance for additional lanes. The need for accommodating future travel lanes will be made on a case-by-case basis.

11.9.6.2 Vertical Clearances

The vertical clearance for underpassing roadways will significantly impact the vertical alignment of the overpassing structure and may dictate the selection of the superstructure type. Figure 11.9-A summarizes NDOT's minimum vertical clearance criteria. Provide these clearances over the entire roadway beneath the bridge from edge of roadway to edge of roadway. Where barriers are present, provide the minimum vertical clearance from face of barrier to face of barrier. If a bridge is likely to be widened in the future, the minimum vertical clearance should also extend over the potential width of the widening.

11.9.6.3 Falsework

Falsework may unduly interfere with traffic passing beneath the structure or may create an unacceptable safety hazard. The bridge designer shall contact the District Office to judge the impact of using falsework over traffic. The minimum vertical clearance for falsework on all facilities is 16′-0″. Vertical clearances for collector and local roads may be reduced to 14′-6″ if a readily available detour for over-height vehicles is available and approved by the District Engineer and owner of the local road. All falsework shall have protection from high-load hits unless it has a vertical clearance of more than 18′-0″. Falsework can be protected by one of the following methods:

Facility Type	Minimum Clearance		
	New/Replaced Bridges	**Rehabilitated/Existing Bridges to Remain**	**Temporary Structures** (1)
Freeway Under	16′-6″	16′-0″ (5)	16′-0″
Arterial Under	16′-6″	16′-0″ (5)	16′-0″
Collector Under	16′-6″ (4)	16′-0″ (5)	16′-0″
Local Under	16′-6″ (4)	16′-0″ (5)	16′-0″
Highway Under Overhead Sign or Pedestrian Bridge (2)	18′-0″	18′-0″	N/A
Railroad Under Highway:			
(Non-Electrified)	23′-4″	23′-4″	21′-0″
Electrified (25-kv line) (3)	24′-3″	24′-3″	(6)
Electrified (50-kv line) (3)	26′-3″	26′-3″	(6)

Notes:

(1) *See Section 11.9.6.3 "Falsework." Contact the District Office and/or Railroad Company for concurrence on a case-by case basis.*

(2) *AASHTO* A Policy on Geometric Design of Highways and Streets *recommends a minimum vertical clearance of 17′-0″. NDOT has adopted a vertical clearance of 18′-0″ for both new structures and existing structures.*

(3) *The additional vertical clearance for electrification is acceptable only after the Railroad Company has submitted justification that it will provide electrification on the track line. See Section 21.1.3.3.*

(4) *AASHTO* A Policy on Geometric Design of Highways and Streets *recommends a minimum vertical clearance of 14′-6″. NDOT has adopted a vertical clearance of 16′-6″.*

(5) *AASHTO* A Policy on Geometric Design of Highways and Streets *recommends a minimum vertical clearance of 14′-6″. NDOT has adopted a vertical clearance of 16′-0″.*

(6) *Contact the Railroad Company for acceptable temporary vertical clearances.*

MINIMUM VERTICAL CLEARANCES

Figure 11.9-A

1. Overhead Barrier Girder. A stout girder is placed ahead of the falsework. The overhead barrier girder shall not be part of the falsework but its own structure. Overhead barrier girders should not be used over roadways with high traffic volumes or posted speed limits of more than 35 mph. Criteria for the design of the barrier girder shall be established on a case-by-case basis.

2. Over-height Detection System. An electronic eye detects an over-height vehicle and warns the vehicle driver and construction workers. Adequate stopping distances and turnaround areas must be provided. Evaluate the cost of the detection system and the cost and time to supply power when considering this detection system.

3. Adjacent Robust Structures. Falsework protection is not required if robust structures are located on the approaches to the falsework and if these structures have vertical clearances less than the falsework vertical clearance.

The designer shall coordinate with the Traffic Division to determine if lane closures, truck re-routing, detours or a complete road closure is an option where falsework is proposed. Additional traffic control and road user costs associated with the use of falsework shall be considered when selecting a structure type.

11.10 STRUCTURE LENGTH CALCULATIONS

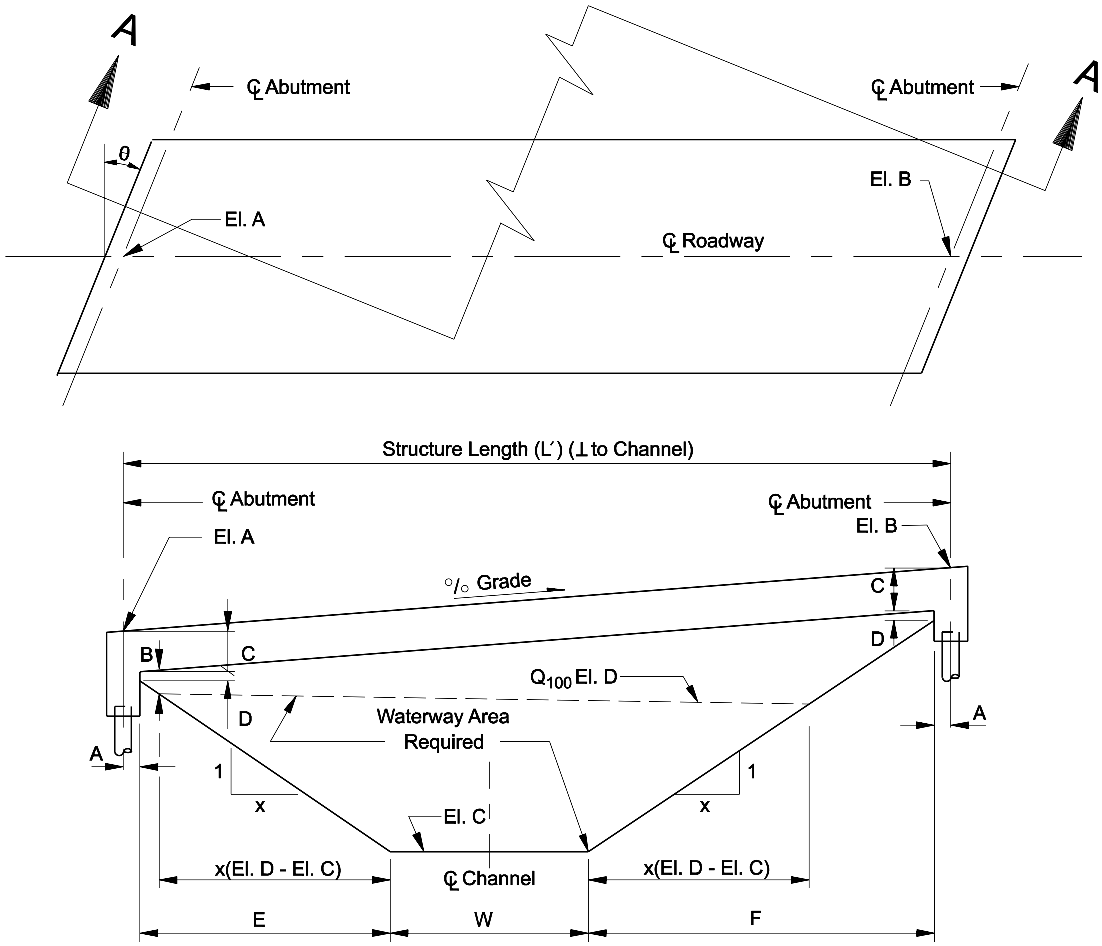
The overall structure length is measured from the centerline of abutment to the centerline of abutment. The following figures present criteria for determining structure length:

- Figure 11.10-A "Structure Length for Stream Crossings (Open Abutment)"
- Figure 11.10-B "Structure Length for Highway Crossings (Open Abutment)"
- Figure 11.10-C "Structure Length for Highway Crossings (Closed Abutment)"

The major variables that determine the structure length are:

- the use of an open abutment or closed abutment;
- seat width;
- for open abutments, the backslope;
- for waterway crossings, the waterway opening dimensions;
- for highway crossings, the width of the underpassing roadway cross section and clear zones;
- the longitudinal gradient along the roadway centerline; and
- the skew angle of the bridge.

The following figures assume that the bridge is on tangent and on a constant longitudinal gradient. The presence of a horizontal curve and/or a vertical curve will increase the length of the structure.



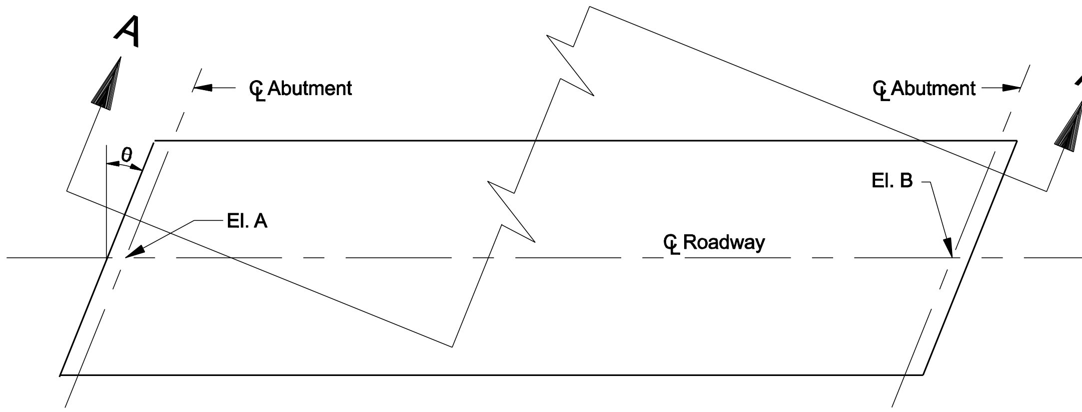
SECTION A-A
(Perpendicular to Channel)

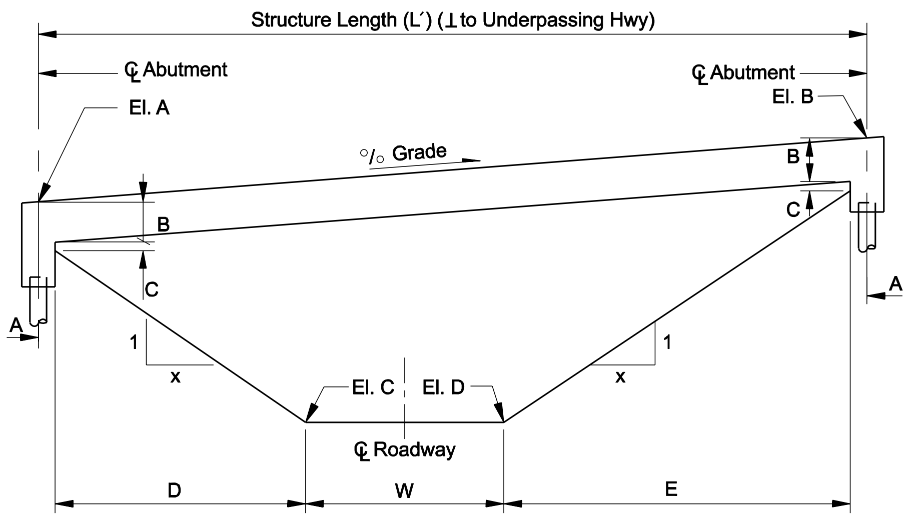
- θ = Angle of skew
- A = One half of the abutment width
- B = Freeboard
- C = Anticipated depth of superstructure
- D = Distance from bottom of superstructure to top of abutment backslope (2′ minimum)
- E = (x) (El. A – C – D – El. C)
- F = (x) (El. B – C – D – El. C)
- W = Width of channel (perpendicular to channel)
- El. A = Elevation of top of deck
- El. B = Elevation of top of deck
- El. C = Bottom of channel elevation
- El. D = Elevation of water surface at Q_{100}

- L′ = Structure length perpendicular to channel from ℄ abutment to ℄ abutment
- L = Structure length along ℄ roadway from ℄ abutment to ℄ abutment
- L′ = A + E + W + F + A
- L = L′/cos θ

STRUCTURE LENGTH FOR STREAM CROSSINGS
(Open Abutment)

Figure 11.10-A





SECTION A-A
(Perpendicular to Underpassing Hwy)

θ = Angle of skew
A = One half of the abutment width
B = Anticipated depth of superstructure
C = Distance from bottom of superstructure to top of abutment slope (2′ minimum)
D = (x) (El. A – B – C – El. C)
E = (x) (El. B – B – C – El. C)
W = Width of underpassing roadway section
El. A = Elevation of top of deck
El. B = Elevation of top of deck
El. C = Elevation of toe of slope
El. D = Elevation of toe of slope

L′ = Structure length perpendicular to underpassing highway from ℄ abutment to ℄ abutment

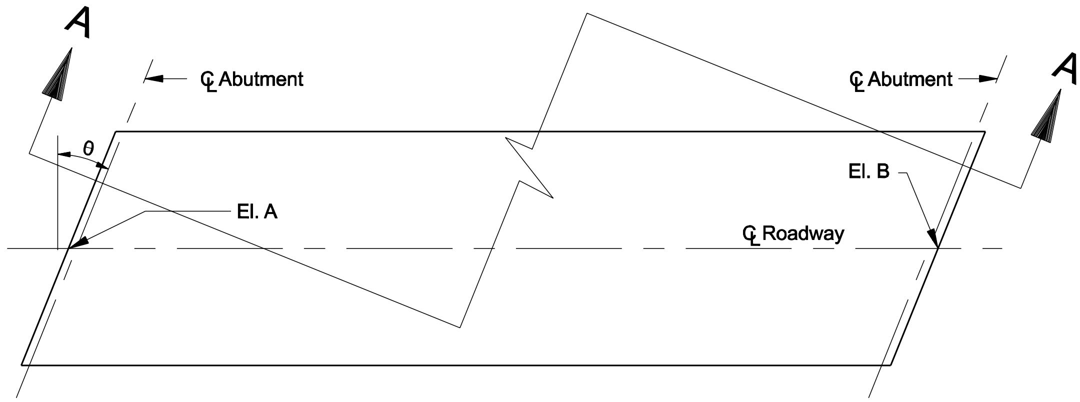
L = Structure length along ℄ roadway from ℄ abutment to ℄ abutment

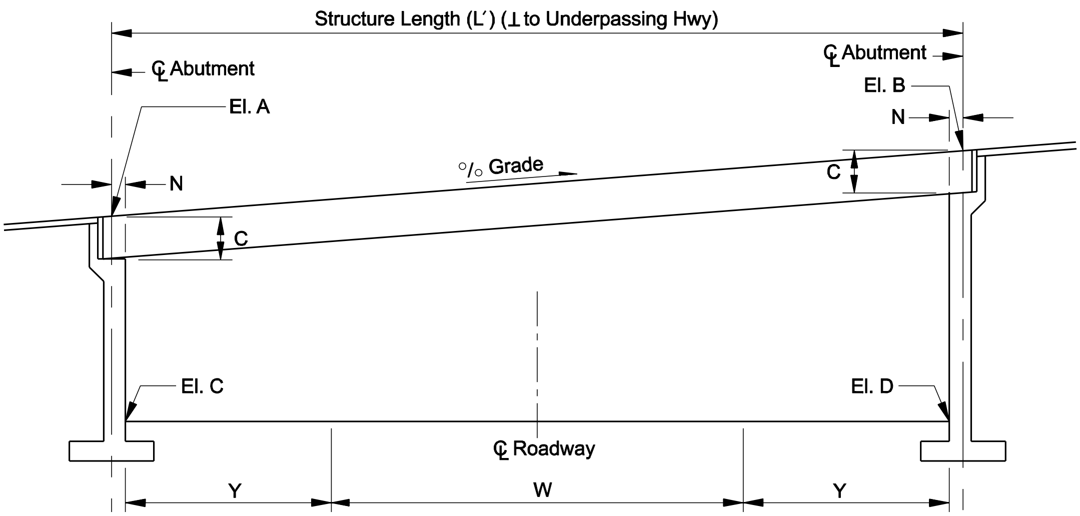
L′ = A + D + W + E + A

L = L′/cos θ

STRUCTURE LENGTH FOR HIGHWAY CROSSINGS
(Open Abutment)

Figure 11.10-B





SECTION A-A
(Perpendicular to Underpassing Hwy)

θ = Angle of skew
C = Anticipated depth of superstructure
N = One-half of seat width
W = Width of underpassing roadway section
Y = Width of clear zone, sidewalk, future expansion, bike lane
El. A = Elevation of top of deck at centerline of abutment
El. B = Elevation of top of deck at centerline of abutment
El. C = Elevation of toe of closed abutment
El. D = Elevation of toe of closed abutment

L′ = Structure length perpendicular to underpassing highway from ℄ abutment to ℄ abutment

L = Structure length along ℄ roadway from ℄ abutment to ℄ abutment

$L' = N + Y + W + Y + N$

$L = L'/\cos\theta$

STRUCTURE LENGTH FOR HIGHWAY CROSSINGS
(Closed Abutment)

Figure 11.10-C

Chapter 12

LOADS AND LOAD FACTORS

NDOT STRUCTURES MANUAL

September 2008

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(Continued)

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Chapter 12
LOADS AND LOAD FACTORS

Sections 1 and 3 of the *LRFD Bridge Design Specifications* discuss various aspects of loads. Unless noted otherwise in Chapter 12 of the *NDOT Structures Manual*, the *LRFD Specifications* applies to loads and load factors in Nevada. Chapter 12 also presents additional information on NDOT practices.

12.1 GENERAL

12.1.1 <u>Load Definitions</u>

Reference: LRFD Article 3.3.2

12.1.1.1 Permanent Loads

Reference: LRFD Article 3.5

Permanent loads are loads that are always present in or on the bridge and do not change in magnitude during the life of the bridge. Specific permanent loads include:

1. <u>Gravitational Dead Loads</u>.

    - DC – dead load of all of the components of the superstructure and substructure, both structural and non-structural.
    - DW – dead load of additional non-integral wearing surfaces, future overlays and any utilities crossing the bridge.
    - EL – accumulated lock-in, or residual, force effects resulting from the construction process, including the secondary forces from post-tensioning (which are not gravitational dead loads).
    - EV – vertical earth pressure from the dead load of earth fill.

2. <u>Earth Pressures</u>.

    Reference: LRFD Article 3.11

    - EH – horizontal earth pressure.
    - ES – earth pressure from a permanent earth surcharge (e.g., an embankment).
    - DD – loads developed along the vertical sides of a deep-foundation element tending to drag it downward typically due to consolidation of soft soils underneath embankments reducing its resistance.

12.1.1.2 Transient Loads

Transient loads are loads that are not always present in or on the bridge or change in magnitude during the life of the bridge. Specific transient loads include:

1. Live Loads.

   Reference: LRFD Article 3.6

   - LL – static vertical gravity loads due to vehicular traffic on the roadway.
   - PL – vertical gravity loads due to pedestrian traffic on sidewalks.
   - IM – dynamic load allowance to amplify the force effects of statically applied vehicles to represent moving vehicles, traditionally called impact.
   - LS – horizontal earth pressure from vehicular traffic on the ground surface above an abutment or wall.
   - BR – horizontal vehicular braking force.
   - CE – horizontal centrifugal force from vehicles on a curved roadway.

2. Water Loads.

   Reference: LRFD Article 3.7

   - WA – pressure due to differential water levels, stream flow or buoyancy.

3. Wind Loads.

   Reference: LRFD Article 3.8

   - WS – horizontal and vertical pressure on superstructure or substructure due to wind.
   - WL – horizontal pressure on vehicles due to wind.

4. Extreme Events.

   - EQ – loads due to earthquake ground motions.

     Reference: LRFD Article 3.10

   - CT – horizontal impact loads on abutments or piers due to vehicles or trains.

     Reference: LRFD Article 3.6.5

   - CV – horizontal impact loads due to aberrant ships or barges.

     Reference: LRFD Article 3.14

- IC – horizontal static and dynamic forces due to ice action.

  Reference: LRFD Article 3.9

5. Superimposed Deformations.

   Reference: LRFD Article 3.12

   - TU – uniform temperature change due to seasonal variation.
   - TG – temperature gradient due to exposure of the bridge to solar radiation.
   - SH – differential shrinkage between different concretes or concrete and non-shrinking materials, such as metals and wood.
   - CR – creep of concrete or wood.
   - SE – the effects of settlement of substructure units on the superstructure.

6. Friction Forces.

   Reference: LRFD Article 3.13

   - FR – frictional forces on sliding surfaces from structure movements.

12.1.2 Limit States

Reference: LRFD Article 1.3.2

The *LRFD Specifications* groups the traditional design criteria together within groups termed "limit states." The *LRFD Specifications* assigns load combinations to the various limit states.

12.1.2.1 Basic LRFD Equation

Components and connections of a bridge are designed to satisfy the basic LRFD equation for all limit states:

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n \qquad \text{(LRFD Eq. 1.3.2.1-1)}$$

Where:

γ_i = load factor
Q_i = load or force effect
ϕ = resistance factor
R_n = nominal resistance
η_i = load modifier as defined in LRFD Equations 1.3.2.1-2 and 1.3.2.1-3

The left-hand side of LRFD Equation 1.3.2.1-1 is the sum of the factored load (force) effects acting on a component; the right-hand side is the factored nominal resistance of the component. The Equation must be considered for all applicable limit state load combinations. Similarly, the Equation is applicable to superstructures, substructures and foundations.

For the Strength limit states, the *LRFD Specifications* is basically a hybrid design code in that the force effect on the left-hand side of the LRFD Equation is based upon elastic structural response, while resistance on the right-hand side of the Equation is determined predominantly by applying inelastic response principles. The *LRFD Specifications* has adopted the hybrid nature of strength design on the assumption that the inelastic component of structural performance will always remain relatively small because of non-critical redistribution of force effects. This non-criticality is assured by providing adequate redundancy and ductility of the structures, which is NDOT's general policy for the design of bridges.

12.1.2.2 Load Modifier

The load modifier η_i relates the factors η_D, η_R and η_i to ductility, redundancy and operational importance. The location of η_i on the load side of the LRFD Equation may appear counterintuitive because it appears to be more related to resistance than to load. η_i is on the load side for a logistical reason. When η_i modifies a maximum load factor, it is the product of the factors as indicated in LRFD Equation 1.3.2.1-2; when η_i modifies a minimum load factor, it is the reciprocal of the product as indicated in LRFD Equation 1.3.2.1-3. These factors are somewhat arbitrary; their significance is in their presence in the *LRFD Specifications* and not necessarily in the accuracy of their magnitude. The LRFD factors reflect the desire to promote redundant and ductile bridges.

NDOT uses η_i values of 1.00 for all limit states, because bridges designed in accordance with the *NDOT Structures Manual* will demonstrate traditional levels of redundancy and ductility. Rather than penalize less redundant or less ductile bridges, such bridges are not encouraged. NDOT may on a case-by-case basis designate a bridge to be of special operational importance and specify an appropriate value of η_i.

The load modifier accounting for importance of LRFD Article 1.3.5, η_I, should not be confused with the importance categories for seismic design of LRFD Articles 3.10.3 and 4.7.4.3. The importance load modifier is used in the basic LRFD Equation, but the importance categories are used to determine the minimum seismic analysis requirements.

12.1.3 <u>Load Factors and Combinations</u>

Reference: LRFD Article 3.4.1

LRFD Table 3.4.1-1 provides the load factors for all of the load combinations of the *LRFD Specifications*.

12.1.3.1 Strength Load Combinations

The load factors for the Strength load combinations are calibrated based upon structural reliability theory and represent the uncertainty of their associated loads. The significance of the Strength load combinations can be simplified as follows:

1. <u>Strength I Load Combination</u>. This load combination represents random traffic and the heaviest truck to cross the bridge in its 75-year design life. During this live-load event, a significant wind is not considered probable.

2. Strength II Load Combination. In the *LRFD Specifications*, this load combination represents an owner-specified permit load model. This live-load event has less uncertainty than random traffic and, thus, a lower live-load load factor. This load combination is used for design in conjunction with the permit live load design vehicle (P loads) discussed in Section 12.3.2.7

3. Strength III Load Combination. This load combination represents the most severe wind during the bridge's 75-year design life. During this severe wind event, no significant live load is assumed to cross the bridge.

4. Strength IV Load Combination. This load combination represents an extra safeguard for bridge superstructures where the unfactored dead load exceeds seven times the unfactored live load. Thus, the only significant load factor would be the 1.25 dead-load maximum load factor. For additional safety, and based solely on engineering judgment, the *LRFD Specifications* has arbitrarily increased the load factor for DC to 1.5. This load combination need not be considered for any component except a superstructure component, and never where the unfactored dead-load force effect is less than seven times the unfactored live-load force effect. This load combination typically governs only for longer spans, approximately greater than 200 ft in length. Thus, this load combination will only be necessary in relatively rare cases.

5. Strength V Load Combination. This load combination represents the simultaneous occurrence of a "normal" live-load event and a "55-mph" wind event with load factors of 1.35 and 0.4, respectively.

For components not traditionally governed by wind force effects, the Strength III and Strength V load combinations should not govern. Generally, the Strength I and Strength II load combinations will govern for a typical multi-girder highway bridge.

12.1.3.2 Service Load Combinations

Unlike the Strength load combinations, the Service load combinations are material dependent. The following applies:

1. Service I Load Combination. This load combination is applied for controlling cracking in reinforced concrete components and compressive stresses in prestressed concrete components. This load combination is also used to calculate deflections and settlements of superstructure and substructure components.

2. Service II Load Combination. This load combination is applied for controlling permanent deformations of compact steel sections and the "slip" of slip-critical (i.e., friction-type) bolted steel connections.

3. Service III Load Combination. This load combination is applied for controlling tensile stresses in prestressed concrete superstructure components under vehicular traffic loads. The Service III load combination need not apply to the design permit live load design vehicle.

4. Service IV Load Combination. This load combination is applied for controlling tensile stresses in prestressed concrete substructure components under wind loads. For components not traditionally governed by wind effects, this load combination should not govern.

12.1.3.3 Extreme-Event Load Combinations

The Extreme-Event limit states differ from the Strength limit states, because the event for which the bridge and its components are designed has a greater return period than the 75-year design life of the bridge (or a much lower frequency of occurrence than the loads of the Strength limit state). The following applies:

1. Extreme-Event I Load Combination. This load combination is applied to earthquakes. Because of the high seismicity in specific regions of Nevada, this load combination often governs design. Earthquakes in conjunction with scour (which is considered a change in foundation condition, not a load) can result in a very costly design solution if severe scour is anticipated. In this case, NDOT practice is to combine one-half of the total design scour (sum of contraction, local and long-term scour) with the full seismic loading.

2. Extreme-Event II Load Combination. This load combination is applied to various types of collisions (vessel, vehicular or ice) applied individually.

12.1.3.4 Fatigue-and-Fracture Load Combination

The Fatigue-and-Fracture load combination, although strictly applicable to all types of superstructures, only affects the steel elements, components and connections of a limited number of steel superstructures. Chapter 15 discusses fatigue and fracture for steel.

12.1.3.5 Application of Multiple-Valued Load Factors

12.1.3.5.1 *Maximum and Minimum Permanent-Load Load Factors*

In LRFD Table 3.4.1-1, the variable γ_P represents load factors for all of the permanent loads, shown in the first column of load factors. This variable reflects that the Strength and Extreme-Event limit state load factors for the various permanent loads are not single constants, but they can have two extreme values. LRFD Table 3.4.1-2 provides these two extreme values for the various permanent load factors, maximum and minimum. Permanent loads are always present on the bridge, but the nature of uncertainty is that the actual loads may be more or less than the nominal specified design values. Therefore, maximum and minimum load factors reflect this uncertainty.

The designer should select the appropriate maximum or minimum permanent-load load factors to produce the more critical load effect. For example, in continuous superstructures with relatively short-end spans, transient live load in the end span causes the bearing to be more compressed, while transient live load in the second span causes the bearing to be less compressed and perhaps lift up. To check the maximum compression force in the bearing, place the live load in the end span and use the maximum DC load factor of 1.25 for all spans. To check possible uplift of the bearing, place the live load in the second span and use the minimum DC load factor of 0.90 for all spans.

Superstructure design uses the maximum permanent-load load factors almost exclusively, with the most common exception being uplift of a bearing as discussed above. The AASHTO *Standard Specifications* treated uplift as a separate load combination. With the introduction of maximum and minimum load factors, the *LRFD Specifications* has generalized load situations such as uplift where a permanent load (in this case a dead load) reduces the overall force effect (in this case a reaction). Permanent load factors, either maximum or minimum, must be selected for each load combination to produce extreme force effects.

Substructure design routinely uses the maximum and minimum permanent-load load factors from LRFD Table 3.4.1-2. An illustrative yet simple example is a spread footing supporting a cantilever retaining wall. When checking bearing, the weight of the soil (EV) over the heel is factored up by the maximum load factor, 1.35, because greater EV increases the bearing pressure, q_{ult}, making the limit state more critical. When checking sliding, EV is factored by the minimum load factor, 1.00, because lesser EV decreases the resistance to sliding, Q_{τ}, again making the limit state more critical. The application of these maximum and minimum load factors is required for foundation and substructure design; see Chapters 17 and 18.

12.1.3.5.2 *Load Factors for Superimposed Deformations*

The load factors for the superimposed deformations (TU, CR, SH) for the Strength limit states also have two specified values — a load factor of 0.5 for the calculation of stress, and a load factor of 1.2 for the calculation of deformation. The greater value of 1.2 is used to calculate unrestrained deformations (e.g., a simple span expanding freely with rising temperature). The lower value of 0.5 for the elastic calculation of stress reflects the inelastic response of the structure due to restrained deformations. For example, one-half of the temperature rise would be used to elastically calculate the stresses in a constrained structure. Using 1.2 times the temperature rise in an elastic calculation would overestimate the stresses in the structure. The structure resists the temperature inelastically through redistribution of the elastic stresses.

12.2 PERMANENT LOADS

12.2.1 General

Reference: LRFD Article 3.5

The *LRFD Specifications* specifies seven components of permanent loads, which are either direct gravity loads or caused by gravity loads. The primary forces from prestressing are considered to be part of the resistance of a component and has been omitted from the list of permanent loads in Section 3 of the *LRFD Specifications*. However, when designing anchorages for prestressing tendons, the prestressing force is the only load effect, and it should appear on the load side of the LRFD Equation. The permanent load EL includes secondary forces from pre-tensioning or post-tensioning. As specified in LRFD Table 3.4.1-2, use a constant load factor of 1.0 for both maximum and minimum load factors for EL.

As discussed in Section 12.1.3.5.1, the permanent force effects in superstructure design are factored by the maximum permanent-load load factors almost exclusively, with the most common exception being the check for uplift of a bearing. In substructure design, the permanent force effects are routinely factored by the maximum or minimum permanent-load load factors from LRFD Table 3.4.1-2 as appropriate.

12.2.2 Deck Slab Dead Load

12.2.2.1 General

Loads applied to the composite cross section (i.e., the girder with the slab over it) include the weight of any raised median, rail, sidewalk or barrier placed after the deck concrete has hardened. Include a uniform load of 38 psf to account for a wearing surface over the entire deck area between the face of rails or sidewalks.

12.2.2.2 Composite Girders

Reference: LRFD Articles 6.10.1.1.1 and 9.7.4

Bridge deck slab dead load (DL) for design consists of composite and non-composite components. Loads applied to the non-composite cross section (i.e., the girder alone) include the weight of the plastic concrete, forms and other construction loads typically required to place the deck. Calculate the non-composite DL using the full-slab volume including haunches.

Where steel stay-in-place formwork is used, the designer shall account for the steel form weight and any additional concrete in the flutes of the formwork. The combined weight of the form and concrete in the flutes shall not exceed 15 psf.

12.2.2.3 Cast-in-Place Concrete Box Girders

The designer shall account for the weight of lost deck forms by including an additional load of 12 psf.

12.2.3 Distribution of Dead Load to Girders

Reference: LRFD Article 4.6.2.2.1

For the distribution of the weight of plastic concrete to the girders, including that of an integral sacrificial wearing surface, assume that the formwork is simply supported between interior girders and cantilevered over the exterior girders.

Superimposed dead loads (e.g., curbs, barriers, sidewalks, parapets, railings, future wearing surfaces) placed after the deck slab has cured may be distributed equally to all girders as traditionally specified by AASHTO except for girder bridges with more than six girders. For wider bridges with more than six girders, assume that the superimposed dead loads of sidewalks, parapets or railings are carried by the three girders immediately under and adjacent to the load. In some cases, such as staged construction and heavier utilities, the bridge designer should conduct a more refined analysis, as discussed in Section13.2, to determine a more accurate distribution of superimposed dead loads.

For cast-in-place concrete box girders, assume equal distribution across the full bridge deck width.

12.2.4 Downdrag on Deep Foundations

Reference: LRFD Article 3.11

Deep foundations (i.e., driven piles and drilled shafts) through unconsolidated soil layers may be subject to downdrag, DD. Downdrag is a load developed along the vertical sides of a deep-foundation element tending to drag it downward typically due to consolidation of soft soils underneath embankments reducing its resistance. Calculate this additional load as a skin-friction effect. If possible, the bridge designer should detail the deep foundation to mitigate the effects of downdrag; otherwise, it is necessary to design considering downdrag. Section 17.3.3.1 discusses mitigation methods.

12.2.5 Differential Settlement

Differential settlement between adjacent substructure units or transversely across a single substructure unit induces stresses in continuous structures and deflections in simple structures. Although most bridges can easily resist these stresses and deflections, the potential effects of differential settlement should be considered for all structures. The effects of differential settlement in the longitudinal direction need not be considered if its magnitude is ½ in or less. The effects of differential settlement in the transverse direction should be considered on a case-by-case basis.

12.3 TRANSIENT LOADS

12.3.1 General

The *LRFD Specifications* recognizes 19 transient loads. Static water pressure, stream pressure, buoyancy and wave action are integrated as water load, WA. Creep, settlement, shrinkage and temperature (CR, SE, SH, TU and TG) are elevated in importance to "loads," being superimposed deformations which, if restrained, will result in force effects. For example, restrained strains due to uniform-temperature increase induces compression forces. The *LRFD Specifications* has considerably increased the vehicular braking force (BR) to reflect the improvements in the mechanical capability of modern trucks in comparison with the traditional values of the AASHTO *Standard Specifications*.

12.3.2 Vehicular Live Load (LL)

12.3.2.1 General

Reference: LRFD Articles 3.6.1.1, 3.6.1.2 and 3.6.1.3

For short and medium span bridges, which predominate in Nevada, vehicular live load is the most significant component of load. Dead loads become more significant for long-span bridges. Long-span bridges are defined as those governed by the Strength IV load combination where the dead load is seven times or more greater than the live load.

12.3.2.2 The Nature of the Notional Load

The HL-93 live-load model is a notional load in that it is not a true representation of actual truck weights. Instead, the force effects (i.e., the moments and shears) due to the superposition of vehicular and lane load within a single design lane are a true representation of the force effects due to actual trucks.

The components of the HL-93 notional load are:

- a vehicle, either the familiar HS-20 truck, now called the design truck, or a 50-kip design tandem, similar to the Alternate Loading, both of the *Standard Specifications*; and

- a 0.64 k/ft uniformly distributed lane load, similar to the lane load of the *Standard Specifications*, but without any of the previous associated concentrated loads.

Note that the dynamic load allowance (IM) of 0.33 is applicable only to the design trucks and the design tandems, but not to the uniformly distributed lane load.

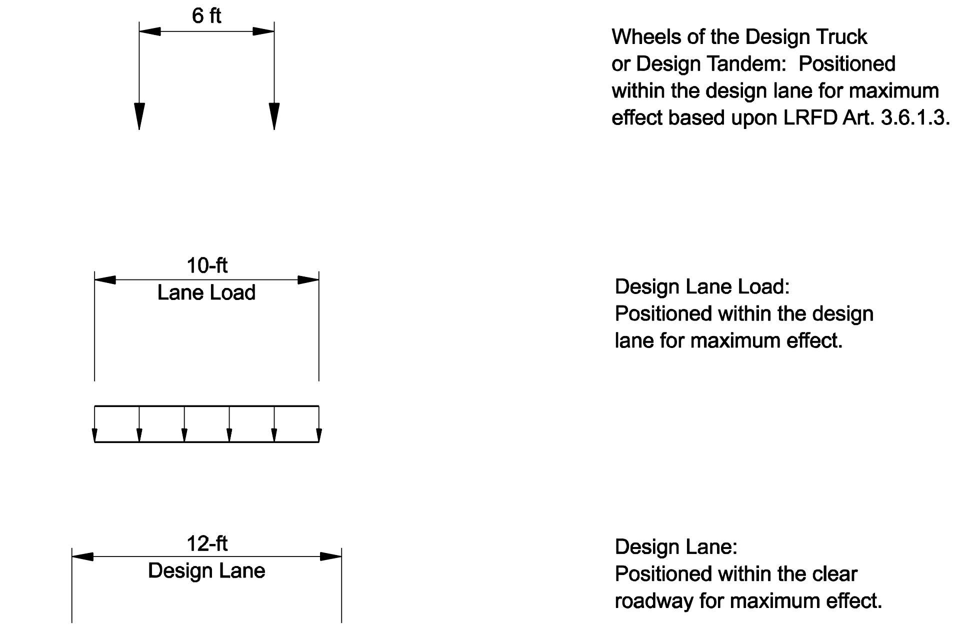
The force effects of the traditional HS-20 truck alone are less than that of the legal loads. Thus, a heavier vehicle is appropriate for design. As specified for the HL-93 live-load model, the concept of superimposing the design vehicle force effects and the design lane force effects was developed to yield moments and shears representative of real trucks on the highways. The moments and shears produced by the HL-93 load model are essentially equivalent to those of a 57-ton truck.

12.3.2.3 Multiple Presence Factors

The multiple presence factor of 1.0 for two loaded lanes, as given in LRFD Table 3.6.1.1.2-1, is the result of the *LRFD Specifications'* calibration for the notional load, which has been normalized relative to the occurrence of two side-by-side, fully correlated, or identical, vehicles. The multiple presence factor of 1.2 for one loaded lane should be used where a single design tandem or single design truck governs, such as in overhangs, decks, etc. The multiple-presence factors should not be applied to fatigue loads.

12.3.2.4 Application of Vehicles and Lanes

The *LRFD Specifications* retains the traditional design lane width of 12 ft and the traditional spacing of the axles and wheels of the HS-20 truck. Both vehicles (the design truck and design tandem) and the lane load occupy a 10-ft width placed transversely within the design lane for maximum effect, as specified in LRFD Article 3.6.1.3 and illustrated schematically in Figure 12.3-A.



PLACEMENT OF THE DESIGN LOADS WITHIN THE DESIGN LANES

Figure 12.3-A

12.3.2.5 Special Load Applications

12.3.2.5.1 *Two Design Trucks in a Single Lane for Negative Moment and Interior Reactions*

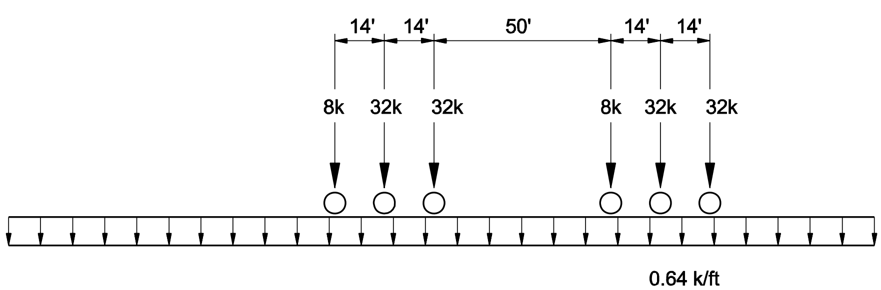
Reference: LRFD Article 3.6.1.3.1

The combination of the lane load and a single vehicle (either a design truck or a design tandem) does not always adequately represent the real-life loading of two heavy vehicles closely following one another, interspersed with other lighter traffic. Thus, a special load case has been specified in the *LRFD Specifications* to calculate these force effects. Two design trucks, with a fixed rear axle spacing of 14 ft and a clear distance not less than 50 ft between them, superimposed upon the lane load, all within a single design lane and adjusted by a factor of 0.90 approximates a statistically valid representation of negative moment and interior reactions due to closely spaced heavy trucks. This sequence of highway loading is specified for negative moment and reactions at interior piers due to the shape of the influence lines for such force effects. This sequence is not extended to other structures or portions of structures because it is not expected to govern for other influence-line shapes. This loading is illustrated in Figure 12.3-B.

In positioning the two trucks to calculate negative moment or the interior reaction over an internal support of a continuous girder, spans should be at least 90 ft in length to be able to position a truck in each span's governing position (over the peak of the influence line). If the spans are larger than 90 ft in length, the trucks remain in the governing positions but, if they are smaller than 90 ft, the maximum force effect can only be attained by trial-and-error with either one or both trucks in off-positions (i.e., non-governing positions for each individual span away from the peak of the influence line). These effects are illustrated in Figure 12.3-C.

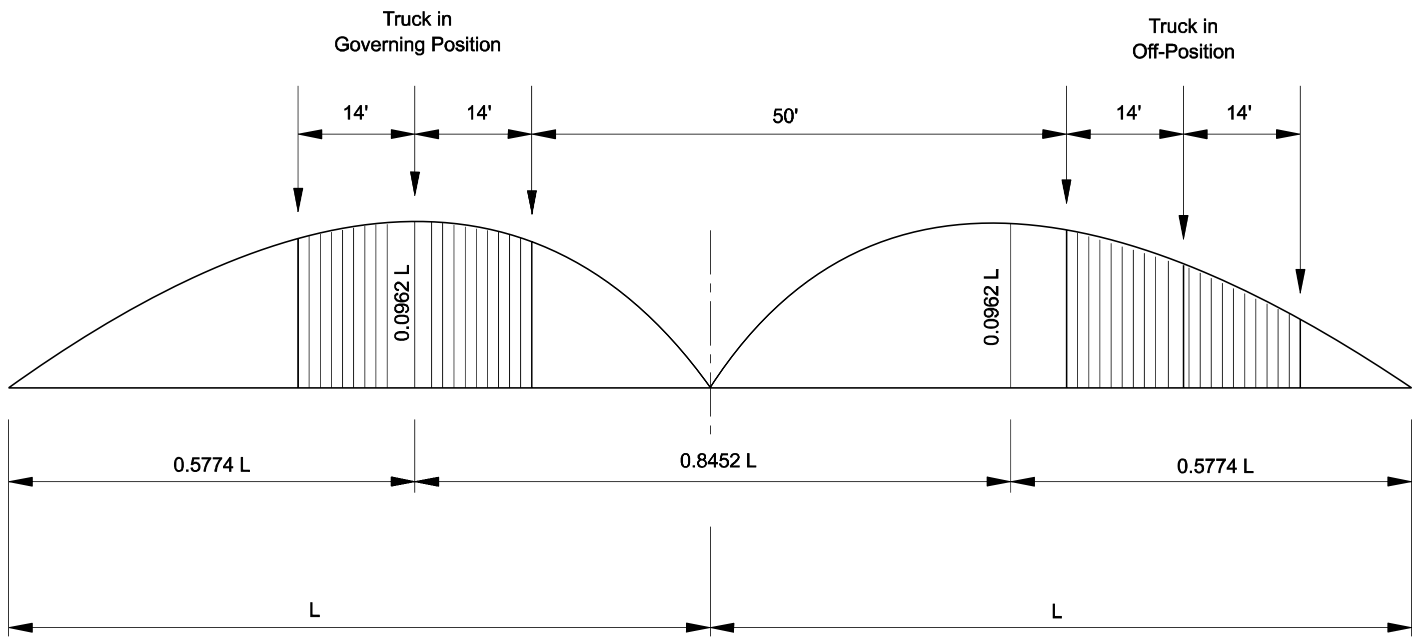
12.3.2.5.2 *Application of Horizontal Superstructure Forces to the Substructure*

The transfer of horizontal superstructure forces to the substructure depends on the type of superstructure to substructure connection. Centrifugal force (CE), braking force (BR) and wind on live load (WL) are all assumed to act horizontally at a distance of 6 ft above the roadway. Connections can be fixed, pinned or free for both moment and shear.



SPECIAL LOADING FOR NEGATIVE MOMENT AND INTERIOR REACTIONS OF CONTINUOUS SPANS

Figure 12.3-B



APPLICATION OF DESIGN VEHICULAR LIVE LOAD – LRFD ARTICLE 3.6.1.3

Figure 12.3-C

If the horizontal superstructure force is being applied to the substructure through a pinned connection, there is no moment transfer. The designer should apply the superstructure force to the substructure at the connection.

For a fixed or moment connection, apply the superstructure horizontal force with an additional moment to the substructure. The additional moment is equal to the horizontal force times the distance between the force's line of action and the point of application.

12.3.2.6 Wheel Load for Deck Design

Reference: LRFD Article 3.6.1.3.3

Bridge decks shall be designed to carry axles consisting of two 20-kip wheels with dynamic allowance, alone or in combination with the lane load as appropriate. This axle load is consistent with the HS-25 truck.

12.3.2.7 Permit Loads for Design (P Load)

NDOT has adopted one of the Caltrans "Standard Permit Design Vehicles" for the design of structures to provide a minimum permit-load capacity on all highway structures to account for vehicles that exceed the legal limits and that operate on highways and structures under special transportation permits. This load is commonly called the "P" load. Typically, all State-owned bridges are designed for the Strength II, Service I and Service II load combinations with the P load in all lanes. The application of the P load to non-State owned bridges is determined on a case-by-case basis.

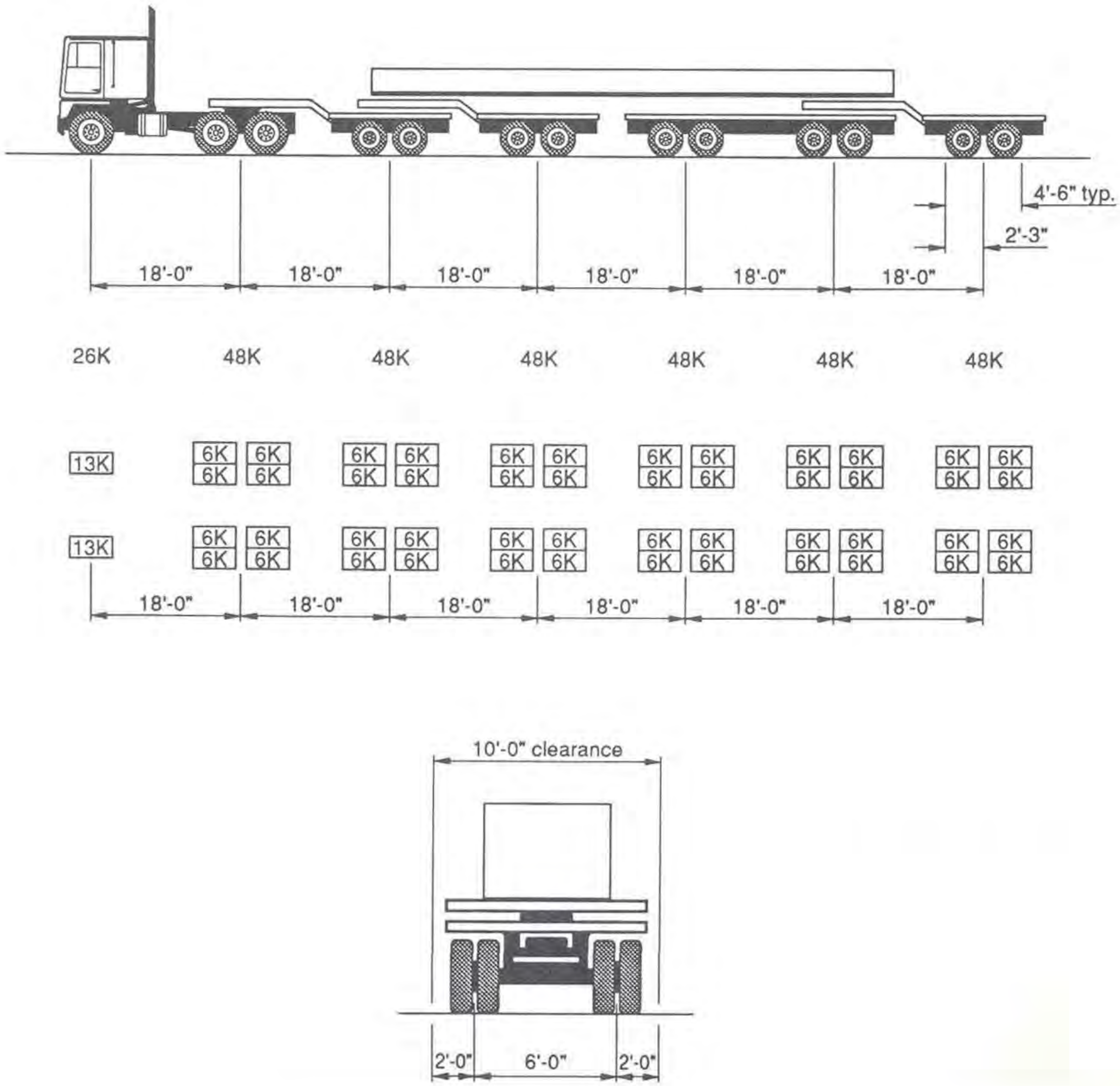
The P load, specifically the Caltrans P-13, is illustrated in Figure 12.3-D.

12.3.2.8 Fatigue Loads

Reference: LRFD Articles 3.6.1.4.1, 3.6.1.4.2

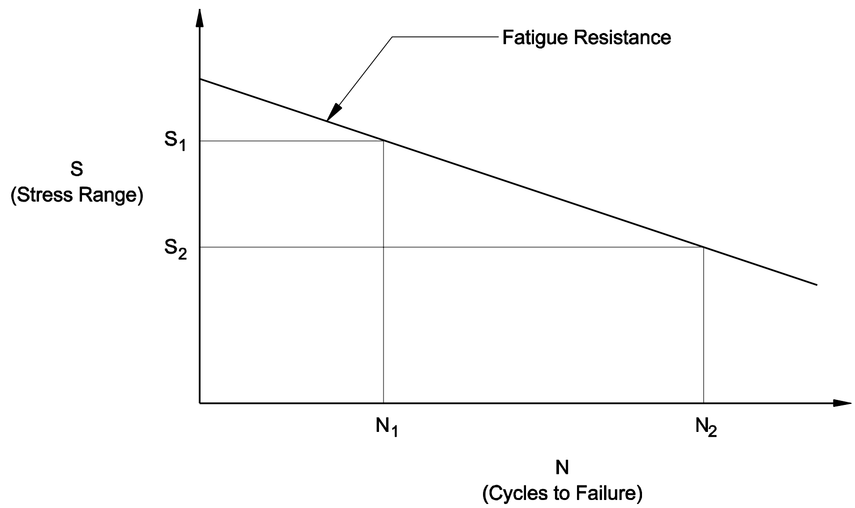
The *LRFD Specifications* defines the fatigue load for a particular bridge component by specifying both a magnitude and a frequency. The magnitude of the fatigue load consists of a single design truck per bridge with a load factor of 0.75 (i.e., the factored force effects are equivalent to those of an HS-15 truck). This single-factored design truck produces a considerable reduction in the stress range in comparison with the stress ranges of the AASHTO *Standard Specifications*. However, fatigue designs using the *LRFD Specifications* are virtually identical to those of the *Standard Specifications*. This equivalence is accomplished through an increase in the frequency from values on the order of two million cycles in the *Standard Specifications*, which represented "design" cycles, to frequencies on the order of tens and hundreds of millions of cycles, which represent actual cycles in the *LRFD Specifications*.

This change to more realistic stress ranges and cycles, illustrated in the S-N curve (a log-log plot of stress range versus cycle to failure) of Figure 12.3-E, increases the designer's understanding of the extremely long fatigue lives of steel bridges. In Figure 12.3-E, S_1 represents the controlling stress range for multiple lanes of strength-magnitude loading typically in accordance with the *Standard Specifications*, with N_1 being its corresponding number of design cycles. S_2 represents the controlling stress range for a single fatigue truck in accordance



PERMIT DESIGN LIVE LOADS
(For P-13 Vehicle)

Figure 12.3-D



COMPARISON OF THE FATIGUE LOADS OF THE *LRFD SPECIFICATIONS* AND *STANDARD SPECIFICATIONS*

Figure 12.3-E

with the *LRFD Specifications*, with N_2 being its corresponding number of actual cycles. The increase in the number of cycles compensates for the reduction in stress range, yet both cases fall on the resistance curve producing a similar fatigue design.

The bridge designer shall also apply P loads, also with a load factor of 0.75, to the fatigue design for structural steel. In lieu of better information, the average daily truck traffic in a single lane, $ADTT_{SL}$, for the P load shall be taken as 10 trucks per day.

12.3.2.9 Distribution of Live Load to Piers

Reference: LRFD Article 3.6.1.3.1

To promote uniformity of distribution of live load to piers and other substructure components, the following procedure is suggested unless a more exact distribution of loads is used:

1. Live-Load Distribution Factor. The live-load distribution factor for each girder shall be determined assuming that the deck is acting as a simple girder between interior girders and as a cantilever spanning from the first interior girder over the exterior girder.

2. Live Load on Design Lanes. Design lanes shall be placed on the bridge to produce the maximum force effect for the component under investigation. Separate loadings of the HL-93 live load or the P load shall be placed within an individual design lane to likewise produce the maximum effect. The bridge designer shall consider one, two, three or more design lanes in conjunction with the multiple presence factors of LRFD Table 3.6.1.1.2-1, as can be accommodated on the roadway width.

3. Reaction on Piers. For piers with drop caps, live loads are transmitted to the pier through the girder bearings, and the cap shall be designed using the shears determined from the girder line analysis. For integral caps, the designer may distribute the live load to the cap using a wheel line method, a girder and axle method, or a combination of the two. The wheel line method and the girder and axle method are described in Example 12.3-1. For both drop caps and integral caps, the designer shall analyze multiple lane positions to maximize load effects (e.g., side-by-side lanes to maximize negative cap bending at an interior pier support, lanes placed in every other cap span to maximize positive bending).

* * * * * * * * * *

Example 12.3-1 — Live Load Placement on Integral Bent Caps

Given:

- Two-span bridge, 145-ft and 160-ft spans, zero skew, box girder depth of 6′-6″
- Girder spacing = 9′-4″
- Column Spacing = 18′8″ (with zero skew, pier is normal to bridge centerline)
- From the superstructure analysis, the reaction at the center pier for a single HL-93 lane with both spans loaded was determined to be 200k
- HL-93 loading is depicted in this example. Treat permit loads in a similar fashion. Apply superstructure dead load to the integral cap at girder lines

a. Wheel Line Method (Simplified Approach)

Determine wheel line loads from HL-93 lane reaction:

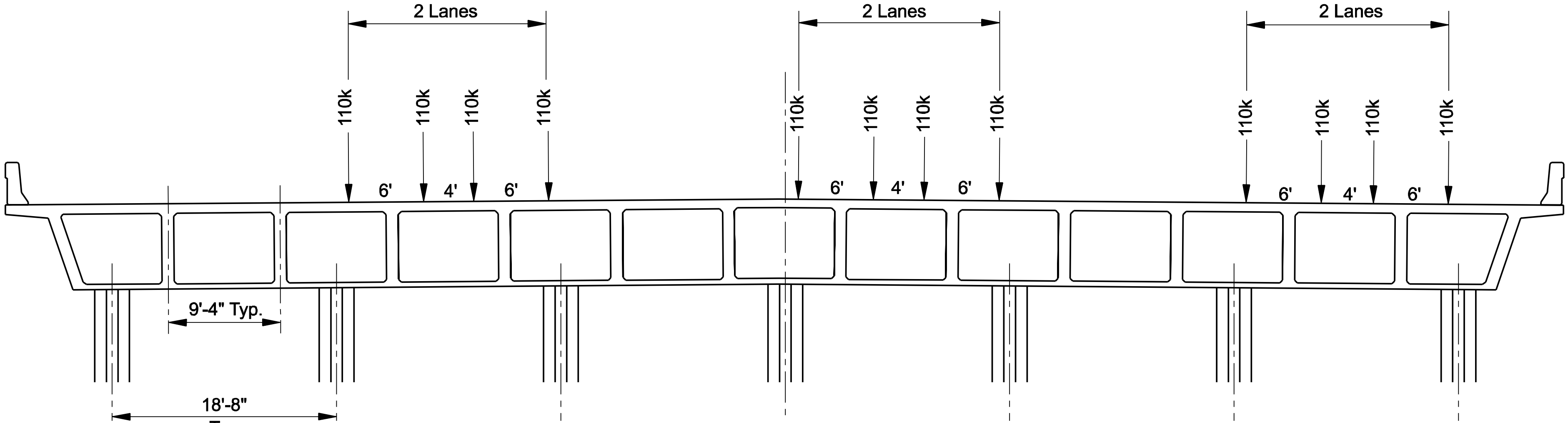
$$W_{HL\text{-}93} = \tfrac{1}{2}\text{ (lane reaction)}$$
$$= \tfrac{1}{2}\text{ (220k)} = 110\text{ k}$$

Wheel lines are applied 6 ft apart in a lane and 4 ft apart between lanes. As positioned in Figure 12.3-F(a), wheel lines are located to maximize positive bending in the cap beam. Analyze additional wheel line patterns to maximize load effects along the length of the cap beam (i.e., to develop moment and shear envelopes). A "train" of wheel lines running across the cap as a moving load is an easy approach to generating the envelopes.

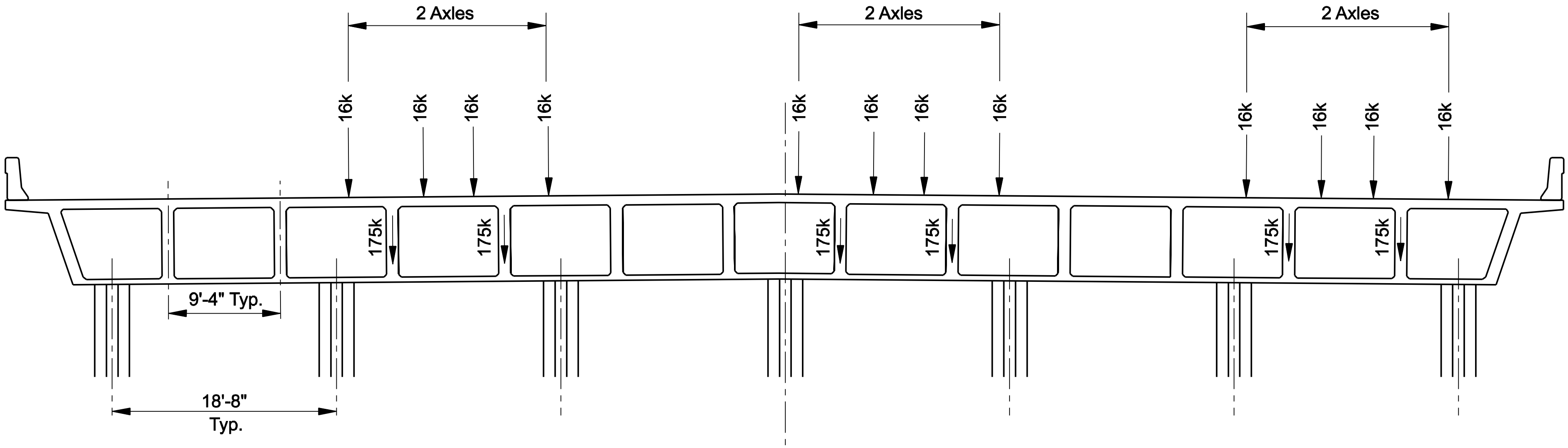
b. Girder and Axle Method (Refined Approach)

This refined approach recognizes that the majority of the lane load is transferred to the cap through the girder lines while a portion of the lane load could be positioned anywhere on the cap as an axle passes over. To represent this condition, the lane loading is divided between that which reaches the cap through the girders and that which is caused by the heaviest axle from the design vehicle applied directly to the cap. Determine the loads to girders assuming that the deck is simply supported between girder lines. From the full lane load, subtract the heaviest vehicle axle for direct application to the pier cap.

Figure 12.3-G represents girder and axle load placement to produce maximum positive bending in the cap. From statics:



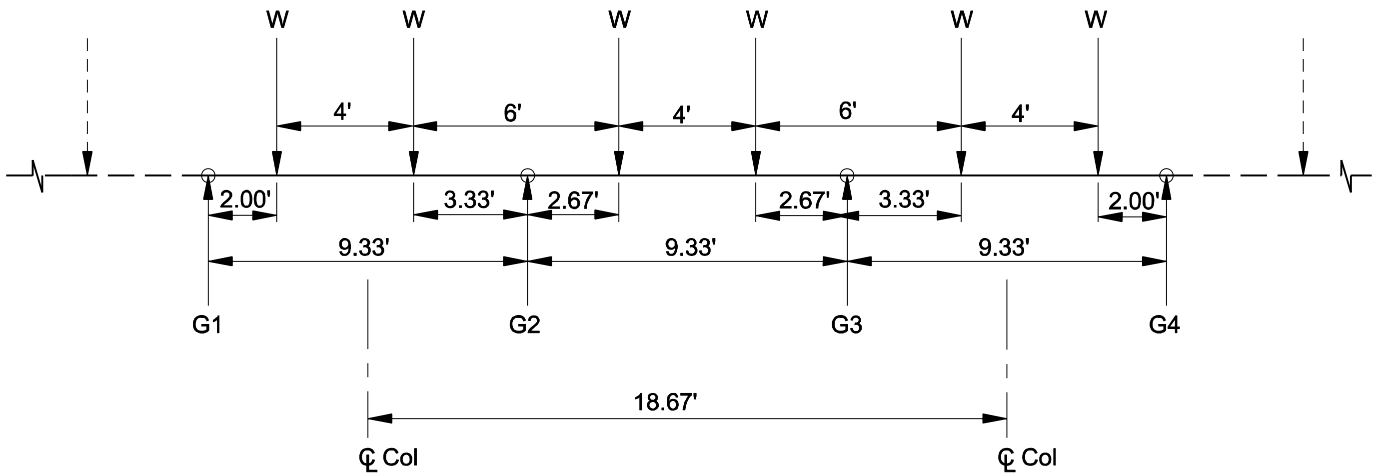
(a) WHEEL LINE METHOD



(b) GIRDER AND AXLE METHOD

LIVE LOAD APPLICATION
(Integral Cap)

Figure 12.3-F



PARTIAL CAP ELEVATION

Figure 12.3-G

$$G2 = G3 = W + 6W/9.33 + 2W/9.33$$
$$= 1.86W$$
$$= 0.93 \text{ lanes}$$

$$\text{HL-93 axle} = 32k$$
$$G2 = G3 = 0.93\,(220k - 32k)$$
$$= 175k$$

See Figure 12.3-F(b) for placement of loads across the integral cap.

* * * * * * * * * *

12.3.2.10 Sidewalk Loading

Reference: LRFD Article 3.6.1.6

Where sidewalks are present on the bridge, the bridge designer shall design for the dead load and pedestrian live load on the sidewalk; however, the full width of the bridge, including sidewalks, shall also be designed for the traffic live load assuming that traffic can mount the sidewalk.

Pedestrian and traffic loads will not be applied together. Sidewalks separated from traffic lanes by barrier rail shall also be designed for vehicular loads due to the potential for future widening.

12.3.2.11 Vehicular Collision Force (CT)

Reference: LRFD Article 3.6.5

Bridge abutments and piers over highways or railroads within a distance of:

- 30 ft to the edge of the roadway, or
- 50 ft to the centerline of the railroad track

shall be protected as specified in LRFD Article 3.6.5.1. If this is deemed to be impractical and with the approval of the Chief Structures Engineer, the abutment or pier shall be designed for a collision force of 400 kips acting in a horizontal plane in any direction at a distance of 4 ft above ground, as specified in LRFD Article 3.6.5.2.

12.3.3 <u>Friction Forces (FR)</u>

Reference: LRFD Article 3.13

LRFD Article 3.13 discusses the determination of horizontal friction forces from an expansion bearing sliding on its bearing plate on the supporting substructure component.

The bridge designer should adjust the frictional forces from sliding bearings to account for unintended additional friction forces due to the future degradation of the coefficient of friction of the sliding surfaces. Consider the horizontal force due to friction conservatively. Include friction forces where design loads would increase, but neglect friction forces where design loads would decrease.

12.3.4 Thermal Loads

Reference: LRFD Article 3.12.2

The bridge designer shall use Procedure A of LRFD Article 3.12.2.1 to determine the appropriate design thermal range. For Nevada-specific ranges of temperatures and procedures, see Section 19.1.

12.3.5 Earthquake Effects

Reference: LRFD Article 3.10

The seismic provisions of the *LRFD Specifications* shall be applied to bridge design in Nevada. The seismicity of Nevada varies greatly across the State. Nevada includes all four seismic zones specified in the *LRFD Specifications*. Earthquake force effects shall be determined in accordance with LRFD Article 3.10; however, the minimum seismic coefficients shown in Figure 12.3-H shall be applied unless otherwise approved by the Chief Structures Engineer.

Other Chapters in the *NDOT Structures Manual* present NDOT's seismic detailing practices. For example, Chapter 15 presents NDOT's seismic detailing practices for steel superstructures.

County	Peak Ground Acceleration (PGA) Coefficient	Short-Period Spectral Acceleration Coefficient (S_s)	Long-Period Spectral Acceleration Coefficient (S_l)
Carson City, Douglas, Esmerelda, Washoe	0.50	1.25	0.50
Lyon, Mineral, Storey	0.40	1.00	0.40
Churchill, Nye	0.35	0.80	0.30
Eureka, Lander, Lincoln, Pershing	0.25	0.60	0.20
Clark, Elko, Humboldt, White Pine	0.15	0.40	0.15

MINIMUM SEISMIC COEFFICIENTS BY COUNTY

Figure 12.3-H

12.3.6 Live-Load Surcharge (LS)

Reference: LRFD Article 3.11.6.2

Where reinforced concrete approach slabs are provided at bridge ends, live-load surcharge need not be considered on the abutment; however, the bridge designer shall consider the reactions on the abutment due to the axle loads on the approach slabs. Because approach slabs are required at all bridges in Nevada, live-load surcharge is not used for abutments.

Retaining walls that retain soil supporting a roadway must be able to resist the lateral pressure due to the live-load surcharge. See Section 23.1 for retaining walls.

Chapter 13

STRUCTURAL ANALYSIS AND EVALUATION

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Section **Page**

Chapter 13
STRUCTURAL ANALYSIS AND EVALUATION

Section 4 of the *LRFD Specifications* discusses the methods of structural analysis for the design and evaluation of bridge superstructures; analysis procedures for substructures are not specifically discussed in Section 4. Chapter 13 provides an elaboration on the provisions of LRFD Section 4 to discuss specific NDOT practices on structural analysis. Chapters 17 and 18 provide provisions on structural analysis procedures for foundations and substructures (e.g., seismic).

13.1 LIVE-LOAD DISTRIBUTION

13.1.1 General

Reference: LRFD Article 4.6.3.1

13.1.1.1 Definition

Live-load distribution, for application of the *NDOT Structures Manual*, refers to the determination of the maximum number of loaded lanes that an individual girder of the superstructure will be expected to carry.

13.1.1.2 Modeling Concrete Bridge Rails

The *LRFD Specifications* allows the structural contribution of any structurally continuous railing, barrier or median to be used to resist transient loads at the Service and Fatigue-and-Fracture limit states as a part of the cross section of the exterior girder. NDOT does not permit this allowance of structural contribution for new designs, but it may be considered in the evaluation or design for bridge rehabilitation if the contribution of the railing, barrier or median is significant.

13.1.2 Approximate Methods

Reference: LRFD Article 4.6.2

13.1.2.1 General

Traditionally, bridges have been analyzed using live-load distribution factors. These distribution factors result in a simple, approximate analysis of bridge superstructures. Live-load distribution factors uncouple the transverse and longitudinal distribution of force effects in the superstructure. Live-load force effects are assumed to be distributed transversely by proportioning the design lanes to individual girders through the application of distribution factors. The force effects are subsequently distributed longitudinally between the supports through the one-dimensional (1-D) structural analysis over the length of the girders.

Distribution factors reduce the necessity of modeling the entire bridge from a 2-D or 3-D analysis to a 1-D analysis of a girder. This 1-D, line-girder analysis is NDOT's preferred method of analysis, where suitable.

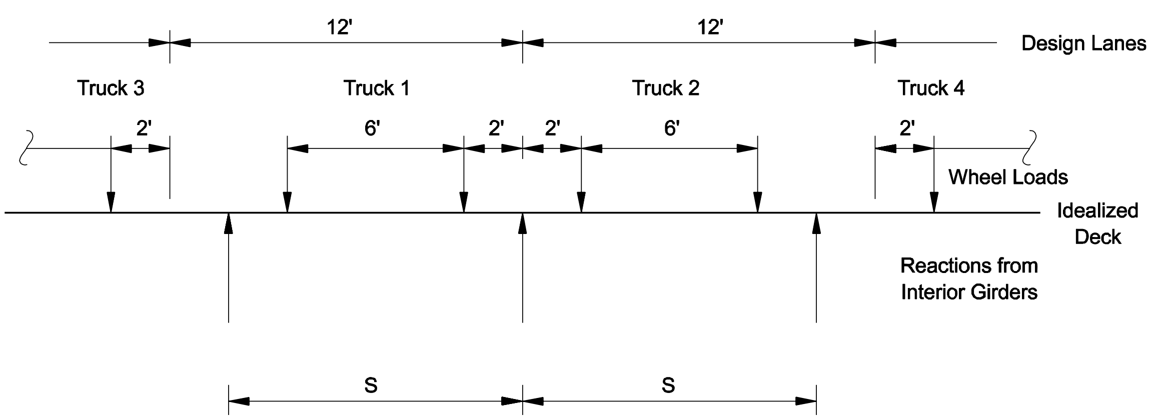
13.1.2.2 Simplified Analysis

Reference: LRFD Article 4.6.2.2

13.1.2.2.1 *General*

LRFD Article 4.6.2.2.2 presents several common bridge superstructure types, with empirically derived equations for live-load distribution factors for each type. These more sophisticated distribution-factor equations are analytically superior to the former AASHTO *Standard Specifications* "S-over" factors that have been used for bridges with spans and girder spacings far beyond those for which they were originally intended. Each distribution factor provides a number of design lanes to be applied to a girder to evaluate the girder for moment or shear. The factors account for interaction among loads from multiple lanes.

The distribution factors represent the placement of design lanes to generate the extreme effect in a specific girder as illustrated in Figure 13.1-A. The location of design lanes is not related to the location of striped lanes on the bridge. Summing all of the distribution factors for all girders produces a number of design lanes greater than the bridge can physically carry. This apparent overdesign occurs because each girder must be designed for the maximum load to which it could individually be subjected. Collectively, the individual load conditions producing the distribution factors cannot exist simultaneously on the bridge, yet each girder must be designed for its own worst case.



DESIGN LANE AND TRUCK PLACEMENT PRODUCING THE WORST CASE FOR AN INDIVIDUAL INTERIOR GIRDER

Figure 13.1-A

13.1.2.2.2 *Limitations*

The tables of distribution-factor equations given in LRFD Article 4.6.2.2 include a column entitled “Range of Applicability.” The *LRFD Specifications* specifies that bridges with parameters falling outside the indicated ranges be designed using the refined analysis requirements of LRFD Article 4.6.3. In fact, these ranges of applicability do not necessarily represent limits of usefulness of the distribution-factor equations, but the ranges represent the range over which bridges were examined to develop the coefficients and exponents of the empirical equations. Other State DOTs have conducted parametric studies to study the use of these equations beyond these ranges for typical bridges in their States. These studies have demonstrated that the factors may be used beyond the range of parameters that were specifically studied. However, it is NDOT policy to require the approval of the Chief Structures Engineer before using the distribution-factor equations beyond the “Range of Applicability” without the use of a refined analysis. See Section 13.2 for a discussion on refined analyses.

13.1.2.2.3 *Skewed Bridges*

Simplified analyses using the specified distribution factors of LRFD Article 4.6.2.2 can be used for skewed bridges provided that adjustments are made.

The bending moment in the longitudinal direction in a skewed bridge is generally smaller than the bending moment in a rectilinear bridge of the same span. NDOT currently does not take advantage of the reduction in load distribution factors for moment in longitudinal girders on skewed supports.

Torsional moments exist about the longitudinal axis in skewed bridges due to gravity loads (both dead and live load). These moments increase the reactions and shear forces at the obtuse corners compared to the acute corners.

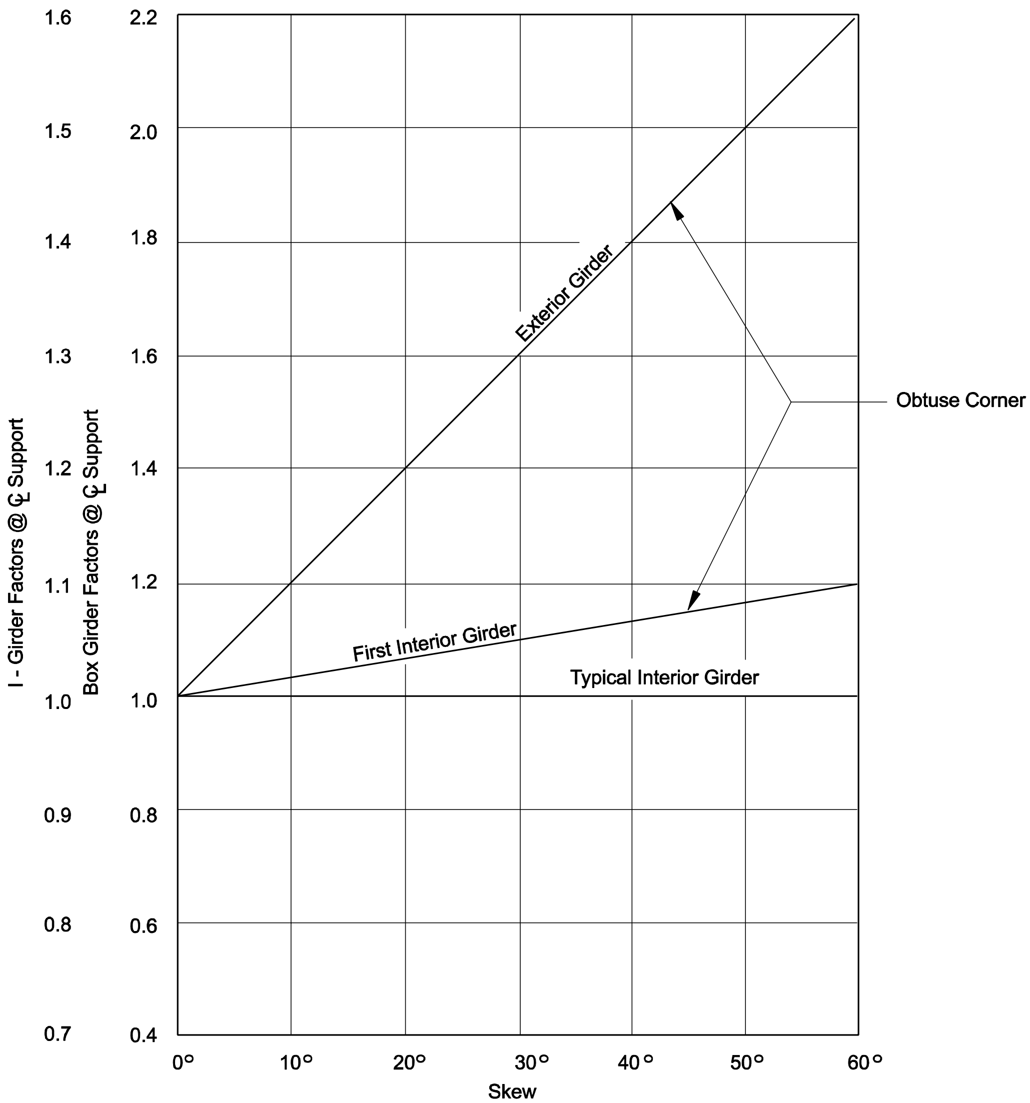
The potential exists for reactions to become very small or negative at acute corners, and should be avoided whenever possible during design. This can be achieved in post-tensioned bridges by the appropriate choice of the prestressing forces and the tendon profiles. The bridge designer should account for the higher reactions at the obtuse corners in the design of bearings and the supporting elements.

The skew correction factors for shear of LRFD Table 4.6.2.2.3c-1 shall be used to adjust the live load shears and reactions in skewed bridges. Figure 13.1-B shall be used to adjust the dead load shears and reactions. For shear design, the factors are assumed to vary linearly from the maximum value at the support to unity at midspan.

Curved bridges with supports skewed off of the radial direction by relatively large skew angles should be analyzed using a refined analysis; see Section 13.2.

13.1.3 <u>Example</u>

The following presents an example of the live-load distribution factors for the approximate analysis of a cast-in-place, post-tensioned box girder.



DEAD LOAD SHEAR AND RESISTANCE FACTORS FOR SKEWED BRIDGES

Figure 13.1-B

* * * * * * * * * *

Given: Cross Section (see Figure 13.1-C). The span length = 160 ft.

Problem: Determine the live-load distribution factors for moment and shear.

Solution: Reference: LRFD Article 4.6.2.2

Distribution Factors for Moment

Interior Girders: Reference: LRFD Table 4.6.2.2.2b-1

Two or more design lanes loaded:

$$g = \left(\frac{13}{N_c}\right)^{0.3}\left(\frac{S}{5.8}\right)\left(\frac{1}{L}\right)^{0.25}$$

Where:

N_c = number of cells in a concrete box girder = 4
S = spacing of girders or webs (ft) = 9.25 ft
L = span of girders (ft) = 160 ft

$$g = \left(\frac{13}{4}\right)^{0.3}\left(\frac{9.25}{5.8}\right)\left(\frac{1}{160}\right)^{0.25} = 0.64$$

Whole-Width Design: Reference: LRFD Table 4.6.2.2.1

$g_{\text{int erior girder}} = 0.64$
No. of girders = 5
g = (5)(0.64) = 3.20

Distribution Factors for Shear

Interior Girder: Reference: LRFD Table 4.6.2.2.3a-1

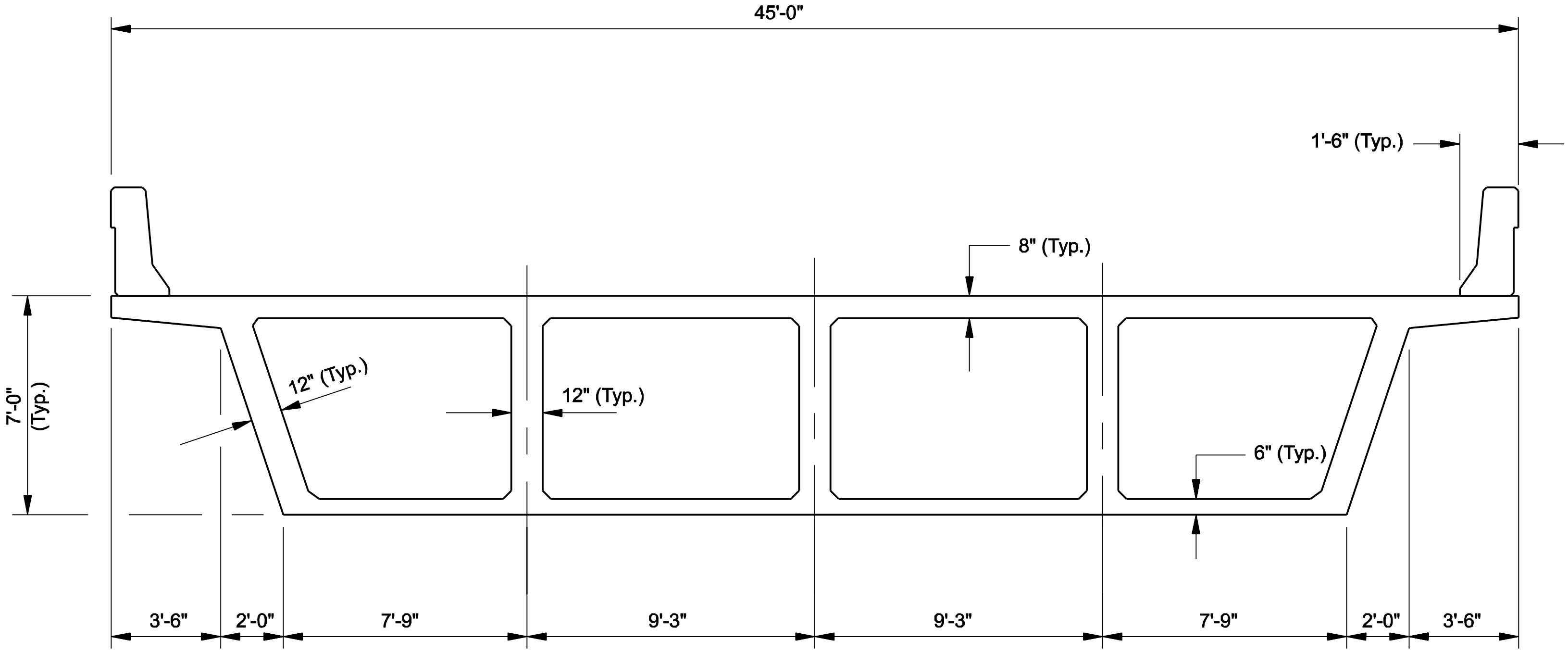
Two or more design lanes loaded:

$$g = \left(\frac{S}{7.3}\right)^{0.9}\left(\frac{d}{12.0L}\right)^{0.1}$$

Where:

d = depth of girder (in) = 84 in

$$g = \left(\frac{9.25}{7.3}\right)^{0.9}\left(\frac{84}{12.0 \times 160}\right)^{0.1} = 0.90$$



CROSS SECTION
(CIP, Post-Tensioned Box Girder)

Figure 13.1-C

Whole-Width Design: Reference: LRFD Table 4.6.2.2.1

$g_{interior\ girder} = 0.90$
No. of girders = 5
g = (5)(0.90) = 4.50

Summary

Force Effect	Interior Girder	Whole-Width Bridge
Moment	0.64	3.20
Shear	0.90	4.50

* * * * * * * * * *

13.2 REFINED ANALYSIS

Reference: LRFD Articles 4.6.2.2 and 4.6.3

13.2.1 General

Refined analyses include both 2-D and 3-D models (sometimes called grid and finite-element models, respectively). 2-D models are composed of elements lying in a single plane with the third dimension represented only through the stiffness properties of the elements. (The approximate methods of analysis of LRFD Article 4.6.2 employing distribution factors are essentially 1-D models where the only dimension used in the analysis is span length.) Typically, in a grid analysis, longitudinal elements represent the girders including any composite deck, and the transverse elements represent the deck. 3-D models are composed of elements in all three dimensions or of elements with three dimensions (such as brick elements). LRFD Article 4.6.3.3 provides general requirements for grid and finite-element analyses in terms of numbers of elements and aspect ratios.

13.2.2 2-D Analysis

13.2.2.1 Straight, Zero-Skew Bridges

A 2-D analysis is only warranted for a straight, zero-skew bridge with the complicated geometry of non-standard girder framing such as an urban interchange bridge or a bridge with varying width.

13.2.2.2 Horizontally Curved Bridges

The design of all superstructures must account for the effect of curvature where the components are constructed on horizontal curves. The magnitude of the effect of horizontal curvature is primarily a function of the curve radius, girder spacing, span length, diaphragm spacing and, to a lesser degree, web depth and flange proportions. The effect of curvature develops in two ways. First, the general tendency is for each girder to overturn, which has the effect of transferring both dead and live load from one girder to another transversely. The net result of this load transfer is that some girders carry more load and others carry less. The load transfer is carried through the diaphragms and the deck. The second effect of curvature is the concept of flange bending caused by torsion in curved components being almost totally resisted by horizontal shear in the flanges. The horizontal shear results in moments in the flanges. The stresses caused by these moments either add to or reduce the stresses from vertical bending. The torsion also causes warping of the girder webs.

Refined analysis methods, either grid or finite-element, shall be used for the analysis of horizontally curved bridges. LRFD Article 4.6.2.2.4 states that approximate analysis methods may be used for the analysis of curved bridges but then highlights the deficiencies of these analyses, specifically the V-load method for I-girders and the M/R method for boxes. Therefore, NDOT does not allow the use of these methods for curved bridges. The V-load method can be used for preliminary design purposes or as an order-of-magnitude checking tool.

13.2.2.3 Skewed Bridges

Reference: LRFD Article 4.6.2.2.3c

A 2-D refined analysis may be warranted for skewed bridges with an angle of skew greater than 30°.

13.2.3 <u>3-D Analysis</u>

A 3-D analysis, and its associated increase in costs, may not be warranted for the initial design of a bridge. For the analysis of complex structures or for the investigation of a problematic bridge (e.g., a bridge experiencing unexplained fatigue cracking), a 3-D analysis may be warranted.

13.3 SEISMIC ANALYSIS

13.3.1 General

The objective of seismic analysis is to assess the force and deformation demands and capacities on the structural system and its individual components. Equivalent static analysis (ESA) and linear elastic dynamic analysis (EDA) are the appropriate analytical tools for estimating the displacement demands for Ordinary Standard bridges. Inelastic static analysis (ISA) is the appropriate analytical tool to establish the displacement capacities for Ordinary Standard bridges.

13.3.2 Equivalent Static Analysis

ESA can be used to estimate displacement demands for structures where a more sophisticated dynamic analysis will not provide additional insight into behavior. ESA is best suited for structures or individual frames with well-balanced spans and uniformly distributed stiffnesses where the response can be captured by a predominant translational mode of vibration. The seismic load shall be assumed as an equivalent static horizontal force applied to individual frames. The total applied force shall be equal to the product of the acceleration response spectrum (ARS) and the tributary weight. The horizontal force shall be applied at the vertical center of mass of the superstructure and distributed horizontally in proportion to the mass distribution.

13.3.3 Elastic Dynamic Analysis

EDA shall be used to estimate the displacement demands for structures where ESA does not provide an adequate level of sophistication to estimate the dynamic behavior. A linear elastic multi-modal spectral analysis using the appropriate response spectrum shall be performed. The number of degrees of freedom and the number of modes considered in the analysis shall be sufficient to capture at least 90% mass participation in the longitudinal and transverse directions. A minimum of three elements per column and four elements per span shall be used in the linear elastic model.

EDA, based on design spectral accelerations, will likely produce stresses in some elements that exceed their elastic limit. The presence of such stresses indicates nonlinear behavior. The bridge designer should recognize that forces generated by linear elastic analysis could vary considerably from the actual force demands on the structure. Sources of nonlinear response that are not captured by EDA include the effects of the surrounding soil, yielding of structural components, opening and closing of expansion joints, and nonlinear restrainer and abutment behavior. EDA modal results shall be combined using the complete quadratic combination (CQC) method.

Typically, the entire bridge is modeled. For longer structures, the bridge designer should model a boundary frame (or abutment, where appropriate) at each end of the frame under investigation as a minimum.

13.3.4 Inelastic Static Analysis

ISA, commonly referred to as “push-over” analysis, shall be used to determine the reliable displacement capacities of a structure or frame as it reaches its limit of structural stability. ISA shall be performed using expected material properties of modeled members. ISA is an incremental linear analysis, which captures the overall nonlinear behavior of the elements, including soil effects, by pushing them laterally to initiate plastic action. Each increment pushes the frame laterally, through all possible stages, until the potential collapse mechanism is achieved. Because the analytical model accounts for the redistribution of internal actions as components respond inelastically, ISA is expected to provide a more realistic measure of behavior than can be obtained from elastic analysis procedures.

Structural system or global analysis is required when it is necessary to capture the response of the entire bridge system. Bridge systems with irregular geometry (especially horizontally curved bridges and skewed bridges, multiple transverse expansion joints, massive substructure components, and foundations supported by soft soil) can exhibit dynamic response characteristics that are not necessarily obvious and may not be captured in a separate subsystem analysis.

The two-dimensional plane frame "push-over" analysis of a bent or frame can be simplified to a column model (fixed-fixed or fixed-pinned), if it does not cause a significant loss in accuracy in estimating the displacement demands or the displacement capacities. The effect of overturning on the column axial load and associated member capacities must be considered in the simplified model. The simplified analytical technique for calculating frame capacity is only permitted if either Equations 13.3-1 and 13.3-2 or 13.3-3 and 13.3-4 below are satisfied. . Equations 13.3-1 and 13.3-3 apply to any two columns within a bent and any two bents within a frame. Equations 13.3-2 and 13.3-4 apply to *adjacent* columns within a bent and *adjacent* bents within a frame.

For constant-width frames:

$$\frac{k_i^e}{k_j^e} \geq 0.5 \qquad \text{(Equation 13.3-1)}$$

$$\frac{k_i^e}{k_j^e} \geq 0.75 \qquad \text{(Equation 13.3-2)}$$

For variable-width frames:

$$\frac{k_i^e / m_i}{k_j^e / m_j} \geq 0.5 \qquad \text{(Equation 13.3-3)}$$

$$\frac{k_i^e / m_i}{k_j^e / m_j} \geq 0.75 \qquad \text{(Equation 13.3-4)}$$

Where:

k_i^e = the smaller effective bent or column stiffness

k_j^e = the larger effective bent or column stiffness

m_i = tributary mass of column or bent i

m_j = tributary mass of column or bent j

In addition, the ratio of fundamental periods of vibration for adjacent frames in the longitudinal and transverse direction shall satisfy Equation 13.3-5:

$$\frac{T_i}{T_j} \geq 0.7 \qquad \text{(Equation 13.3-5)}$$

Where:

T_i = natural period of the less flexible frame
T_j = natural period of the more flexible frame

13.4 INELASTIC REDISTRIBUTION OF GRAVITATIONAL FORCE EFFECTS

Reference: LRFD Appendices A6 and B6

The *LRFD Specifications* presents simplified approaches to inelastic redistributions of moments in girder bridges. LRFD Article 5.7.3.5 provides a simple multiplier for negative-moment redistribution based upon ductility of a reinforced concrete section. LRFD Articles 6.10 and 6.11 present a simplified approach to inelastic redistribution of moments in steel girder bridges. The simplified approach allows the moment in a section to approach 1.3 times the moment at first yield, acknowledging the inherent ability of positive moments to inelastically redistribute to negative-moment steel sections regardless of the compactness of the negative-moment section.

NDOT prohibits the use of the LRFD Appendices to Section 6, which include more rigorous inelastic procedures for steel girders. LRFD Appendix A6 specifies a more rigorous and thus more extensive redistribution of positive moments to compact negative-moment sections. LRFD Appendix B6 gives similar provisions for the redistribution of moments at compact negative-moment sections.

Chapter 14

CONCRETE STRUCTURES

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Chapter 14
CONCRETE STRUCTURES

Section 5 of the *LRFD Bridge Design Specifications* presents unified design requirements for concrete, both reinforced and prestressed, in all structural elements. The American Concrete Institute (ACI) similarly uses unified provisions in ACI 318. This Chapter presents NDOT supplementary information specifically on the properties of concrete and reinforcing steel and the design of structural concrete members.

14.1 STRUCTURAL CONCRETE DESIGN

14.1.1 Member Design Models

Reference: LRFD Articles 5.6.3, 5.8.1, 5.8.3 and 5.13.2

Where it is reasonable to assume that a planar section remains planar after loading, the *LRFD Specifications* allows two approaches to the design for concrete members — the strut-and-tie model and the traditional sectional design model. Their basic application is as follows:

1. Sectional Design Model. The sectional design model is appropriate for the design of typical bridge girders, slabs and other regions of components where the assumptions of traditional girder theory are valid. This sectional design model assumes that the response at a particular section depends only on the calculated values of the sectional force effects such as moment, shear, axial load and torsion. This model does not consider the specific details of how the force effects were introduced into the member. LRFD Article 5.8.3 discusses the sectional design model. Subarticles 1 and 2 describe the applicable geometry required to use this technique to design for shear.

2. Strut-and-Tie Model. The strut-and-tie model should be used in regions near discontinuities (e.g., abrupt changes in cross section, openings, coped (dapped) ends, deep girders, corbels). See LRFD Articles 5.6.3 and 5.13.2.

The following Sections discuss each of these member design approaches.

14.1.2 Sectional Design Model

Reference: LRFD Article 5.8.3

14.1.2.1 Flexural Resistance

Reference: LRFD Article 5.7

The flexural resistance of a girder section is typically obtained using the rectangular stress distribution of LRFD Article 5.7.2.2. In lieu of using this simplified, yet accurate approach, a strain compatibility approach may be used as outlined in LRFD Article 5.7.3.2.5. The general equation for structural concrete flexural resistance of LRFD Article 5.7.3.2.1 is based upon the rectangular stress block.

14.1.2.2 Limits for Flexural Steel Reinforcement

14.1.2.2.1 *Maximum*

Reference: LRFD Articles 5.7.3.3.1 and 5.5.4.2.1

The current LRFD provisions eliminate the traditional maximum limit of reinforcement. Instead, a phi-factor varying linearly between the traditional values for flexure and compression members represented by LRFD Equations 5.5.4.2.1-1 or 5.5.4.2.1-2 is applied to differentiate between tension- and compression-controlled sections.

14.1.2.2.2 *Minimum*

Reference: LRFD Articles 5.7.3.3.2 and 5.4.2.6

The minimum flexural reinforcement of a component should provide flexural strength at least equal to the lesser of:

- 1.2 times the cracking moment of the concrete section, defined by LRFD Equation 5.7.3.3.2-1 and assuming that cracking occurs at the Modulus of Rupture, taken as $0.37\sqrt{f'_c}$ for normal-weight concrete; or

- 1.33 times the factored moment required by the governing load combination.

14.1.2.3 Distribution of Reinforcement

Reference: LRFD Article 5.7.3.4

In addition to the provisions of LRFD Article 5.7.3.4, the following will apply:

1. Negative Moments. For the distribution of negative moment tensile reinforcement continuous over a support, the effective tension flange width should be computed separately on each side of the support in accordance with LRFD Article 5.7.3.4. The larger of the two effective flange widths should be used for the uniform distribution of the reinforcement into both spans.

2. Girders. Within the negative moment regions of continuous cast-in-place structures, the top side face bar on each face of the girder web shall be #8 bar.

3. Integral Pier Caps. For integral pier caps, reinforcement shall be placed approximately 3 in below the construction joint between the deck and cap, or lower if necessary to clear prestressing ducts. This reinforcement shall be designed by taking M_u as 1.3 times the dead load negative moment of that portion of the cap and superstructure located beneath the construction joint and within 10 ft of each side face of the cap. Service load checks and shear design are not required for this condition. This reinforcement may be included in computing the flexural capacity of the cap only if a stress and strain compatibility analysis is made to determine the stress in the bars.

14.1.2.4 Crack Control Reinforcement

Reference: LRFD Article 5.7.3.4

Reinforcing bars in all reinforced concrete members in tension shall be distributed to control cracking in accordance with LRFD Article 5.7.3.4. When designing for crack control, the following values shall be used, unless a more severe condition is warranted:

- γ_e = 0.75 (Class 2 exposure condition) for footings and other components in contact with soil or brackish water, for decks, slabs, barrier rail, tops of abutment caps below expansion joints, and other components susceptible to deicing agent exposure; and

- γ_e = 1.00 (Class 1 exposure condition) for all other components.

Several smaller reinforcing bars at moderate spacing are more effective in controlling cracking than fewer larger bars.

14.1.2.5 Shear Resistance

Reference: LRFD Article 5.8.3

14.1.2.5.1 *Sectional Design Models*

Sectional design models are appropriate for flexural regions, regions away from reactions, applied loads and changes in cross section, where conventional methods for the strength of materials are applicable and strains are linear. The *LRFD Specifications* presents two alternative sectional shear design models for estimating the shear resistance of concrete members.

<u>General Procedure: Modified Compression Field Theory (MCFT)</u>

Reference: LRFD Article 5.8.3.4.2

The nominal shear resistance is taken as the lesser of:

$$V_n = V_c + V_s + V_p \qquad \text{(LRFD Eq. 5.8.3.3-1)}$$

$$V_n = 0.25 f'_c b_v d_v + V_p \qquad \text{(LRFD Eq. 5.8.3.3-2)}$$

For non-prestressed sections, $V_p = 0$.

LRFD Equation 5.8.3.3-2 represents an upper limit of V_n to ensure that the concrete in the web will not crush prior to yielding of the transverse reinforcement.

The nominal shear resistance provided by tension in the concrete is computed by:

$$V_c = 0.0316\beta\sqrt{f'_c}\, b_v d_v \qquad \text{(LRFD Eq. 5.8.3.3-3)}$$

The contribution of the web reinforcement is given by:

$$V_s = \frac{A_v f_y d_v \left(\cot\theta + \cot\alpha\right)\sin\alpha}{s} \qquad \text{(LRFD Eq. 5.8.3.3-4)}$$

where the angles, θ and α, represent the inclination of the diagonal compressive forces measured from the longitudinal axis and the angle of the web reinforcement relative to the longitudinal axis, respectively.

For the usual case where the web shear reinforcement is vertical (α = 90°), V_s simplifies to:

$$V_s = \frac{A_v f_y d_v \cot\theta}{s}$$

Both θ and β are functions of the longitudinal steel strain (ε_x) which, in turn, is a function of θ. Therefore, the design process is an iterative one. LRFD Article 5.8.3.4.2 provides the detailed methodology and the design tables. For sections containing at least the minimum amount of transverse reinforcement specified in LRFD Article 5.8.2.5, the values of β and θ should be taken from LRFD Table 5.8.3.4.2-1. For sections that do not meet the minimum transverse reinforcement requirements, LRFD Table 5.8.3.4.2-2 should be used to determine β and θ.

Sections meeting the requirements of LRFD Article 5.8.3.4.1 may be designed using a value of 2.0 for β and a value of 45° for θ. However, these traditional values of θ and β have proven seriously unconservative for large members not containing transverse reinforcement (footings, for example).

Transverse shear reinforcement shall be provided when:

$$V_u > 0.5\,\phi\left(V_c + V_p\right) \qquad \text{(LRFD Eq. 5.8.2.4-1)}$$

Where transverse reinforcement is required, the area of steel shall not be less than:

$$A_v = 0.0316\,\sqrt{f'_c}\,\frac{b_v s}{f_y} \qquad \text{(LRFD Eq. 5.8.2.5-1)}$$

For the usual case where the reaction introduces compression into the end of the member, the critical section for shear is taken as d_v, measured from the face of the support (see LRFD Article 5.8.3.2).

The sectional model requires a check of the adequacy of the longitudinal reinforcement in LRFD Article 5.8.3.5. This requirement acknowledges that shear causes tension in the longitudinal reinforcement. All steel on the flexural tension side of the member, prestressed and non-prestressed, may be used to satisfy this requirement.

<u>Simplified Procedure</u>

Reference: LRFD Article 5.8.3.4.3

The simplified procedure is similar to the traditional approach in the *Standard Specifications*. In this procedure, the lesser of two components, V_{ci} and V_{cw}, is used to quantify the concrete contribution to shear resistance. Although this procedure is not iterative, it can be more conservative than the MCFT approach.

14.1.2.5.2 *Shear Friction*

Reference: LRFD Article 5.8.4

The steel required to comply with the provisions of LRFD Article 5.8.4 shall be considered additive to the steel required from other analyses, except as provided for in LRFD Article 5.10.11.

14.1.3 Strut-and-Tie Model

Reference: LRFD Article 5.6.3

The strut-and-tie model is used to determine internal force effects in disturbed regions, regions near reactions, applied loads or changes in cross section, where the sectional models are not appropriate. Further, it is only applicable to the strength and Extreme-Event limit states because significant cracking must be present for the model to be valid.

This method of modeling concrete components originated around 1900, but it has only recently been incorporated into the AASHTO bridge design code. Members, when loaded, indicate the presence of definite stress fields that can individually be represented by tensile or compressive resultant forces as their vectorial sums. It has been observed that the "load paths" taken by these resultants form a truss-like pattern that is optimum for the given loading and that the resultants are in reasonable equilibrium, especially after cracking. The designer's objective is to conceive this optimum pattern (truss) in developing the strut-and-tie model. The closer the designer's assumption is to this optimum pattern (truss), the more efficient the use of materials. For relatively poorly conceived strut-and-tie models, the materials will be used less efficiently, yet the structure will be safe. The compressive concrete paths are the struts, and the reinforcing steel groups are the ties. The model does not involve shear or moment because the stresses are modeled as axial loads alone.

The application of the strut-and-tie model encompasses several simple steps:

1. The truss model must be envisioned that carries the applied loads to the reactions and, subsequently, the truss geometry established.

2. The struts are proportioned according to the provisions of LRFD Article 5.6.3.3, and the ties according to LRFD Article 5.6.3.4.

3. The nodal regions connecting the truss members are proportioned according to the provisions of LRFD Article 5.6.3.5, wherein concrete compression stresses are limited.

4. Finally, crack control reinforcement is provided according to LRFD Article 5.6.3.6 to control the significant cracking necessary to facilitate the strut-and-tie model.

The strut-and-tie model has significant application to bridge components such as pier caps, girder ends, post-tensioning anchorage zones, etc. A thorough presentation of the model can be found in:

- NCHRP 20-7, Task 217 *Verification and Implementation of Strut-and-Tie Model in LRFD Bridge Design Specifications*, November 2007;

- D. Mitchell, M. Collins, S. Bhidé and B. Rabbat, AASHTO "LRFD Strut-and-Tie Model Design Examples," EB231, Portland Cement Association (PCA);

- Chapter 8 of the *PCI Precast Prestressed Concrete Bridge Design Manual*; and

- J. Schlaich, et al, "Towards a Consistent Design of Structural Concrete," PCI Journal, Vol. 32, No. 3, 1987.

The *LRFD Specifications* provides adequately for design; even if the strut-and-tie model is not used for actual proportioning, the model provides a fast check to ensure the adequacy of the design, especially for the appropriate anchorage of the steel.

Cracking is associated with at least partial debonding and, thus, the bonding capacity of cracked concrete cannot be considered completely reliable. The *LRFD Specifications* generally requires that reinforcing steel should not be anchored in cracked zones of concrete. Improperly anchored reinforcing steel is an area that is commonly overlooked.

14.1.4 Fatigue

Reference: LRFD Articles 3.4.1, 3.6.1.4 and 5.5.3

The fatigue limit state is not normally a critical issue for concrete structures. Fatigue need not be considered for decks nor where the permanent stress f_{min} is compressive and exceeds twice the maximum tensile live load stress. Also, fatigue need not be considered for strands in fully prestressed concrete members.

Assuming r/h = 0.3, LRFD Equation 5.5.3.2-1 for mild reinforcement may be rearranged for easier interpretation:

$$f_f + 0.33\, f_{min} \leq 24 \text{ ksi}$$

14.1.5 Torsion

Reference: LRFD Article 5.8

Torsion is not normally a major consideration in most highway bridges. Where torsion effects are present, the member shall be designed in accordance with LRFD Articles 5.8.2 and 5.8.3.6. Situations that may require a torsion design include:

- cantilever brackets connected perpendicular to a concrete girder, especially if a diaphragm is not located opposite the bracket;
- concrete diaphragms used to make precast girders continuous for live load where the girders are spaced differently in adjacent spans; and
- abutment caps, if they are unsymmetrically loaded.

14.2 MATERIALS

14.2.1 Structural Concrete

Reference: LRFD Article 5.4.2.1

Figure 14.2-A presents NDOT criteria for the minimum compressive strength of concrete in structural elements.

Structural Element		Minimum 28-Day Compressive Strength (f'_c)
Bridge Decks, Approach Slabs and Barrier Rails	Clark County	4 ksi
	Rest of State	4.5 ksi
CIP Concrete Slabs	Clark County	4 ksi
	Rest of State	4.5 ksi
Prestressed Concrete (Post-Tensioned)		4 ksi*
Prestressed Concrete (Precast)		5 ksi*
Piers and Columns		4 ksi
Abutments		4 ksi
Wingwalls		4 ksi
Spread Footings		4 ksi
Drilled Shafts		4 ksi
Reinforced Concrete Boxes (non-standard)		4 ksi

* *The maximum strength for post-tensioned and precast, prestressed concrete shall not exceed 6.5 ksi and 7.5 ksi, respectively. Higher strengths shall not be used without the approval of the Chief Structures Engineer and a review by the NDOT Materials Division*.

COMPRESSIVE STRENGTH OF CONCRETE

Figure 14.2-A

14.2.2 Reinforcing Steel

Reference: LRFD Article 5.4.3.1

For general application, reinforcing steel shall conform to the requirements of ASTM A615, Grade 60. For seismic applications, reinforcing steel shall conform to the requirements of ASTM A706, Grade 60. The modulus of elasticity, E_s, is equal to 29,000 ksi.

Where reinforced concrete elements are designed to resist seismic forces beyond the elastic limit of the reinforcing steel, the bridge designer shall specify A706, Grade 60 reinforcing steel. ASTM A706 reinforcing steel is manufactured with controlled material properties. These properties include a maximum yield strength and a minimum ratio between the tensile and yield strengths. In addition, ASTM A706 reinforcing steel is manufactured with a controlled chemical composition making it more weldable. All welding of this reinforcing steel should be in accordance with AWS D1.4.

If A706 reinforcing steel is specified for elements in a bridge, it should be used for the entire bridge. This eliminates the need for separate inventories and increased inspection at the job site.

Reinforcing steel with a yield strength greater than 60 ksi may be used with the approval of the Chief Structures Engineer for minor structures (e.g., culverts, sound walls). However, the design must satisfy all limit states, including serviceability (i.e., cracking). Do not exceed a strength greater than 75 ksi as the basis for design.

14.2.3 Welded Wire Reinforcement

Welded Wire Reinforcement (WWR), also referred to as welded wire fabric, is an alternative to conventional concrete reinforcing steel for approved applications; see Section 14.3.2. WWR is prefabricated in a series of parallel wires welded with cross wires to form square or rectangular grids. All wires in one direction are the same diameter, but the wires in the transverse direction are commonly different. Each wire intersection is resistance welded.

The wires used in WWR can be either smooth or deformed. WWR generally used for MSE soil reinforcement and precast reinforced concrete box culverts conforms to AASHTO M55 (ASTM A185) "Steel Welded Wire Reinforcement, Plain, for Concrete." WWR used in structural applications, such as precast soundwall panels, conforms to AASHTO M221 (ASTM A497) "Steel Welded Wire Reinforcement, Deformed, for Concrete."

14.2.4 Prestressing Strand

Reference: LRFD Article 5.4.4.1

Prestressing strand shall be low-relaxation, 7-wire strand with a minimum tensile strength of f_{pu} = 270 ksi and a minimum yield strength of f_{py} = 243 ksi. The minimum modulus of elasticity, E_p, is equal to 28,500 ksi.

14.2.5 Prestressing Bars

Reference: LRFD Article 5.4.4.1

Prestressing bars shall be plain or deformed bars with a minimum tensile strength of f_{pu} = 150 ksi, with a yield strength of 127.5 ksi for plain bars and 120 ksi for deformed bars. The minimum modulus of elasticity, E_p, is equal to 30,000 ksi.

14.3 REINFORCEMENT

14.3.1 Reinforcing Steel

14.3.1.1 Bar Sizes

Reinforcing steel is referred to in the bridge plans and specifications by number, and they vary in size from #3 to #18 in US Customary units. Figure 14.3-A presents the sizes and properties of the bars used by NDOT.

Bar Size Designation		Nominal Dimensions		
US Customary	Metric	Weight (lbs/ft)	Diameter (in)	Area (in^2)
#3	#10	0.376	0.375	0.11
#4	#13	0.668	0.500	0.20
#5	#16	1.043	0.625	0.31
#6	#19	1.502	0.750	0.44
#7	#22	2.044	0.875	0.60
#8	#25	2.670	1.000	0.79
#9	#29	3.400	1.128	1.00
#10	#32	4.303	1.270	1.27
#11	#36	5.313	1.410	1.56
#14	#43	7.650	1.693	2.25
#18	#57	13.600	2.257	4.00

REINFORCING STEEL SIZES

Figure 14.3-A

14.3.1.2 Concrete Cover

Reference: LRFD Article 5.12.3

Figure 14.3-B presents NDOT criteria for minimum concrete cover for various applications. These are the minimums regardless of the w/c ratio. All clearances to reinforcing steel shall be shown in the bridge plans.

14.3.1.3 Spacing of Bars

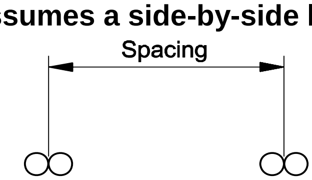
Reference: LRFD Article 5.10.3

Figure 14.3-C presents NDOT criteria for minimum spacing between reinforcement bars based on bar size and spliced vs unspliced. The accompanying sketch illustrates how to measure the spacing for spliced bars.

Structural Element or Condition		Minimum Concrete Cover
Concrete Deck Slabs	Top	2½″
	Bottom	1½″
Exposed to Deicing Salts (Barrier Rails, Approach Slabs, Top of Pier Caps, Abutment Seats)		2½″
Top of Pier Caps not Exposed to Deicing Salts		2″
Drilled Shafts (Diameter ≥ 3′)		6″
Drilled Shafts (Diameter < 3′)		4″
Stirrups and Ties		1½″
Reinforced Concrete Boxes	General	2″
	Against Ground	2½″
Formed Concrete Not Exposed to Ground		1½″
Formed Concrete Exposed to Ground		2″
Concrete Cast Against Ground		3″
Precast Members (Mild Reinforcement)		1½″

CONCRETE COVER

Figure 14.3-B

Bar Size	Minimum Spacing	
	Unspliced Bars	Spliced Bars (assumes a side-by-side lap) 
#3	N/A	N/A
#4	3″	3½″
#5	3½″	4″
#6	3½″	4″
#7	4″	5″
#8	4″	5″
#9	4″	5″
#10	4″	5½″
#11	4″	5½″
#14	4½″	6″
#18	5″	7″

MINIMUM SPACING OF BARS

Figure 14.3-C

Fit and clearance of reinforcing shall be carefully checked by calculations and large-scale drawings. Skews will tend to complicate problems with reinforcing fit. Tolerances normally allowed for cutting, bending and locating reinforcing should be considered. Refer to ACI 315 for allowed tolerances. Some of the common areas of interference are:

- anchor bolts in abutment caps;
- between slab reinforcing and reinforcing in monolithic abutments or piers;
- vertical column bars projecting through main reinforcing in pier caps;
- the areas near expansion devices;
- embedded plates for prestressed concrete girders;
- anchor plates for steel girders;
- at anchorages for a post-tensioned system; and
- between prestressing (pretensioned or post-tensioned) steel and reinforcing steel stirrups, ties, etc.

14.3.1.4 Fabrication Lengths

Use a maximum length of 60 ft for detailing reinforcing steel.

14.3.1.5 Lateral Confinement Reinforcement

14.3.1.5.1 *Columns*

Reference: LRFD Article 5.10.11.4

All lateral column reinforcement shall be detailed for Zones 3 and 4 requirements in LRFD Article 5.10.11.4. Lateral reinforcement for compression members shall consist of either spiral reinforcement, welded hoops or a combination of lateral ties and cross ties. Ties shall only be used when it is not practical to provide spiral or hoop reinforcement. Where longitudinal bars are required outside the spiral or hoop reinforcement, they shall have lateral support provided by bars spaced and hooked as required for cross ties. The hooked bars shall extend into the core of the spiral or hoop a full development length.

14.3.1.5.2 *Drilled Shafts*

The reinforcing steel cage for drilled shafts shall extend the full length of the pile.

The length of the plastic hinge confinement reinforcement shall be determined by appropriate analysis but shall not be less than the requirements of LRFD Article 5.13.4.6.3d.

The designer should maximize the size of longitudinal and transverse reinforcement to increase the openings between all reinforcement to allow concrete to pass through the cage during placement. The maximum spacing requirements of LRFD Article 5.13.6.3d shall be maintained.

14.3.1.5.3 *Headed Reinforcement*

Headed reinforcement can be considered as an alternative to lateral reinforcing steel when conflicts make the use of tie reinforcement impractical. Headed reinforcement consisting of friction welded or internally forged heads shall conform to ASTM A970/A970M.

14.3.1.6 Corrosion Protection

Epoxy-coated bars are not used in Clark County. For projects in the remainder of the State, the following presents NDOT policies for the use of epoxy-coated reinforcement bars:

- bridge decks (both layers),
- reinforcing that extends into bridge decks and/or terminates within 12 in of the top of the deck slab,
- cap shear and primary reinforcement of caps and abutments located under deck joints,
- integral cap shear and top reinforcement,
- bridge approach slabs,
- wingwalls (if not covered by an approach slab),
- barrier rails, and
- sidewalks.

14.3.1.7 Development of Reinforcement

Reinforcement must be developed on both sides of a point of maximum stress at any section of a reinforced concrete member. This requirement is specified in terms of a development length, l_d.

14.3.1.7.1 *Development Length in Tension*

Reference: LRFD Article 5.11.2

The development of bars in tension involves calculating the basic development length, l_{db}, which is modified by factors to reflect bar spacing, cover, enclosing transverse reinforcement, top bar effect, type of aggregate, and the ratio of required area to provide the area of reinforcement to be developed.

The development length, l_d (including all applicable modification factors), must not be less than 12 in.

14.3.1.7.2 *Development Length in Compression*

Columns shall not be considered compression members for development length computations. When designing column bars with hooks to develop the tension, ensure that the straight length is also adequate to develop the bar in compression because hooks are not considered effective in developing bars in compression. This practice ensures that columns in bending will have adequate development in both tension and compression.

14.3.1.7.3 *Standard End Hook Development Length in Tension*

Reference: LRFD Article 5.11.2.4

Standard hooks use a 90° and 180° bend to develop bars in tension where space limitations restrict the use of straight bars. End hooks on compression bars are not effective for development length purposes.

Refer to the figure in the commentary of LRFD Article C5.11.2.4.1 for hooked-bar details for the development of standard hooks. Use the same figure for both uncoated and coated bars, modified as appropriate by the factors noted in Section 14.3.1.7.1.

14.3.1.8 Splices

Reference: LRFD Article 5.11.5

14.3.1.8.1 *Types/Usage*

The following presents NDOT practices on the types of splices and their usage:

1. Lap Splices. NDOT uses conventional lap splices whenever practical. Use the Standard Minimum Lap Splice Lengths shown in Figure 14.3-D for all tension and compression lap splices unless a longer splice length is required by calculation. It is NDOT practice to use as a minimum a Class C splice for #4 through #8 bars and a Class B splice for #9 through #11 bars. Where feasible, stagger lap splices for main-member reinforcement such that no more than 50% are lapped in any one location. A minimum stagger of 2 ft between adjacent centerlines of splices is required for individual and bundled bars.

   If transverse reinforcing steel in a bridge deck is lapped near a longitudinal construction joint, the entire lap splice shall be placed on the side of the construction joint that will be poured last.

2. Mechanical Splices. (Reference: LRFD Articles 5.11.5.2.2, 5.11.5.3.2 and 5.11.5.5.2). A second method of splicing is by mechanical splices, which use proprietary splicing mechanisms. Mechanical splices are appropriate away from plastic hinges and where interference problems preclude the use of more conventional lap splices, and in staged construction. Even with mechanical splices, it is frequently necessary to stagger splices. The designer must check clearances. In addition, fatigue shall be considered. Mechanical splices shall develop 125% of the bar yield strength for reinforcing steel in non-yielding areas. Mechanical splices shall develop 160% of the bar yield strength for reinforcing steel in yielding areas not subject to plastic hinging.

Bar Size	Area (in^2)	Diameter (in)	Class	Uncoated (in)	Epoxy Coated (in)
#4	0.20	0.500	C	21	25
#5	0.31	0.625	C	26	31
#6	0.44	0.750	C	31	37
#7	0.60	0.875	C	39	46
#8	0.79	1.000	C	51	61
#9	1.00	1.128	B	49	59
#10	1.27	1.27	B	62	75
#11	1.56	1.41	B	77	92

Note: Lap splice lengths based on f'_c = 4 ksi, f_y = 60 ksi, non-top bars, uncoated bars spaced less than 6 in or with a clear cover of less than 3 in, epoxy coated bars spaced more than 6 bar diameters or with a clear cover of more than 3 bar diameters, and normal weight concrete.

STANDARD MINIMUM SPLICE LENGTHS FOR BARS IN TENSION AND COMPRESSION

Figure 14.3-D

3. Welded Splices. Splicing of reinforcing bars by welding, although allowed by the *LRFD Specifications*, is seldom used by NDOT and not encouraged primarily because of quality issues with field welding. However, shop-fabricated, butt-welded hoops can be used as confinement reinforcement for columns. Welding of reinforcing steel is not addressed by the AASHTO/ANSI/AWS *D1.5 Bridge Welding Code*, and the designer must reference the current *Structural Welding Code — Reinforcing Steel* of AWS (D1.4).

4. Full Mechanical/Welded Splices. See LRFD Article 5.11.5.3.2.

14.3.1.8.2 *Plastic Hinge Regions*

In columns and drilled shafts, there shall be no splices in the longitudinal reinforcing or splicing of spiral reinforcing within the plastic hinge regions. These regions shall be clearly identified in the contract documents.

14.3.1.9 Bundled Bars

Reference: LRFD Articles 5.11.2.3 and 5.11.5.2.1

NDOT allows the use of two-bundled or three-bundled bars; NDOT prohibits the use of four-bundled bars.

The development length of bars within a bundle shall be taken as that of an individual bar as specified in Section 14.3.1.7, increased by 20% for a three-bar bundle.

Lap splices of bundled bars shall be based upon development lengths as specified above. Entire bundles shall not be lap spliced at the same location. Individual bars within a bundle may be lap spliced, but the splices shall not overlap. Fit and clearance of reinforcing shall be carefully checked by calculations and large-scale drawings.

14.3.2 <u>Welded Wire Reinforcement (WWR)</u>

14.3.2.1 Design and Detailing

Concrete cover, development length and lap length for WWR shall meet the requirements of the *LRFD Specifications*. For corrosion protection, WWR can be provided with an epoxy coating conforming to ASTM A884, Class A.

Standard practice for detailing WWR can be found in the Wire Reinforcement Institute's *Manual of Standard Practice, Structural Welded Wire Reinforcement*. This document also provides commonly available wire sizes, spacing and available mat lengths.

14.3.2.2 Application and Limitations

Welded wire reinforcement may be considered as a substitute for AASHTO M31 (ASTM A615) reinforcing steel for minor structural applications, including:

- precast box culverts,
- precast MSE wall panels,
- precast sound barrier panels,
- drainage structures and appurtenances,
- channel linings, and
- slope paving.

Approval by the Chief Structures Engineer is required for all other applications. WWR shall not be used as a substitute for ASTM A706 reinforcing steel.

The following applies to the usage of WWR:

1. WWR can be provided as a direct replacement, with equivalent cross-sectional area, for the specified reinforcing steel. This is the preferred method of substitution.

2. WWR can be provided as a proposed redesign to take advantage of the higher yield strength of WWR. Supporting calculations and drawings sealed by a registered Nevada professional civil/structural engineer shall be submitted for approval. The design must satisfy all limit states, including serviceability (e.g., cracking). Yield strengths in excess of 75.0 ksi shall not be used for design purposes.

Material certifications must also be provided; the bridge designer should consult with the Materials Division to determine appropriate testing.

14.3.3 Prestressing Strands and Tendons

14.3.3.1 Pretensioned Girders

14.3.3.1.1 *Strand Size*

Common sizes of prestressing strand used in bridge construction are ½-in and 0.6-in diameter. The preferred diameter of the prestressing strands in pretensioned girders is ½ in.

14.3.3.1.2 *Strand Spacing*

The minimum spacing of strands shall not be less than 2 in center to center.

14.3.3.1.3 *Strand Profile*

It is acceptable to use either a straight or draped strand profile for precast members. NDOT prefers "draped" strand (i.e., deviated, harped, deflected) to "debonded," because of the greater shear capacity. However, a combination of debonded and draped strands may be used when necessary to satisfy design requirements. The advantages of straight trajectories include their simplicity of fabrication and greater safety. Debonded or draped strands are used to control stresses and camber. Debonded strands are easier to fabricate because a hold-down point is not required in the stressing bed. For debonded strands, see Section 14.5.4.4.

14.3.3.1.4 *Draped Strand*

The following applies to draped strands in precast, pretensioned girders:

- At ends of girders, maintain a minimum of 4 in between the top draped strands and any straight strands that are located directly above the draped strands.
- At each hold-down point, the vertical force should be limited to a maximum of 48 kips for all draped strands and 4 kips for each individual draped strand.
- The slope of the draped strands should not exceed 9°.
- Where practical, hold-down points should be located 5 ft on each side of the centerline of the girder (10 ft apart).

14.3.3.1.5 *Strand Patterns*

The designer must fully detail the strand pattern showing the total number of strands, layout and spacing, edge clearances, which strands will be draped and/or debonded, and the layout of all mild reinforcing steel. Frequently, precast, pretensioned girders of the same size and similar length in the same bridge or within bridges on the same project may be designed with a slightly different number of strands. In this case, the designer should consider using the same number and pattern of strands (including height of draping) for these girders to facilitate fabrication.

14.3.3.1.6 *Strand Splicing*

Splicing of prestressing strand is not allowed.

14.3.3.2 Post-Tensioned Members

14.3.3.2.1 *Strand Size*

The preferred diameter of the prestressing strand used for post-tensioning is 0.6 in. Although ½-in strand can also be used, the 0.6-in strand is more efficient.

14.3.3.2.2 *Tendons*

Tendons are proprietary systems that consist of an anchorage, duct, grout injection pipes and prestressing strand. Smaller tendons used in decks have ducts usually made from HDPE and contain up to four strands. Girder tendons use ducts made from galvanized metal and plastic and usually contain from 12 to 31 strands. The outside diameter of the ducts vary from 3 in to 5 in depending upon the number of strands and system supplier. Consult specific post-tensioning system brochures for the actual size of ducts. Two to five tendons are usually needed for each girder web to satisfy design requirements. The center of gravity specified at anchorages shall be consistent with tendon anchorage requirements (e.g., anticipated size(s) of bearing plates).

For cast-in-place, post-tensioned box girder bridges, tendons are internal to the girder webs. Segmental bridges can have tendons either external or internal to the girder web but not a mixture of the two.

14.4 CAST-IN-PLACE REINFORCED CONCRETE SLABS

14.4.1 <u>General</u>

Reference: LRFD Article 5.14.4

This Section presents information for the design of CIP concrete slabs that amplify or clarify the provisions in the *LRFD Specifications*. The Section also presents design information specific to NDOT practices.

14.4.1.1 Haunches

Haunches at interior supports of continuous bridges allow an increase in span by reducing the maximum positive moment and increasing the negative moment resistance. Parabolic haunches are preferred if aesthetics are important; otherwise, use straight haunches because they are easier to construct. The length of haunch on either side of an interior support should be 15% of the interior span. The depth of haunch at an interior support should be approximately 20% deeper than the structure depth at the location of maximum positive moment.

14.4.1.2 Minimum Reinforcement

Reference: LRFD Articles 5.7.3.3.2, 5.10.8 and 5.14.4.1

In both the longitudinal and transverse directions, at both the top and bottom of the slab, the minimum reinforcement should be determined according to the provisions of LRFD Articles 5.7.3.3.2 and 5.10.8. The first Article is based on the cracking flexural strength of a component, and the second Article reflects requirements for shrinkage and temperature. In CIP concrete slabs, the two Articles provide nearly identical amounts of minimum reinforcement in the majority of cases.

According to LRFD Article 5.14.4.1, the bottom transverse reinforcement (the above minimum provisions notwithstanding) may be determined either by two-dimensional analysis or as a percentage of the maximum longitudinal positive moment steel in accordance with LRFD Equation 5.14.4.1-1. The span length, L, in the equation should be taken as that measured from the centerline to centerline of the supports. For bridges with a skew greater than 60° and/or horizontally curved bridges, the analytical approach is recommended.

Section 14.4.5 presents a simplified approach for shrinkage and temperature steel requirements.

14.4.2 <u>Allowance for Dead-Load Deflection and Settlement</u>

Reference: LRFD Article 5.7.3.6.2

In setting falsework for CIP concrete slabs, an allowance shall be made for the deflection of the falsework, for any settlement of the falsework, for the dead-load deflection of the span, and for the long-term dead-load deflection of the span such that, on removal of the falsework, the top of the structure shall conform to the theoretical finished grade plus the allowance for long-term deflection.

14.4.3 Construction Joints

Longitudinal construction joints on CIP concrete slab bridges are undesirable. However, bridge width, staged construction, the method of placing concrete, rate of delivery of concrete, and the type of finishing machine used by the contractor dictate whether or not a CIP concrete slab bridge must be poured in one or more pours.

If the slab will be built in stages, show the entire lap splice for all transverse reinforcing steel on the side of the construction joint that will be poured last.

14.4.4 Longitudinal Edge Beam Design

Reference: LRFD Articles 5.14.4.1, 9.7.1.4, and 4.6.2.1.4

Edge beams must be provided along the edges of CIP concrete slabs. Structurally continuous barriers may only be considered effective for the Service limit states, not the Strength or Extreme-Event limit states. The edge beams shall consist of more heavily reinforced sections of the slab. The width of the edge beams may be taken to be the width of the equivalent strip as specified in LRFD Article 4.6.2.1.4b.

14.4.5 Shrinkage and Temperature Reinforcement

Reference: LRFD Articles 5.6.2 and 5.10.8

NDOT practice is that evaluating the redistribution of force effects as a result of shrinkage, temperature change, creep and movements of supports is not necessary when designing CIP concrete slabs. Figure 14.4-A provides the shrinkage and temperature reinforcement as a function of slab thickness.

14.4.6 Cap Design

NDOT typically uses an integral cap design in conjunction with CIP concrete slabs. However, NDOT allows the use of non-integral drop caps where aesthetics are not an issue.

Slab Thickness	Reinforcement (Top and Bottom)
< 18″	#4 @ 12″
18″ to 28″	#5 @ 12″
> 28″	Design per LRFD Article 5.10.8.2

SHRINKAGE AND TEMPERATURE REINFORCEMENT FOR CIP CONCRETE SLABS

Figure 14.4-A

14.4.7 Distribution of Concrete Barrier Railing Dead Load

The dead load of the barrier shall be assumed to be distributed uniformly over the entire bridge width.

14.4.8 Distribution of Live Load

Reference: LRFD Article 4.6.2.3

The following specifically applies to the distribution of live load to CIP concrete slabs:

1. For continuous slabs with variable span lengths, one equivalent strip width (E) shall be developed using the shortest span length for the value of L_1. This strip width should be used for moments throughout the entire length of the bridge.
2. The equivalent strip width (E) is the transverse width of slab over which an "axle" unit is distributed.
3. Different strip widths are specified for the CIP concrete slab itself and its edge beams in LRFD Articles 4.6.2.3 and 4.6.2.1.4, respectively.
4. In most cases, using LRFD Equation 4.6.2.3-3 for the reduction of moments in skewed slab-type bridges will not significantly change the reinforcing steel requirements. Therefore, for simplicity of design, NDOT does not require the use of the reduction factor "r."

14.4.9 Shear Resistance

Reference: LRFD Article 5.14.4.1

Single-span and continuous-span CIP concrete slabs, designed for moment in conformance with LRFD Article 4.6.2.3, may be considered satisfactory for shear.

14.4.10 Minimum Thickness of Slab

Reference: LRFD Article 2.5.2.6.3

When using the equations in LRFD Table 2.5.2.6.3-1, it is assumed that:

- S is the length of the longest span.
- The calculated thickness includes the ½-in sacrificial wearing surface.
- The thickness used may be greater than the value obtained from the LRFD Table.
- Minimum slab thickness is 18 in.

14.4.11 Development of Flexural Reinforcement

Reference: LRFD Article 5.11.1.2

LRFD Article 5.11.1.2 presents specifications for the portion of the longitudinal positive-moment reinforcement that must be extended beyond the centerline of support. Similarly, LRFD Article

5.11.1.2.3 addresses the location of the anchorage (embedment length) for the longitudinal negative-moment reinforcement.

14.4.12 Skews on CIP Concrete Slabs

Reference: LRFD Article 9.7.1.3

For skew angles up to 20°, the transverse reinforcement typically runs parallel to the skew, providing for equal bar lengths. For skews in excess of 20°, the transverse reinforcement should be placed perpendicular to the centerline of the bridge. This provision concerns the direction of principal tensile stresses, because these stresses develop in heavily skewed structures, and the provision is intended to prevent excessive cracking.

14.4.13 Abutment Type

For CIP concrete slabs, NDOT generally prefers the use of a seat type abutment.

14.5 PRESTRESSED CONCRETE SUPERSTRUCTURES

14.5.1 General

Reference: LRFD Article 5.2

The generic word “prestressing” relates to a method of construction in which a steel element is tensioned and anchored to the concrete. Upon release of the tensioning force, the concrete will largely be in residual compression and the steel in residual tension. There are two methods of applying the prestressing force, as discussed in the following Sections. A combination of these two methods may be used if approved by the Chief Structures Engineer.

14.5.1.1 “Pretensioning”

In the pretensioning method, tensioning of the steel strands is accomplished before the concrete is placed. When the concrete surrounding the steel strands attains a specified minimum strength, the strands are released thereby transmitting the prestressing force to the concrete by bond-and-wedge action at the girder ends. The initial prestress is immediately reduced due to the elastic shortening of the concrete. Further losses will occur over time due to shrinkage and creep of concrete and relaxation of prestressing steel.

The generic word “prestress” is often used to mean “pretensioning” as opposed to “post-tensioning.”

14.5.1.2 “Post-Tensioning”

In the post-tensioning method, tensioning of the steel is accomplished after the concrete has attained a specified minimum strength. The tendons, usually comprised of several strands, are loaded into ducts cast into the concrete. After stressing the tendons to the specified prestressing level, it is anchored to the concrete and the jacks are released. Several post-tensioning systems and anchorages are used in the United States; the best information may be directly obtained from the manufacturers. Post-tensioned concrete is also subject to losses from shrinkage and creep, although at a reduced magnitude because a significant portion of shrinkage usually occurs by the time of stressing, and the rate of creep decreases with the age at which the prestress is applied. After anchoring the tendons, the ducts are pressure filled with grout, which protects the tendons against corrosion and provides composite action by bonding the strand and the girder. Post-tensioning can be applied in phases to further increase the load-carrying capacity and better match the phased dead loads being applied to the girder.

14.5.1.3 “Partial Prestressing”

In this hybrid design, both mild reinforcement and prestressing are present in the tension zone of a girder. The idea of partial prestressing, at least to some extent, originated from a number of research projects that indicated fatigue problems in prestressed girders. Fatigue is a function of the stress range in the strands, which may be reduced by placing mild steel parallel to the strands in the cracked tensile zone to share live-load induced stresses. In these projects, based on a traditional model, however, the fatigue load was seriously overestimated. The fatigue load provided by the *LRFD Specifications* is a single design vehicle with reduced weight that is not likely to cause fatigue problems unless the girder is grossly under-reinforced.

The problems with partial prestressing include:

- Partially prestressed designs usually result in more tension in the girder at Service loads.
- Analytical tools are not readily available to predict accurately stress-strain levels of different steels in the cross section.

It is NDOT practice to not use partial prestressing. Although uncommon, NDOT occasionally uses partial prestressing in post-tensioning applications to counteract dead-load creep; e.g., partial prestressing may be used for the widening portion of conventionally reinforced bridges. This allows the widening to be tied into the existing bridge as soon as the post-tensioning is complete without waiting for an additional 60 to 90 days for creep deflection mitigation. However, in these applications, the widening must be designed to resist all forces with the mild reinforcement; the post-tensioning is supplemental and for the bridge dead load only.

14.5.2 <u>Basic Criteria</u>

This discussion applies to both pretensioned and post-tensioned concrete members.

14.5.2.1 Concrete Stress Limits

Reference: LRFD Article 5.9.4

Tensile stress limits for fully prestressed concrete members shall conform to the requirements for "Other Than Segmentally Constructed Bridges" in LRFD Article 5.9.4, except that the tensile stress at the Service limit state, after losses, shall be limited as follows: For components with bonded prestressing tendons or reinforcement, the tensile stress in the precompressed tensile zone shall be limited to:

$$0.095\sqrt{f'_c}$$

14.5.2.2 Concrete Strength at Release

Reference: LRFD Article 5.9.4.1

At release of the prestressing force, the minimum compressive concrete strength shall be the greater of 3.0 ksi or 60% of the specified 28-day strength. The specified concrete compressive strength at release should be rounded to the next highest 0.1 ksi.

14.5.2.3 Loss of Prestress

Reference: LRFD Article 5.9.5

Loss of prestress is defined as the difference between the initial stress in the strands and the effective prestress in the member. This definition of loss of prestress includes both instantaneous and time-dependent losses and gains.

The 2005 interim changes to the *LRFD Specifications* include many revisions to the process of calculating the loss of prestress.

14.5.2.4 Strand Transfer Length and Development Length

Reference: LRFD Article 5.11.4

The transfer length is the length of strand over which the prestress force is transferred to the concrete by bond and friction. The *LRFD Specifications* indicates that the transfer length may be assumed to be 60 strand diameters. The stress in the strand is assumed to vary linearly from zero at the end of the member, or the point where the strand is bonded if debonding is used, to the full effective prestress force at the end of the transfer length.

The development length is the length of strand required to develop the stress in the strand corresponding to the full flexural strength of the member; i.e., strand development length is the length required for the bond to develop the strand tension at nominal flexural resistance. The transfer length is included as part of the development length. LRFD Equation 5.11.4.2-1 is used to calculate the required development length (l_d). Prestressing strands shall be considered fully bonded beyond the critical section for development length. The development length for debonded strands shall be in accordance with LRFD Article 5.11.4.3.

14.5.2.5 Skew

Reference: LRFD Article 4.6.2.2

The behavior of skewed bridges is different from those of rectangular layout. The differences are largely proportional to skew angle. Although normal flexural effects due to live load tend to decrease as the skew angle increases, shear does not, and there is a considerable redistribution of shear forces in the end zone due to the development of negative moments therein. For skew angles less than 20°, it is considered satisfactory to ignore the effects of skew and to analyze the bridge as a straight bridge.

LRFD Articles 4.6.2.2.2e and 4.6.2.2.3c provide tabulated assistance to roughly estimate the live-load effects from skew. The factors shown in these tables can be applied to both simple span and continuous span skewed bridges. The correction factors for shear theoretically only apply to support shears of the exterior girder at the obtuse corner. In practice, the end shears of all girders in a multi-girder bridge are conservatively modified by the skew correction factor. Shear in portions of the girder away from the end supports do not need to be corrected for skew effects.

To obtain a better assessment of skewed behavior and to use potential benefits in reduced live-load moments, more sophisticated methods of analysis are used. The refined methods most often used to study the behavior of bridges are the grillage analysis and the finite element method. See Section 13.2 for more discussion. The finite element analysis requires the fewest simplifying assumptions in accounting for the greatest number of variables that govern the structural response of the bridge. However, input preparation time and derivation of overall forces for the composite girder are usually quite tedious. On the other hand, data preparation for the grillage method is simpler, and the integration of stresses is not needed.

14.5.3 Cast-in-Place, Post-Tensioned Box Girders

Reference: LRFD Articles 5.4.5 and 5.4.6

14.5.3.1 Ducts

In post-tensioned construction, ducts are cast into the concrete to permit placement and stressing of the tendons. Girder ducts are typically galvanized corrugated steel (semi-rigid). Ducts in top slabs are typically high-density polyethylene. For external tendons on segmental bridges, NDOT typically uses smooth polyethylene. The contract documents shall indicate the type of duct material to be used.

The wall thickness shall be no less than 28 gage. Prebending of ducts will be required for bend radii less than 30 ft and should be specified in the contract documents. Radii that require prebending should be avoided whenever possible. The minimum bend radius of ducts shall not be less than 20 ft, except in anchorage zones where 12 ft will be permitted. The bending radius of polyethylene or polypropylene ducts shall not be less than 30 ft.

If the bridge is constructed by post-tensioning precast components together longitudinally and/or transversely by use of a cast-in-place concrete joint, then the end of the duct should be extended beyond the concrete interface by not less than 3 in and not more than 6 in to facilitate joining the ducts. If necessary, the extension could be in a local blockout at the concrete interface. Joints between sections of ducts shall be positive metallic connections, which do not result in angle changes at the joints. Waterproof tape shall be used at all connections.

For multiple-strand tendons, the outside diameter of the duct shall be no more than 40% of the least gross concrete thickness at the location of the duct. The majority of bridges use a 12-in web, which limits the outside duct diameter to 4.8 in. Larger ducts may be required for shallow bridges with high P-jack forces (more than 1200 kips per girder). A wider web of 14 in may be used to ensure that the post-tensioning system fits. During design, the bridge designer must lay out an acceptable duct arrangement that matches the post-tensioning center of gravity to determine if a wider web is needed. Ducts preferably should not extend into either the top or bottom slabs. The internal free area of the duct shall be at least 2.5 times the net area of the prestressing steel. See LRFD Article 5.4.6.2.

Section 503 of the *NDOT Standard Specifications* discusses ducts for post-tensioned construction.

14.5.3.2 Grouting

Upon completion of post-tensioning, the ducts must be grouted. The strength of the grout should be comparable to that of the girder concrete but is not specified due to the high strengths that typically result from tendon grouts.

NDOT requires pre-approved bagged grout for tendon grouting. Multiple injection and bleed ports are required at the ends of the tendons and at all low and high points. Flushing of tendons due to blockage is discouraged but not disallowed using vacuum grouting as a consideration for repairs. Drilling into a percentage of the tendons at the anchorage to inspect for voids is a requirement of the *NDOT Standard Specifications*. If any voids are found, all tendons are inspected.

14.5.3.3 Tendon Profile

The geometry of a typical tendon profile is predominantly composed of second-degree parabola curved segments. The tendons are essentially straight segments near the anchorages. The tendon group center of gravity and the bridge's neutral axis should coincide at the following locations — at the centerlines of abutments, hinges and points of dead-load contraflexure.

Show offset dimensions to post-tensioning duct profiles from fixed surfaces or clearly defined reference lines. In regions of tight reverse curvature of short sections of tendons, offsets shall be shown at sufficiently frequent intervals to clearly define the reverse curve.

Curved ducts that run parallel to each other, ducts in curved girders, ducts in chorded girders where angle changes occur between segments, or ducts placed around a void or re-entrant corner shall be sufficiently encased in concrete and reinforced as necessary to avoid radial failure (pull-out into the other duct or void).

14.5.3.4 Anchorages

There are several types of commercially available anchorages. These anchorages normally consist of a steel block with holes in which the strands are individually anchored by wedges. In the vicinity of the anchor block (or coupler), the strands are fanned out to accommodate the anchorage hardware. The fanned out portion of the tendon is housed in a transition shield, often called a trumpet, which could be either steel or polyethylene, regardless of the duct material. Trumpets must have a smooth, tangential transition to the ducts.

If the distance between anchorages exceeds 300 ft, jacking at both ends should be considered. One-end or two-end stressing will be determined by design and specified in the contract documents.

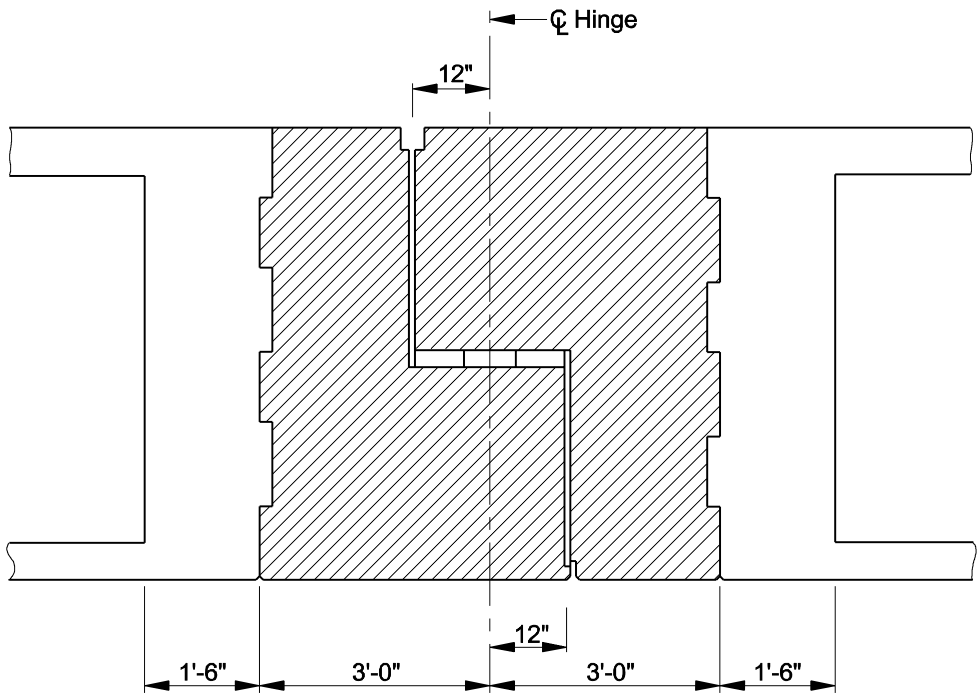
Values of the wobble and curvature friction coefficients and the anchor set loss assumed for the design shall be shown in the contract documents.

14.5.3.5 Hinges

For cast-in-place, post-tensioned, concrete superstructures, an in-span hinge is a complicated element that requires special consideration related to its design, detailing and construction sequencing. Intermediate expansion joints are often introduced into the superstructure of longer bridges, thereby dividing the structure into shorter frames with the intention of reducing thermal, creep and shrinkage forces in outlying supports. It is preferable to locate expansion joints atop intermediate piers, presuming there is adequate vertical clearance to accommodate a drop cap and bearing seat. Where a drop cap is not feasible nor aesthetically desirable, it may be necessary to introduce an in-span hinge.

Figure 14.5-A illustrates a longitudinal cross section of a typical in-span hinge for a cast-in-place, post-tensioned concrete box girder. As noted, the cross hatched area indicates the portion of the hinge concrete that is cast after the supporting (short cantilever) and supported spans have been post-tensioned. The supporting and supported sides of the hinge shall be designed as corbel elements and confirmed with a strut-and-tie approach to establish an adequate load path through the hinge.

The effects of post-tensioning in both the supporting and supported spans shall be considered in the design and detailing. For example, when the supported span is tensioned prior to hinge casting, the dead load will be redistributed from interior falsework supports to the temporary bent beneath the hinge end of the span. The contract documents should include the dead load reaction at the hinge for the contractor's use in the design of this temporary support.

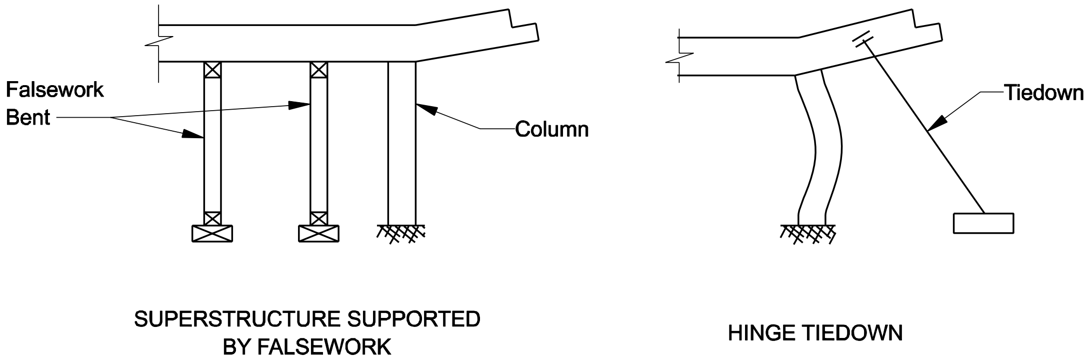


LONGITUDINAL CROSS SECTION OF IN-SPAN HINGE

Figure 14.5-A

After the supporting span is tensioned but before the load from the supported span has been transferred through the hinge, there is a tendency for the unloaded short cantilever to deflect upward. This phenomenon is commonly termed “hinge curl” and will increase over time with concrete creep and shrinkage. Unless properly accounted for, hinge curl can negatively impact the final deck profile and may result in the need for extensive grinding to correct the problem.

The designer is reminded that there is a variable period of time (usually between 30 and 180 days, occasionally much longer) in which the short cantilever remains unloaded after it has been stressed. The period of time and, therefore, the extent of hinge curl are not predictable until the contractor’s schedule is known. Furthermore, experience indicates that the hinge does not always deflect downward at load transfer as much as it had previously deflected upward under the influence of the prestressing force. The procedure presented below to address hinge curl assumes that the falsework in the adjacent back span will remain in place until load transfer occurs at the hinge. The back span falsework will, to some extent, assist in resisting the column top rotation that can contribute further to the upward movement of the short cantilever (see Figure14.5-B). Where it is necessary to remove back span falsework prior to hinge load transfer, the procedure identified below is not sufficient, and the designer should consider tying down the short cantilever or preloading it with temporary weights placed on the deck.



HINGE CURL COUNTERMEASURE

Figure 14.5-B

Notations and nomenclature:

e	=	eccentricity of prestressing at hinge of short cantilever
E	=	concrete modulus of elasticity
I	=	moment of inertia of box girder section
L	=	length of short cantilever
P_h	=	horizontal component of Pjack at anchorage of short cantilever
P_v	=	vertical component of Pjack as anchorage of short cantilever
T	=	transfer load at short cantilever
w	=	unit dead load of short cantilever

Deflection of short cantilever:

$$\Delta_{curl} = -(P_vL^3)/(3EI) \;\; +/- \;\; (P_heL^2)/2EI \; + \; wL^4/8EI \; + \; PL/3EI$$

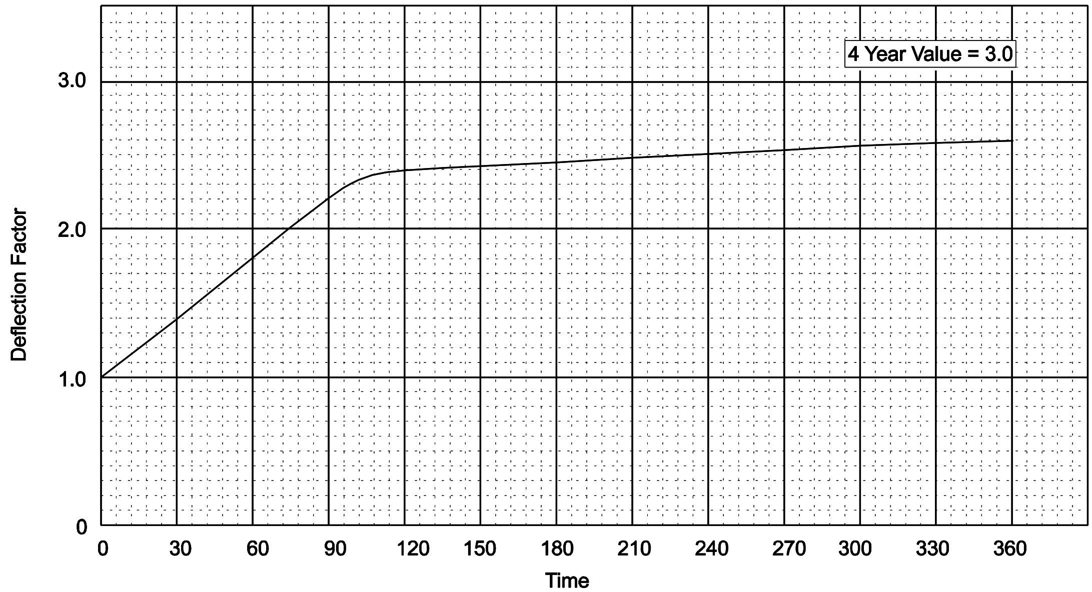
(Positive indicates upward deflection)

Deflection components:

$(P_vL^3)/(3EI)$	=	prestressing vertical component
$(P_heL^2)/2EI$	=	prestressing horizontal component
$wL^4/8EI$	=	distributed dead load of superstructure without barrier, future overlay, etc.
$PL/3EI$	=	concentrated load from hinge diaphragm or other miscellaneous load

Downward deflection of short cantilever:

$$\Delta_{reaction} = TL^3/3EI$$



TIME vs. DEFLECTION

Figure 14.5-C

Modify deflections for long-term effects using the Time vs. Deflection chart shown in Figure 14.5-C and as follows:

1. **Adjustment "A"**. Profile adjustment required at the long cantilever for transfer dead load less prestress uplift after load transfer (may be positive or negative value):

   30-day value = $2.60 \times \Delta_{reaction} - 1.60 \times \Delta_{curl}$
   60-day value = $2.20 \times \Delta_{reaction} - 1.20 \times \Delta_{curl}$
   90-day value = $1.80 \times \Delta_{reaction} - 0.80 \times \Delta_{curl}$
   120-day value = $1.60 \times \Delta_{reaction} - 0.60 \times \Delta_{curl}$
   180-day value = $1.55 \times \Delta_{reaction} - 0.55 \times \Delta_{curl}$
   240-day value = $1.50 \times \Delta_{reaction} - 0.50 \times \Delta_{curl}$
   360-day value = $1.40 \times \Delta_{reaction} - 0.40 \times \Delta_{curl}$
   720-day value = $1.25 \times \Delta_{reaction} - 0.25 \times \Delta_{curl}$

2. **Adjustment "B"**. Profile adjustment required for short cantilever (may be positive or negative value):

   30-day value = $2.60 \times \Delta_{reaction} - 3.00 \times \Delta_{curl}$
   60-day value = $2.20 \times \Delta_{reaction} - 3.00 \times \Delta_{curl}$
   90-day value = $1.80 \times \Delta_{reaction} - 3.00 \times \Delta_{curl}$
   120-day value = $1.60 \times \Delta_{reaction} - 3.00 \times \Delta_{curl}$
   180-day value = $1.55 \times \Delta_{reaction} - 3.00 \times \Delta_{curl}$
   240-day value = $1.50 \times \Delta_{reaction} - 3.00 \times \Delta_{curl}$
   360-day value = $1.40 \times \Delta_{reaction} - 3.00 \times \Delta_{curl}$
   720-day value = $1.25 \times \Delta_{reaction} - 3.00 \times \Delta_{curl}$

The predicted deflections shall be incorporated into the camber diagram that is included in the contract documents. It is assumed that the long-term effect of creep and shrinkage will result in an ultimate deflection three times greater than the theoretical immediate deflection, and that this will occur over a four-year period. Because the transfer of load from the supported span will occur at an unknown time after prestressing the supporting span, a camber diagram with time-dependent tabulated values shall be shown in the contract documents to account for schedule uncertainty.

Figures 14.5-D and 14.5-E illustrate the camber diagram to be drawn for a hinged span. The normal camber diagram is shown along with an enlarged camber curve for the hinged span. Values of camber are calculated and shown at the hinge. Adjustment "A" is calculated for the position of the supported span hinge side, and Adjustment "B" is calculated for the position of the supporting span hinge side. Point 1 in Figure 14.5-F is the theoretical camber if load transfer could be immediate from supported to supporting span (short cantilever). Point 2 is the adjustment (up or down) to the theoretical camber for the supported span, which is dependent on the time of transfer. Point 3 is the adjustment (up or down) to the theoretical camber for the supporting span (short cantilever).

14.5.3.6 Flexural Resistance

Reference: LRFD Article 5.7.3.2

Flexural resistance for CIP P/T concrete box girders shall be determined using the combined effects of bonded prestressing and mild reinforcing steel in accordance with LRFD Article 5.7.3.2.

14.5.3.7 Shear Resistance

14.5.3.7.1 *Strength Limit State*

Reference: LRFD Article 5.8.3

The shear resistance of CIP, PT boxes shall be determined using the modified compression field theory (MCFT) sectional model of LRFD Article 5.8.3.4.2.

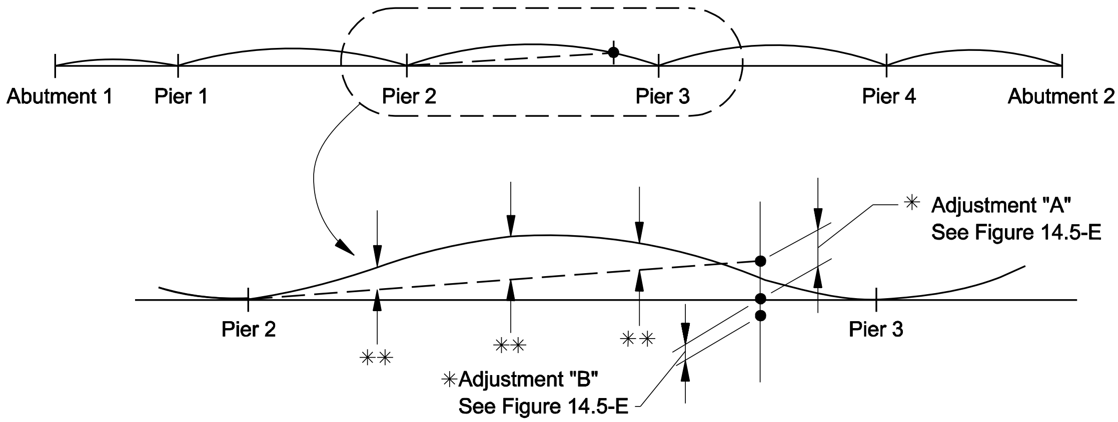
14.5.3.7.2 *Service Limit State*

Reference: LRFD Article 5.8.5

The principal stress-limit requirements of LRFD Article 5.8.5 shall apply to CIP, PT boxes at the Service limit state.

14.5.3.8 Falsework

Cast-in-place, post-tensioned box girder bridges must be supported during their construction. They cannot support even their own dead load until post-tensioning is complete. The temporary supports used are either earth fills, if traffic does not have to be maintained, or falsework. Earth fills must be compacted sufficiently to keep settlement to a minimum. Falsework usually consists of a combination of timber and steel structural components. The falsework is designed to carry the entire dead load of the bridge and construction loads in accordance with the *NDOT Standard Specifications.*



Notes:

✳ See Figure 14.5-E for values to use. These depend on period of time between prestressing of Frame 2 and load transfer from Frame 1.

✳✳ Adjusted values of camber taken from long cantilever values from analysis. Decrease linearly back to pier for profile adjustment (add to profile grade).

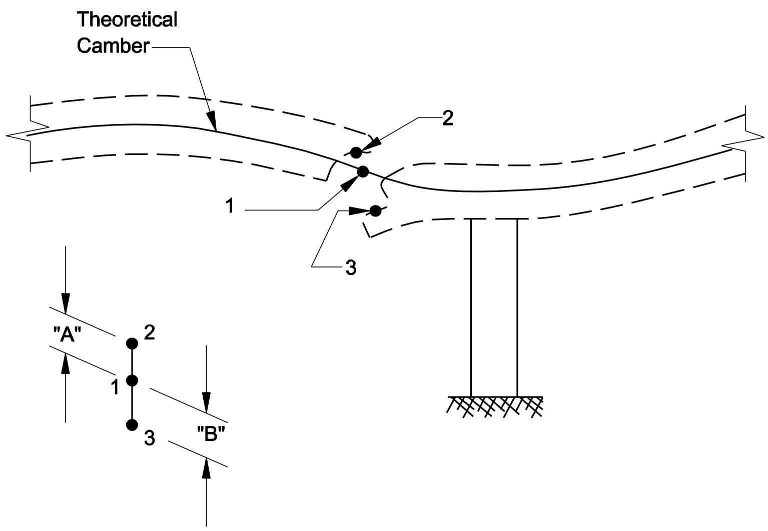
CAMBER DIAGRAM

Figure 14.5-D

Elapsed time in days measured from prestressing short hinge side until closure and load transfer	Adjustment "A"	Adjustment "B"
30 days		
60 days		
90 days		
120 days		
180 days		
240 days		
360 days		
720 days		

TIME-DEPENDENT CAMBER VALUES

Figure 14.5-E



Note: Instead of a field adjustment from theoretical camber as shown above, use Figure 14.5-D and Figure 14.5-E for a direct adjustment to the profile grade.

ADJUSTMENT TO THEORETICAL CAMBER FOR HINGED SPAN

Figure 14.5-F

The contractor submits falsework calculations and shop drawings for review and approval. A Nevada registered professional civil/structural engineer must prepare and stamp the shop drawings. In addition, the registered engineer must inspect the completed falsework and certify that it was built according to the approved falsework drawings.

14.5.3.9 Diaphragms

At a minimum, intermediate diaphragms must be placed at mid-span of CIP concrete box girder superstructures. For longer spans, particularly on curved alignment, additional diaphragms should be considered to enhance the distribution of load among girder webs.

14.5.3.10 Responsibilities (Designer/Contractor)

For CIP, post-tensioned concrete box girder bridges, the designer is responsible for establishing the profile for the center of gravity of the post-tensioning steel (see Section 14.5.3.3) and for defining the total jacking force (Pjack) to be applied to the superstructure. The contractor (usually a specialty subcontractor) will determine the number of strands and tendons that will be supplied in each girder in accordance with the requirements for distribution of prestressing force and stressing sequence that is defined in Standard Plan B-28.1.1 and the *Standard Specifications*.

The contractor is required to submit shop drawings that define the details of the proposed post-tensioning system (including the number of tendons per girder, number of strands per tendon, tendon duct layout, anchorage devices, stressing sequence, jacking force for each tendon and theoretical tendon elongation). The bridge designer will review the shop drawings to confirm that the contractor's system provides the correct center of gravity, that the required total jacking force has been provided, and that the requirements for distribution and sequence of stressing have been satisfied. See Appendix 25A.

Post-tensioning systems require confinement reinforcement to distribute the large concentrated forces. The contractor is responsible for the "local zone" reinforcement. See LRFD Article 5.10.9.2.3. This reinforcing steel controls the concrete cracking around the post-tensioning head and is specific to proprietary post-tensioning systems. This reinforcing steel is determined by the prequalification testing and must be included in the shop drawings. The designer is responsible for the "general zone" reinforcement. See LRFD Article 5.10.9.2.2.

14.5.4 Precast, Prestressed Concrete Girders

14.5.4.1 Precast I-Girder Sections

The type of girders used in the superstructure are selected based upon geometric restraints, economy and appearance. NDOT has not adopted standard precast concrete I-girder sections. PCI has developed standard sections that are used in most locations throughout the United States. However, precasting plants in States adjacent to Nevada may use standard sections specific to their State. Because there are no precasting plants in Nevada, the designer should contact precasters that are likely to provide girders for a project and discuss their product line.

To ensure that the structural system has an adequate level of redundancy, NDOT requires a minimum of four girder lines on new bridges, except as allowed in Section 11.4.5.2 of this *Manual*.

14.5.4.2 General

Reference: LRFD Article 5.9

This Section addresses the general design theory and procedure for precast, prestressed (pre-tensioned) concrete girders. For design examples, consult the *PCI Bridge Design Manual*, Chapter 9.

Bridges consisting of simple-span precast concrete girders and cast-in-place concrete slabs shall be made continuous for live load and superimposed dead loads by using a cast-in-place closure diaphragm at piers whenever possible. The design of the girders for continuous structures is similar to the design for simple spans except that, in the area of negative moments, the member is treated as an ordinary reinforced concrete section, and the bottom flanges of adjoining girders are connected at the interior supports by reinforcement projecting from girder ends into a common diaphragm. The members shall be assumed to be fully continuous with a constant moment of inertia when determining both the positive and negative moments due to loads applied after continuity is established.

The resistance factor "φ" (LRFD Article 5.5.4) for flexure shall be 1.0, except for the design of the negative-moment steel in the deck for structures made continuous for composite loads only and having a poured-in-place continuity diaphragm between the ends of the girders over the piers. For this case, the resistance factor φ shall be the 0.90 value for reinforced concrete members in flexure.

14.5.4.3 Stage Loading

There are four loading conditions that must be considered in the design of a precast, prestressed girder:

1. The first loading condition is when the strands are tensioned in the bed prior to placement of the concrete. Seating losses, relaxation of the strand and temperature changes affect the stress in the strand prior to placement of the concrete. It is the fabricator's responsibility to consider these factors during the fabrication of the girder and to make adjustments to the initial strand tension to ensure that the tension prior to release meets the design requirements for the project. The prestressing shop drawings should present a discussion on the fabricator's proposed methods to compensate for seating losses, relaxation and temperature changes.

2. The second loading condition is when the strands are released and the force is transferred to the concrete. After release, the girder will camber up ("hog up") and be supported at the girder ends only. Therefore, the region near the end of the member is not subject to bending stresses due to the dead load of the girder and may develop tensile stresses in the top of the girder large enough to crack the concrete. The critical sections for computing the critical temporary stresses in the top of the girder should be near the end and at all debonding points. At the designer's option, if he/she chooses to consider the transfer length of the strands at the end of the girder and at the debonding points, then the stress in the strands should be assumed to be zero at the end of the girder or debonding point and vary linearly to the full transfer of force to the concrete at the end of the strand transfer length.

   There are several methods to relieve excessive tensile stresses near the ends of the girder:

   - debonding, where the strands remain straight but wrapped in plastic over a predetermined distance to prevent the transfer of prestress to the concrete through bonding;

   - adding additional strands in the top of the girder that are bonded at the ends but are debonded in the center portion of the girder. These strands are typically detensioned after the girder is erected; or

   - deviating some of the strands to reduce the strand eccentricity at the end of the girder.

   The level of effective prestress immediately after release of the strands, which includes the effects of elastic shortening and the initial strand relaxation loss, should be used to compute the concrete stresses at this stage.

3. The third loading condition occurs several weeks to several months after strand release when the girder is erected and the composite deck is cast. Camber growth and prestress losses are design factors at this stage. If a cast-in-place composite deck is placed, field adjustments to the haunch thickness are usually needed to provide the proper vertical grade on the top of deck and to keep the deck thickness uniform. Reliable estimates of deflection and camber are needed to prevent excessive haunch thickness or to avoid significant encroachment of the top of girder into the bottom of the concrete deck. Stresses at this stage are usually not critical.

See Section 8.7 of the *PCI Bridge Design Manual* for determining the girder camber at erection.

4. The fourth loading condition is after an extended period of time during which all prestress losses have occurred and loads are at their maximum. This is often referred to as the "maximum service load, minimum prestress" stage. The tensile stress in the bottom fibers of the girder at mid-span generally controls the design.

14.5.4.4 Debonded Strands

Debonding of strands at the ends of precast, pretensioned concrete girders will be allowed on projects for NDOT with the following restrictions:

1. A maximum of 25% of the total number of prestressing strands may be debonded to satisfy the allowable stress limits. In any row, debonded strands shall not exceed 40% of the total strands in that row.
2. Not more than 40% of the debonded strands or four strands, whichever is greater, shall be terminated at any section.
3. Strands shall be debonded in a pattern that is symmetrical about the vertical axis of the girder.
4. The theoretical number of debonded strands shall be rounded to the closest even number (pairs) of strands, except that debonded strands will not be permitted in rows containing three strands or less.
5. All exterior strands shall be fully bonded (including the entire bottom row).
6. At each end of a girder, the maximum length for debonding is 15% of the entire girder length.

In analyzing stresses and/or determining the required length of debonding, stresses shall be limited to the values in LRFD Article 5.9.4, except that tension is limited to 0.0948 $\sqrt{f'_c}$ for all exposure conditions.

14.5.4.5 Flexural Resistance

The design of prestressed concrete members in flexure normally begins with the determination of the required prestressing level to satisfy service conditions. All load stages that may be critical during the life of the structure from the time prestressing is first applied should be considered. This is then followed by a strength check of the entire member under the influence of factored loads. The strength check seldom requires additional strands or other design changes.

For checking the stresses in the girder at the Service limit state, the following basic assumptions are made:

1. Planar sections remain plane, and strains vary linearly over the entire member depth. Therefore, composite members consisting of precast concrete girders and cast-in-place decks must be adequately connected so that this assumption is valid and all elements respond to superimposed loads as one unit. Deck concrete is transformed to girder concrete when computing section properties by multiplying the effective deck width by

the ratio of the deck concrete modulus of elasticity to the girder concrete modulus of elasticity. The gross concrete section properties shall be used (i.e., the area of prestressing strands and reinforcing steel is not transformed).

2. The girder is assumed to be uncracked at the Service limit state.

3. Stress limits are not checked for the deck concrete in the negative-moment region because the deck concrete is not prestressed.

14.5.4.6 Interface Shear

Reference: LRFD Article 5.8.4

Cast-in-place concrete decks designed to act compositely with precast concrete girders must be able to resist the interface shearing forces between the two elements. The following formula, substituting LRFD Equation 5.8.4.2-2 into LRFD Equation 5.8.4.2-1, may be used to determine the factored interface shear stress, V_{vi}:

$$V_{ui} = 12\ V_{u1} / d_v$$

The factored interface shear force shall be less than or equal to the factored nominal interface shear resistance; i.e.:

$$V_{ui} \leq \phi V_{ni}$$

where: $$V_{ni} = cA_{cv} + \mu \left(A_{vf}\ f_y + P_c\right) \qquad \text{(LRFD Eq. 5.8.4.1-3)}$$

The permanent net force normal to the interface, P_c, may be conservatively neglected if it is compressive.

14.5.4.7 Diaphragms

Reference: LRFD Article 5.13.2.2

NDOT practice is to place one full-depth, cast-in-place diaphragm between every girder at mid-span. Provide additional diaphragms as required.

For precast, prestressed girder spans, cast-in-place concrete diaphragms shall be used at all supports with the girders embedded a minimum of 6 in into the diaphragm. For spans greater than 40 ft, intermediate diaphragms shall also be used and shall be constructed of cast-in-place concrete. At a minimum, one line of intermediate diaphragms shall be used in each span greater than 40 ft. For skews of 20° or less, the intermediate diaphragms may be placed along the skew of the bridge. For skews in excess of 20°, the intermediate diaphragms shall be placed perpendicular to the girders. The tops of the intermediate diaphragms should be detailed continuous with the deck slab. Slabs shall not be poured until a minimum of seven days after the interior diaphragms are poured or until the diaphragm concrete reaches a compressive strength of 3 ksi.

For continuous precast, prestressed girder spans, the closure diaphragms at the piers shall be cast separately from the deck slab. For integral abutments, the end diaphragms shall also be cast separately from the deck slab.

14.5.4.8 Sole Plates

For an instantaneous slope at the bottom of the girder greater than or equal to 2%, beveled sole plates shall be used to allow for level girder seats.

14.5.4.9 Responsibilities

14.5.4.9.1 *Designer*

The bridge designer is responsible for ensuring that the proposed design will work. The designer will choose a cross section with a center of gravity (force and location) and provide a strand/tendon size and pattern to achieve the required allowable Service limit state stresses and factored flexural resistance. The contract documents will specify the exact value with respect to f'_c that the contractor must reach at release and at 28-days. See Section 14.5.2.1. The designer is also responsible for a preliminary investigation of shipping and handling issues where larger or long precast girders are used or where unusual site access conditions are encountered.

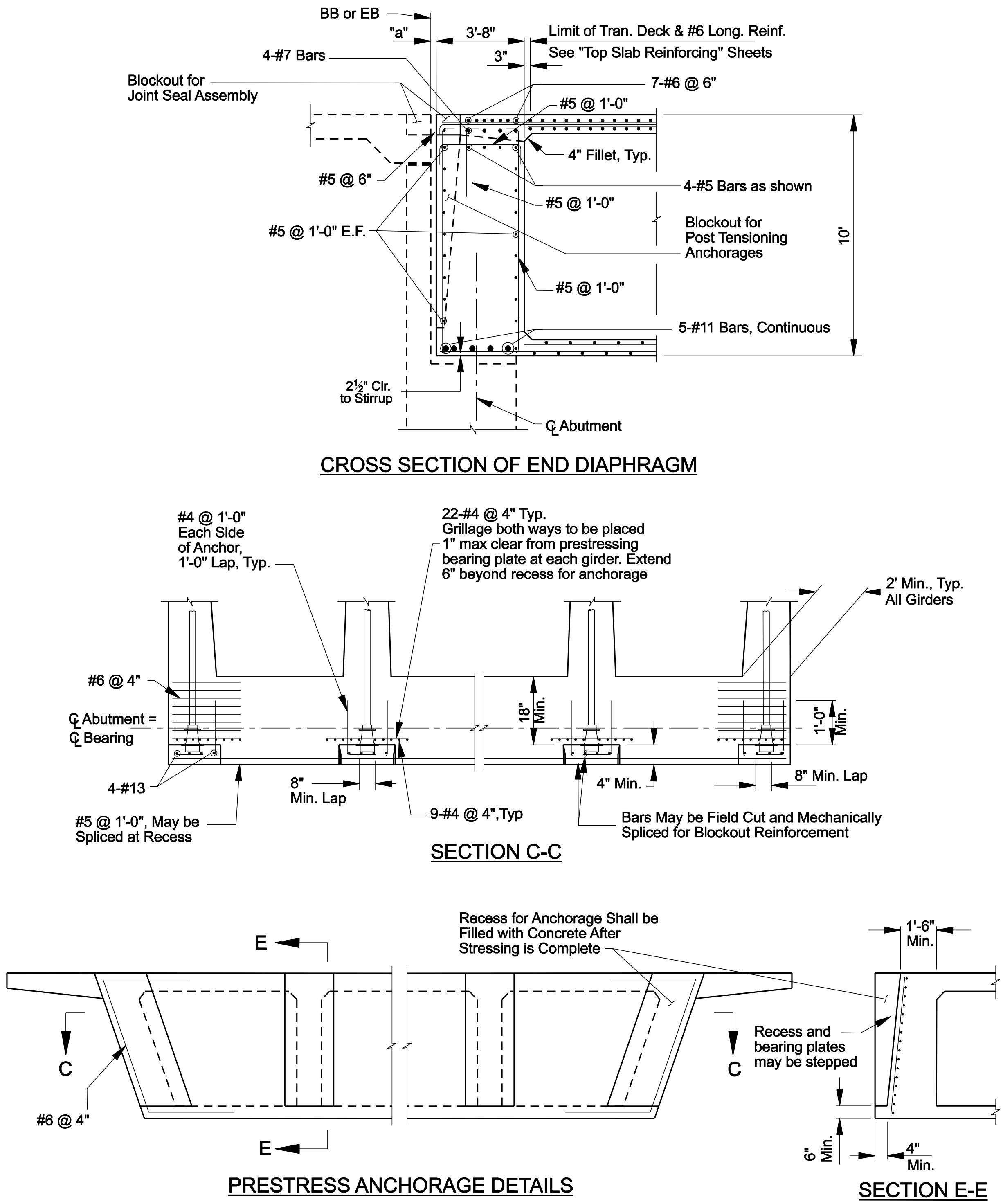
14.5.4.9.2 *Contractor*

In general, the contractor is responsible for implementing the prestressed concrete design according to the bridge designer's specifications. The contractor will provide shop drawings showing all calculations. See Appendix 25A for shop drawing checklists. In addition, for precast girders, the contractor is responsible for investigating stresses in the components during proposed handling, transportation and erection. The contractor may propose changes to the cross sectional shape of the girder. In these cases, the contractor must redesign the girder to meet all requirements of the project. A registered civil/structural engineer licensed in Nevada must submit design calculations and drawings for approval.

14.5.5 Design Details

The designer must prepare a camber diagram that shows the amount of camber needed to counteract the dead load and superimposed dead-load (if any) deflection; see the *NDOT Bridge Drafting Guidelines*. The calculation of the dead-load camber should be based on the gross section properties and LRFD Article 5.7.3.6.

Figure 14.5-G illustrates the typical NDOT design detail for a girder-end diaphragm with recessed anchorages. See the *NDOT Standard Plans* for an anchorage detail on the outside of end diaphragms.



Note: Dimension "a" varies from project to project.

GIRDER-END DIAPHRAGM DETAIL
(With Recessed Anchorages)

Figure 14.5-G

Chapter 15

STEEL STRUCTURES

NDOT STRUCTURES MANUAL

September 2008

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Chapter 15
STEEL STRUCTURES

This Chapter discusses structural steel provisions in Section 6 of the *LFRD Bridge Design Specifications* that require amplification or clarification for NDOT-specific application. Section 11.5 provides criteria for the general site conditions for which structural steel is appropriate. This includes span lengths, girder spacing, geometrics, aesthetics and cost.

15.1 GENERAL

15.1.1 <u>Economical Steel Superstructure Design</u>

15.1.1.1 General

Factors that influence the initial cost of a steel bridge include, but are not limited to, detailing practices, the number of girders (for a girder bridge), the grade of steel, type and number of substructure units (i.e., span lengths), steel tonnage, fabrication, transportation and erection. The cost associated with these factors changes periodically in addition to the cost relationship among them. Therefore, the guidelines used to determine the most economical type of steel girder on one bridge must be reviewed and modified as necessary for future bridges.

Based upon market factors, the availability of steel may be an issue in meeting the construction schedule. It is the responsibility of the bridge designer to verify the availability of the specified steel and beam section. Bridge designers must contact producers and fabricators to ensure the availability of plates and rolled beams. For more detailed information on availability, see Section 1.4 of the AASHTO/NSBA Steel Bridge Collaboration's *Guidelines for Design for Constructibility*, G12.1-2003.

15.1.1.2 Number of Girders/Girder Spacing

See Section 11.4.5.2 for general information on the number of girders in girder bridges. See Section 11.5.3.4 for NDOT criteria on typical girder spacing for steel bridges. For detailed commentary on steel girder spacing, see Section 1.2 of the AASHTO/NSBA Steel Bridge Collaboration's *Guidelines for Design for Constructibility*, G12.1-2003.

15.1.1.3 Exterior Girders

The location of the exterior girder with respect to the overhang is controlled by these factors:

- Locate the exterior girder to limit the dead load and live load on the exterior girder such that the exterior girder does not control the design (i.e., the interior and exterior girders are identical).
- Consider the minimum and maximum overhang widths that are specified in Section 16.2.9.
- The space required for deck drains may have an effect on the location of the exterior girder lines.

15.1.1.4 Span Arrangements

Where pier locations are flexible, the bridge designer should optimize the span arrangement. In selecting an optimum span arrangement, it is critical to consider the cost of the superstructure, substructure, foundations and approaches together as a total system.

To provide a balanced span arrangement for continuous steel bridges, the end spans should be approximately 80% of the length of interior spans. This results in the largest possible negative moments at the piers and smaller resulting positive moments and girder deflections. As a result, the optimum proportions of the girder in all spans will be nearly the same, resulting in an efficient design. End spans less than 50% of the interior span lengths should be avoided to mitigate uplift concerns.

15.1.2 <u>Rolled Beams vs Welded Plate Girders</u>

15.1.2.1 General

Typical NDOT practice is to use rolled beams for spans up to approximately 90 ft. Welded plate girders are used for spans from approximately 90 ft to 400 ft.

When rolled beams are specified, ensure that the selected sections are available consistent with the construction schedule. The *NDOT Standard Specifications* allows the contractor to substitute welded plate girders comprised of plates having a thickness equal to those of the flange and web of the specified rolled beams. For more detailed information, see Section 1.1 of the AASHTO/NSBA Steel Bridge Collaboration's *Guidelines for Design for Constructibility*, G12.1-2003.

15.1.2.2 Welded Plate Girders

Design welded steel plate girders to optimize total cost including material costs while also considering fabrication and erection costs. Top flanges of composite plate girders are typically smaller than their bottom flanges. The flange section is varied along the length of the bridge generally following the moment envelope to save cost by offsetting the increased fabrication costs of welded flange transitions with larger savings in material costs. Typically, only flange thicknesses, not widths, are varied within a field section to reduce fabrication costs. The webs of plate girders are typically deeper and thinner than the webs of rolled beams. To save in total costs, the designer should increase minimum web thicknesses to avoid the use of stiffeners.

Due to buckling considerations, the stability of the compression flange (i.e., the top flange in positive-moment regions and the bottom flange in negative-moment regions) must be addressed by providing lateral-brace locations based upon LRFD Equation 6.10.8.2.3 instead of the 25-ft diaphragm spacing of the AASHTO *Standard Specifications for Highway Bridges*. The traditional 25-ft diaphragm spacing, however, provides a good minimum preliminary value.

On straight bridges (skewed or non-skewed), diaphragms are detailed as secondary members. On horizontally curved bridges, diaphragms must be designed as primary members, because horizontally curved girders transfer a significant amount of load between girders through the diaphragms.

15.1.2.3 Rolled Beams

Rolled steel beams are characterized by doubly symmetrical, as-rolled cross sections with equal-dimensioned top and bottom flanges and relatively thick webs. Thus, the cross sections are not optimized for weight savings (as is a plate girder) but are cost effective due to lower fabrication and erection costs. The relatively thick webs preclude the need for web stiffeners. Unless difficult geometrics or limited vertical clearances control, rolled steel beam superstructures are more cost effective in relatively shorter spans.

Rolled steel beams are available in depths up to 36 in, with beams 24 in and greater rolled less frequently. Before beginning final design, verify with one or more potential fabricators and/or producers that the section size is available. Beams up to 44 in in depth are available but are usually not of domestic origin.

15.1.3 Economical Plate Girder Proportioning

The AASHTO/NSBA Steel Bridge Collaboration has published the *Guidelines for Design for Constructibility*, G12.1-2003. This document presents cost-effective details for steel bridges from the perspective of the steel fabricator. The following Section presents information from the AASHTO/NSBA *Guidelines* that is of interest.

15.1.3.1 General

Plate girders and rolled beams shall be made composite with the bridge deck through shear studs and should be continuous over interior supports where possible. To achieve economy in the fabrication shop, all girders in a multi-girder bridge should be identical where possible. When using plate girders, a minimal number of plate sizes should be used.

15.1.3.2 Haunched Girders

When practical, girders with constant web depths shall be used. Haunched girders are generally uneconomical for spans less than 300 ft. Parabolic haunched girders may be used where aesthetics or other special circumstances are involved, but constant-depth girders will generally be more cost effective.

15.1.3.3 Flange Plate Sizes

The minimum flange plate size for plate girders is 12 in by 1 in to avoid cupping of the flanges due to distortion from welding. Designers should use as wide a flange girder plate as practical, consistent with stress and b/t (flange width/thickness ratio) requirements. The wide flange contributes to girder stability during handling and in-service, and it reduces the number of passes and weld volume at flange butt welds. As a guide, flange width should be approximately 20% to 25% of web depth. Flange widths should not be sized in any set increments but based on mill plate widths minus the waste from torch cutting. Limit the maximum flange thickness to 3 in to ensure more uniform through-thickness properties. Thicker plates demonstrate relatively poor material properties near mid-thickness.

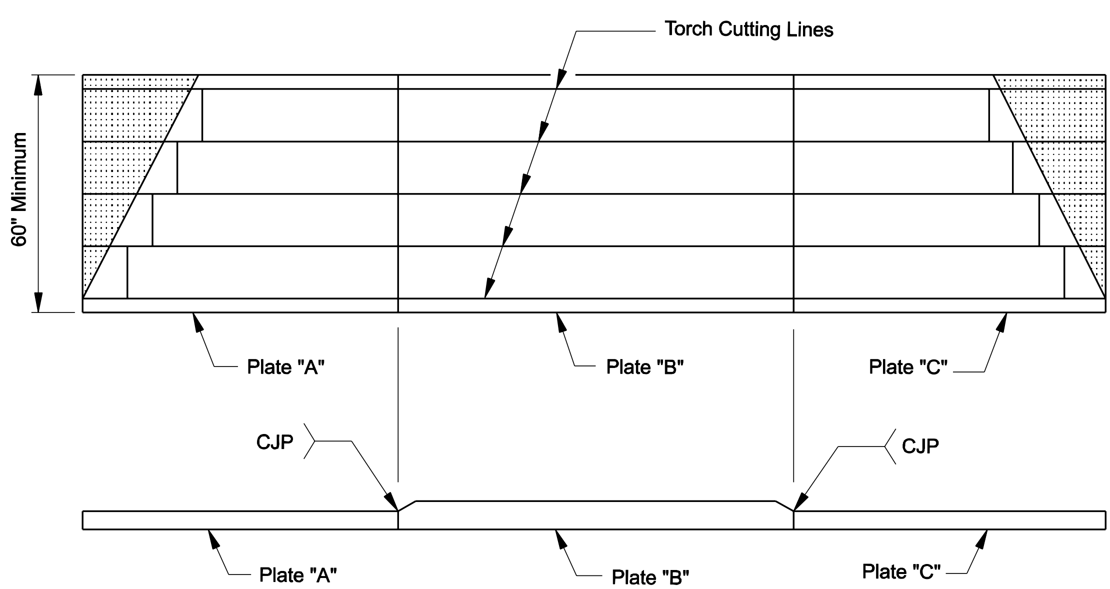
Within a single field section (i.e., an individual shipping piece), the flanges should be of constant width. A design using multiple identical girders with constant-width flanges minimizes fabrication costs.

Proportion flanges so the fabricator can economically cut them from steel plate between 60 in and 120 in wide. The most economical mill widths are 72 in, 84 in, 96 in and 120 in. Allow ¼ in for internal torch cutting lines and ½ in for exterior torch cutting lines; see Figure 15.1-A. Flanges should be grouped to provide an efficient use of the plates. Because structural steel plate is most economically purchased in these widths, it is advantageous to repeat plate thicknesses as much as practical. Many of the plates of like width can be grouped by thickness to meet the minimum width purchasing requirement, but this economical purchasing strategy may not be possible for thicker, less-used plates.

The most efficient method to fabricate flanges is to groove-weld together several wide plates of varying thicknesses received from the mill. After welding and non-destructive testing, the individual flanges are "stripped" from the full plate. This method of fabrication reduces the number of welds, individual runoff tabs for both start and stop welds, the amount of material waste and the number of X-rays for non-destructive testing. The objective, therefore, is for flange widths to remain constant within an individual shipping length by varying material thickness as required. Figure 15.1-A illustrates one example of an efficient fabrication for girders.

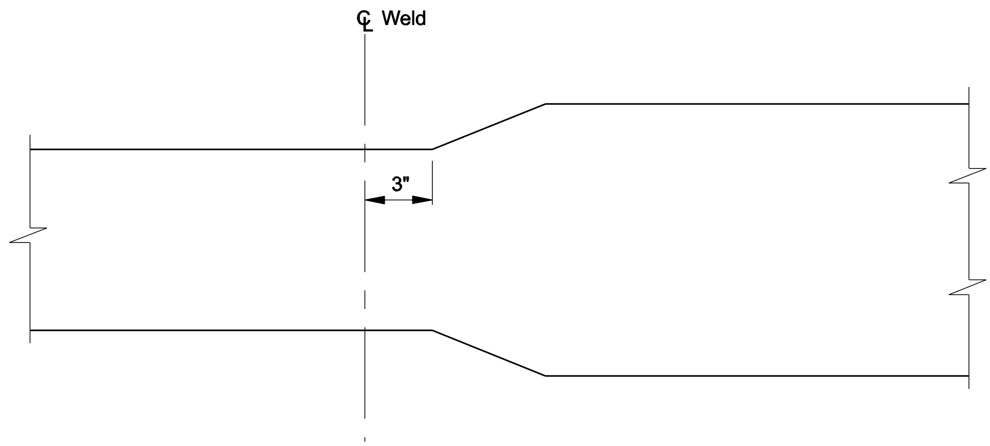
Constant flange width within a field section may not always be practical in girder spans over 300 ft where a flange width transition may be required in the negative bending regions. Though not preferred, if a transition in width must be provided, shift the butt splice a minimum of 3 in from the transition into the narrower flange plate. See Figure 15.1-B. This 3-in shift makes it simpler to fit run-off tabs, weld and test the splice and then grind off the run-off tabs.

For additional information on sizing flange plates, see Section 1.5 of the AASHTO/NSBA Steel Bridge Collaboration's *Guidelines for Design for Constructibility*, G12.1-2003.



GROUPING FLANGES FOR EFFICENT FABRICATION
(From the AASHTO/NSBA Steel Bridge Collaboration)

Figure 15.1-A



FLANGE WIDTH TRANSITION
(Plan View)

Figure 15.1-B

15.1.3.4 Field Splices

Field splices are used to reduce shipping lengths, but they are expensive and their number should be minimized. The preferred maximum length of a field section is 120 ft; however, lengths up to 150 ft are allowed, but field sections greater than 120 ft should not be used without considering shipping, erection and site constraints. As a general rule, the unsupported length in compression of the shipping piece divided by the minimum width of the flange in compression in that piece should be less than approximately 85. Good design practice is to reduce the flange cross sectional area by no more than approximately 25% of the area of the heavier flange plate at field splices to reduce the build-up of stress at the transition. For continuous spans, the field sections over a pier should be of constant length to simplify erection.

NDOT does not specify a maximum weight for field sections.

15.1.3.5 Shop Splices

Include no more than two shop flange splices in the top or bottom flange within a single field section. The designer should maintain constant flange widths within a field section for economy of fabrication as specified in Section 15.1.3.3. In determining the points where changes in plate thickness occur within a field section, the designer should weigh the cost of groove-welded splices against extra plate area. Table 1.5.2.A of the AASHTO/NSBA Steel Bridge Collaboration's *Guidelines for Design for Constructibility*, G12.1-2003, provides guidelines for weight savings for Grade 50 steel required to justify a flange shop splice.

In many cases, it may be advantageous to continue the thicker plate beyond the theoretical step-down point to avoid the cost of the groove-welded splice. The contract documents should allow this alternative.

To facilitate testing of the weld, locate flange shop splices at least 2 ft away from web splices and locate flange and web shop splices at least 6 in from transverse stiffeners.

Section 1.5 of the AASHTO/NSBA Steel Bridge Collaboration's *Guidelines for Design for Constructibility*, G12.1-2003, provides additional guidance on shop splices.

15.1.3.6 Web Plates

Where there are no depth restrictions, the web depth should be optimized. NSBA provides a service to bridge owners to assist in optimizing web depths. Other sources may also be used if they are based upon material use and fabrication unit costs. The minimum web thickness shall be ½ in. Web thickness at any splice should not be changed by less than ⅛ in. Maintain symmetry by aligning the centerlines of the webs at splices.

Web design can have a significant impact on the overall cost of a plate girder. Considering material costs alone, it is desirable to make girder webs as thin as design considerations will permit. However, this practice will not always produce the greatest economy because fabricating and installing transverse stiffeners is one of the most labor-intensive of shop operations. The following guidelines apply to the use of transverse stiffeners:

1. Unstiffened webs are generally more economical for web depths approximately 48 in or less.

2. Between 48-in and 72-in depths, consider options for a partially stiffened and unstiffened web, with unstiffened webs preferred. A partially stiffened web is defined as one whose thickness is 1/16 in less than allowed by specification for an unstiffened web at a given depth and where stiffeners are required only in areas of higher shear.

3. Above 72 in, consider options for partially stiffened or fully stiffened webs, with partially stiffened webs preferred. A fully stiffened web is defined as one where stiffeners are present throughout the span.

15.1.3.7 Transverse Stiffeners

Flat bars (i.e., bar stock rolled to widths up to 8 in at the mill) are typically more economical than plates for stiffeners. The stiffeners can be fabricated by merely shearing flat bars of the specified width to length. Stiffeners that are intended to be fabricated from bars should be proportioned in ¼-in increments in width and in ⅛-in increments in thickness. A fabricator should be consulted for available flat sizes.

15.1.3.8 Longitudinally Stiffened Webs

Longitudinally stiffened webs are typically not used. In addition to being considered uneconomical, the ends of longitudinal stiffeners are fatigue sensitive if subject to applied tensile stresses. Therefore, where used, they must be ended in zones of little or no applied tensile stresses.

15.1.4 Falsework

Steel superstructures shall be designed without intermediate falsework during the placing and curing of the concrete deck slab.

15.1.5 AISC Certification Program

The AISC certification program for structural steel fabricators includes several categories:

1. SBR – Simple Steel Bridge Structures. Includes highway sign structures, parts for bridges (e.g., cross frames) and unspliced rolled beam bridges. NDOT does not require this certification for sign structures. The high-mast lighting standard plan, however, incorporates this certification.

2. CBR – Major Steel Bridges. All bridge structures other than unspliced rolled beam bridges.

3. CSE and ACSE. These steel bridge erector certifications are for simple bridges (CSE) and complex bridges (ACSE). NDOT does not typically use these certifications.

15.1.6 Buy America

23 CFR Part 635.410 presents the "Buy America" provisions for Federal-aid projects. These provisions require that manufacturing processes for steel and iron products and their coatings must occur in the United States. A minimal amount of foreign material can be used, if it does not exceed 1/10 of 1% of the total contract price or $2,500, whichever is more. These "Buy America" provisions are included in the *NDOT Standard Specifications*. Note that "Buy American" provisions are different and do not apply to Federal-aid highway projects.

15.2 MATERIALS

Reference: LRFD Article 6.4

15.2.1 <u>Structural Steel</u>

Reference: LRFD Article 6.4.1

The following presents typical NDOT practices for the material type selection for structural steel members.

15.2.1.1 Grade 36

Grade 36 steel is typically used for the following structural members:

- transverse stiffeners,
- diaphragms, and
- bearing plates.

Grade 36 steel is becoming less used and, thus, less available at times. Generally, there is little or no cost difference between Grade 50 and Grade 36 steel.

15.2.1.2 Grade 50

Grade 50 steel is typically used for the following structural members:

- rolled beams,
- plate girders,
- splice plates,
- diaphragms,
- steel piles, and
- bearing plates.

15.2.1.3 High-Performance Steel

15.2.1.3.1 *Grade HPS70W*

For some plate-girder bridges, a good choice of steel may be Grade HPS70W. In addition to increased strength, the high-performance steels exhibit enhanced weathering, toughness and weldability properties. The premium on material costs is offset by a savings in tonnage. The most cost-effective design solutions tend to be hybrid girders with Grade 50 webs with HPS70W tension and compression flanges in the negative-moment regions and tension flanges only in the positive-moment regions.

HPS70W may be painted for aesthetic reasons.

15.2.1.3.2 *Grade HPS100W*

A new high-performance steel with a minimum specified yield strength of 100 ksi has been recently introduced. It has yet to be proven cost-effective for girder bridge applications and should not be used.

15.2.1.4 Unpainted Weathering Steel

15.2.1.4.1 *General*

In general, NDOT discourages the use of unpainted weathering steel (i.e., Grades 50W, HPS70W) because of aesthetic considerations.

Unpainted weathering steel is often the more cost-effective choice for structural steel superstructures. The initial cost advantage when compared to painted steel can range up to 15%. When future repainting costs are considered, the cost advantage is more substantial. This reflects, for example, environmental considerations in the removal of paint, which significantly increases the life-cycle cost of painted steel. The application of weathering steel and its potential problems are discussed in depth in FHWA Technical Advisory T5140.22 "Uncoated Weathering Steel in Structures," October 3, 1989. Also, the proceedings of the "Weathering Steel Forum," July 1989, are available from the FHWA Office of Implementation, HRT-10.

Despite its cost advantage, the use of unpainted weathering steel is not appropriate in all environments and at all locations. The most prominent disadvantage of weathering steel is aesthetics. The inevitable staining of the steel where susceptible to water leakage from above (e.g., below deck joints) creates a poor image (i.e., one of lack of proper maintenance) to the traveling public. Therefore, NDOT policy is to only consider the use of weathering steel for highway bridges over railroads and over stream crossings that are not adjacent to highways; i.e., where the girders are not visible to the traveling public. In addition, weathering steel shall not be used at locations where the following conditions exist:

1. <u>Environment</u>. Unpainted weathering steel shall not be used in industrial areas where concentrated chemical fumes may drift onto the structure, or where the nature of the environment is questionable.

2. <u>Water Crossings</u>. Unpainted weathering steel shall not be used over bodies of water where the clearance over the ordinary high water is 10 ft or less.

3. <u>Grade Separations</u>. Unpainted weathering steel shall not be used for highway grade separation structures.

The staining potential can be addressed by applying a silane treatment conforming to Section 646 of the *NDOT Standard Specifications* to the substructure elements. The silane treatment should be applied to mature concrete in accordance with the manufacturer's recommendations.

For additional guidance on the appropriate application of unpainted weathering steel, see the AISI publication *Performance of Weathering Steel in Highway Bridges: A Third Phase Report*.

15.2.1.4.2 *Design Details for Weathering Steel*

Where weathering steel girders are used, the bearing plates shall be the same steel as the girders they support. The bolts, nuts, washers and Direct Tension Indicators (DTIs) shall be Type 3 as specified in ASTM A325/ASTM A563 and ASTM F 959.

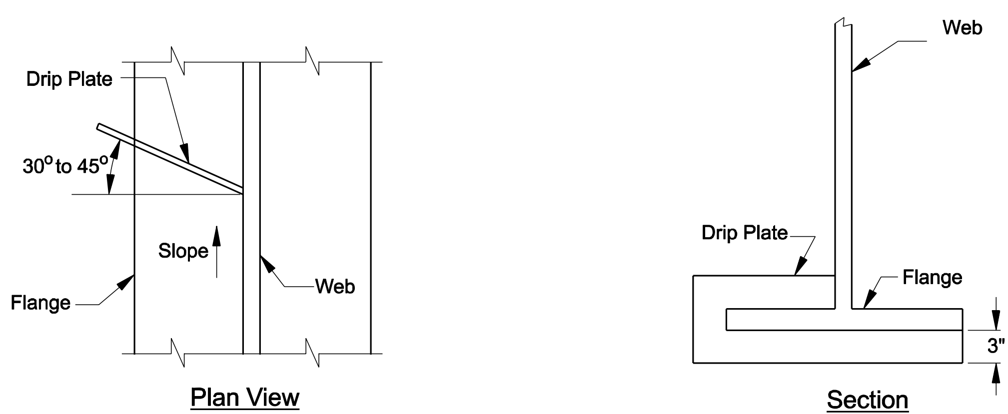
Paint weathering steel at the ends of girders, at expansion joints and over piers for a distance of 10 ft or 1.5 times the web depth, whichever is greater. Use only the prime coat of the approved bridge paint systems.

When using unpainted weathering steel, the following drainage treatments shall be incorporated:

1. Minimize the number of bridge deck drains and extend the drainage outlets below the steel bottom flange.

2. Eliminate details that serve as water and debris "traps." Seal or paint overlapping surfaces exposed to water. This sealing or painting applies to non-slip-critical bolted joints. Slip-critical bolted joints or splices should not produce "rust-pack" when the bolts are spaced according to the *LRFD Specifications* and, therefore, do not require special protection.

3. Place a drip plate or other material transverse across the top of the bottom flange in front of the substructure elements to prevent water from running off the flange onto the concrete. Ensure that these attachments meet all fatigue requirements. Figure 15.2-A shows a typical drip plate detail.



DRIP PLATE DETAIL

Figure 15.2-A

15.2.1.5 Charpy V-Notch Fracture Toughness

Reference: LRFD Article 6.6.2

The temperature zone appropriate for using LRFD Table 6.6.2-1 for the State of Nevada is Temperature Zone 2.

15.2.2 Bolts

Reference: LRFD Article 6.4.3

15.2.2.1 Type

For normal construction, high-strength bolts shall be:

1. Painted Steel: Use ⅞-in A325 (Type 1).
2. Weathering Steel: Use ⅞-in A325 (Type 3).

15.2.2.2 Hole Size

Typically, do not use oversized or slotted holes; these may be used only in unusual circumstances with approval. Use appropriate design considerations when oversized or slotted holes are approved.

15.2.3 Splice Plates

In all cases, steel for all splice and filler plates shall be the same material as used in the web and flanges of plate girders.

15.3 HORIZONTALLY CURVED MEMBERS

Reference: LRFD Articles 6.10 and 6.11

15.3.1 General

Use a curved girder on curved alignments, unless otherwise approved.

The *LRFD Specifications* includes horizontally curved girders as a part of the provisions for proportioning I-shaped and tub girders at both the Strength and Service limit states. In addition, analysis methodologies that detail various required levels of analysis are also specified.

15.3.2 Diaphragms, Bearings and Field Splices

Cross frames and diaphragms shall be considered primary members. However, due to the difficulty of obtaining a Charpy specimen from a rolled shape such as an angle, Charpy V-notch impact-energy testing of the cross frames is not required. All curved steel simple-span and continuous-span bridges shall have diaphragms directed radially except end diaphragms, which should be placed parallel to the centerline of bearings.

Design all diaphragms, including their connections to the girders, to carry the total load to be transferred at each diaphragm location. Cross frames and diaphragms should be as close as practical to the full depth of the girders. Design cross frame and diaphragm connections for the 75% and average load provisions of LRFD Article 6.13.1, unless actual forces in the connections are determined from an appropriate structural model. Using the provisions of LRFD Article 6.13.1 may result in very large connections that are difficult to detail.

Bridges expand and contract in all directions. For typical bridges that are long in relationship to their width, the transverse expansion is ignored. For ordinary geometric configurations where the bridge length is long relative to the bridge width (say, 2½ times the width) and the degree of curvature is moderate (those satisfying the requirements of LRFD Article 4.6.1.2.4b), no additional consideration is necessary for the unique expansion characteristics of horizontally curved structures. Wide, sharply curved or long-span structures may require the use of high-load multi-rotational bearings. The designer must consider providing restraint either radially and/or tangentially to accommodate the transfer of seismic forces and the thermal movement of the structure because the bridge tries to expand in all directions.

Design the splices in flanges of curved girders to carry flange bending or lateral bending stresses and vertical bending stresses in the flanges.

15.4 FATIGUE CONSIDERATIONS

Reference: LRFD Article 6.6

LRFD Article 6.6.1 categorizes fatigue as either "load induced" or "distortion induced." Load induced is a "direct" cause of loading. Distortion induced is an "indirect" cause in which the force effect, normally transmitted by a secondary member, may tend to change the shape of or distort the cross section of a primary member.

15.4.1 <u>Load-Induced Fatigue</u>

Reference: LRFD Article 6.6.1.2

15.4.1.1 General

LRFD Article 6.6.1.2 provides the framework to evaluate load-induced fatigue. This Section provides additional information on the implementation of LRFD Article 6.6.1.2.

Load-induced fatigue is determined by the following:

- the stress range induced by the specified fatigue loading at the detail under consideration;
- the number of repetitions of fatigue loading a steel component will experience during its 75-year design life. For higher truck-traffic volume bridges, this is taken as infinite. For lower truck-traffic volume bridges, this is determined by using anticipated truck volumes; and
- the nominal fatigue resistance for the Detail Category being investigated.

15.4.1.2 NDOT Policy

NDOT policy on load-induced fatigue is as follows:

1. <u>New Bridges</u>. For new steel bridges, it is mandatory to design for infinite life. In addition, all details must have a fatigue resistance greater than or equal to Detail Category C (i.e., Detail Categories A, B, B′, C and C′).
2. <u>Existing Bridges</u>. Section 22.4.3.5 presents NDOT policy for load-induced fatigue for work on existing bridges (e.g., bridge rehabilitation, bridge widening).

Any exceptions to NDOT policy on load-induced fatigue require the approval of the Chief Structures Engineer.

15.4.1.3 Fatigue Stress Range

The following applies:

1. <u>Regions</u>. Fatigue should only be considered in those regions of a steel member that experience a net applied tensile stress, or where the compressive stress of the

unfactored permanent load is less than twice the maximum fatigue tensile stress. Twice the maximum fatigue tensile stress represents the largest stress range that the detail should experience. This requirement checks to determine if the detail will go into tension. If not, fatigue is not a consideration.

2. Range. The fatigue stress range is the difference between the maximum and minimum stresses at a structural detail subject to a net tensile stress. The stress range is caused by a single design truck that can be placed anywhere on the deck within the boundaries of a design lane. If a refined analysis method is used, the design truck shall be positioned to maximize the stress in the detail under consideration. The design truck should have a constant 30-ft spacing between the 32-kip axles. The dynamic load allowance is 0.15 and the fatigue load factor is 0.75.

3. Analysis. Unless a refined analysis method is used, the single design lane load distribution factor in LRFD Article 4.6.2.2 should be used to determine fatigue stresses. These tabularized distribution-factor equations incorporate a multiple presence factor of 1.2 that should be removed by dividing either the distribution factor or the resulting fatigue stresses by 1.2. This division does not apply to distribution factors determined using the lever rule.

15.4.1.4 Fatigue Resistance

LRFD Article 6.6.1.2.3 groups the fatigue resistance of various structural details into eight categories (A through E′). Experience indicates that Detail Categories A, B and B′ are seldom critical. Investigation of details with a fatigue resistance greater than Detail Category C may be appropriate in unusual design cases. For example, Category B applies to base metal adjacent to slip-critical bolted connections and should only be evaluated when thin splice plates or connection plates are used. For Detail Categories C, C′, D, E and E′, the *LRFD Specifications* requires that the fatigue stress range must be less than the specified fatigue resistance for each of the respective Categories.

The fatigue resistance of a category is determined from the interaction of a Category Constant "A" and the total number of stress cycles "N" experienced during the 75-year design life of the structure. This resistance is defined as $(A/N)^{1/3}$. A Constant Amplitude Fatigue Threshold ($(\Delta F)_{TH}$) is also established for each Category. If the applied fatigue stress range is less than ½ of the threshold value, the detail has infinite fatigue life.

For bridges designed for infinite life, the applied fatigue stress range shall be less than ½ of the threshold value, $½(\Delta F)_{TH}$. This practice provides a theoretical design life of infinity. For all other bridges, the fatigue resistance shall be calculated in accordance with LRFD Article 6.6.1.2.3.

Fatigue resistance is independent of the steel strength. The application of higher grade steels causes the fatigue stress range to increase, but the fatigue resistance remains the same. This independence implies that fatigue may become more of a controlling factor where higher strength steels are used.

* * * * * * * * * *

Example 15.4-1

Given: State Highway System bridge
Two-span continuous bridge, 150-ft each

Area investigated is located 13 ft from interior support
Unfactored DL stress in the top flange = 7.9 ksi Tension
Unfactored fatigue stresses in the top flange using unmodified single lane distribution factor = 5.6 ksi Tension and 0.8 ksi Compression

Find: Determine the fatigue adequacy of the top flange with welded stud shear connectors in the negative moment region.

Solution:

Step 1: *The LRFD Specifications classifies this connection as Detail Category C. Therefore:*

- A = Detail Category Constant = 44.0 x 10^8 ksi^3 (LRFD Table 6.6.1.2.5-1)
- $(\Delta F)_{TH}$ = Constant Amplitude Fatigue Threshold = 10.0 ksi (LRFD Table 6.6.1.2.5-3)

Step 2: *Compute the factored live-load fatigue stresses by applying dynamic load allowance and fatigue load factor and removing the multiple presence factor:*

Tension: 5.6(1.15)(0.75)/1.2	=	4.0 ksi
Compression: 0.8(1.15)(0.75)/1.2	=	0.6 ksi
Fatigue Stress Range	=	4.6 ksi

Step 3: *Check for infinite life:*

First, check the infinite life term (see Commentary C6.6.1.2.5 of the *LRFD Specifications* for a table of single-lane ADTT values for each detail category above which the infinite life check governs). This infinite-life term will typically control the fatigue resistance when traffic volumes are large. $(\Delta F)_n = ½(\Delta F)_{TH} = 0.5(10.0) = 5.0$ ksi. Because the fatigue stress range (4.6 ksi) is less than the infinite life resistance (5.0 ksi), the detail should exhibit infinite fatigue life and, therefore, the detail is satisfactory.

Provisions for investigating the fatigue resistance of shear connectors are provided in LRFD Article 6.10.10.2.

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15.4.2 Distortion-Induced Fatigue

Reference: LRFD Article 6.6.1.3

LRFD Article 6.6.1.3 provides specific detailing practices for transverse and lateral connection plates intended to reduce significant secondary stresses that could induce fatigue crack growth. The provisions of the *LRFD Specifications* are concise and direct and require no mathematical computation of stress range.

15.4.3 Other Fatigue Considerations

Reference: Various LRFD Articles

The designer is responsible for ensuring compliance with fatigue requirements for all structural details (e.g., stiffeners, connection plates, lateral bracing) shown in the contract documents.

In addition to the considerations in Section 15.4.1, the designer should investigate the fatigue provisions in other Articles of Chapter 6 of the *LRFD Specifications*. These include:

- Fatigue due to out-of-plane flexing in webs of plate girders — LRFD Article 6.10.6.
- Fatigue at shear connectors — LRFD Articles 6.10.10.1.2 and 6.10.10.2.
- Bolts subject to axial-tensile fatigue — LRFD Article 6.13.2.10.3.

15.5 GENERAL DIMENSION AND DETAIL REQUIREMENTS

Reference: LRFD Article 6.7

15.5.1 Deck Haunches

A deck haunch is an additional thickness of concrete between the top of the girder and the bottom of the deck to provide adjustability between the top of the cambered girder and the roadway profile. The haunch is detailed at the centerline of bearing and varies in the span, if necessary, to accommodate variations in camber, superelevation ordinate and vertical curve ordinate. The maximum positive camber allowed in excess of that specified at mid-span is ¾ in for spans less than 100 ft and 1½ in for spans more than 100 ft. A 2-in haunch is recommended for spans of less than 100 ft, and a 3-in haunch is recommended for spans of more than 100 ft. The haunch is neglected when determining the resistance of the section. See Section 16.2.2.

15.5.2 Minimum Thickness of Steel

Reference: LRFD Article 6.7.3

For welded plate girder fabrication, minimum thickness requirements are mandated to reduce deformations and defects due to welding. The thickness of steel elements should not be less than:

- Plate girder webs: ½ in
- Stiffeners, connection plates: 7⁄16 in, ½ in preferred
- Plate girder flanges: 1 in
- Bearing stiffener plates: 1 in
- Gusset plates: ⅜ in
- Angles/channels: ¼ in

For more detailed information, see Section 1.3 of the AASHTO/NSBA Steel Bridge Collaboration's *Guidelines for Design for Constructibility*, G12.1-2003.

15.5.3 Dead-Load Deflection

15.5.3.1 Deflections from Deck Shrinkage

In addition to the deflection due to dead load for simple-span bridges, the deflection from shrinkage of the concrete deck shall be computed by:

$$\Delta = \frac{0.00002L^2}{Y_{ts}}$$

Where: Δ = centerline span deflection, ft

L = girder span length, ft

Y_{ts} = distance in ft from CG of steel girder section only to top flange at centerline of span

15.5.3.2 Camber

The entire girder length shall be cambered as required by the loading and profile grade. The loading includes the consideration for shrinkage of the concrete deck as presented in Section 15.5.3.1. In addition, where dead load deflection and vertical curve offset are greater than ¼ in, the girders shall have a compensating camber. Camber will be calculated to the nearest 0.01 ft with ordinates at 0.1 points throughout the length of the girder. Show the required camber values from a chord line that extends from point of support to point of support. The camber shall be parabolic.

A camber diagram is required in all contract documents with structural steel girders.

15.5.4 Diaphragms and Cross Frames

Reference: LRFD Articles 6.7.4 and 6.6.1.3.1

Diaphragms on rolled-beam bridges and cross frames on plate-girder bridges are vitally important in steel girder superstructures. They stabilize the girders in the positive-moment regions during construction and in the negative-moment regions after construction. Cross frames also serve to distribute gravitational, centrifugal and wind loads. The spacing of diaphragms and cross frames should be determined based upon the provisions of LRFD Article 6.7.4.1. As with most aspects of steel girder design, the design of the spacing of diaphragms and cross frames is iterative. A good starting point is the traditional maximum diaphragm and cross frame spacing of 25 ft. Most economical steel girder designs will use spacings typically greater than 25 ft in the positive-moment regions.

15.5.4.1 General

The following applies to diaphragms and cross frames:

1. Location. Place diaphragms or cross frames at each support and throughout the span at an appropriate spacing. The location of the field splices should be planned to avoid conflict between the connection plates of the diaphragms or cross frames and any part of the splice material.

2. Skew. Regardless of the angle of skew, place all intermediate diaphragms and cross frames perpendicular to the girders. Locating cross frames near girder supports on bridges with high skews requires careful consideration. When locating a cross frame between two girders, the relative stiffness of the two girders must be similar. Otherwise, the cross frame will act as a primary member supporting the more flexible girder. This may be unavoidable on bridges with exceptionally high skews where a rational analysis of the structural system will be required to determine actual forces.

3. End Diaphragms and Cross Frames. End diaphragms and cross frames should be placed along the centerline of bearing. Set the top of the diaphragm below the top of the girder to accommodate the joint detail and the thickened slab at the end of the superstructure deck, where applicable. The end diaphragms should be designed to support the edge of the slab including live load plus impact.

4. Interior Support Diaphragms and Cross Frames. Generally, interior support diaphragms and cross frames should be placed along the centerline of bearing. They provide lateral stability for the bottom flange and bearings.

5. Curved-Girder Structures. Diaphragms or cross frames connecting horizontally curved girders are considered primary members and shall be oriented radially.

6. Detailing. Diaphragms and cross frames are typically detailed to follow the cross slope of the deck; i.e., the diaphragm or cross frame is parallel to the bottom of the deck. This allows the fabricator to use a constant drop on each connection plate (i.e., the distance from the bottom of the flange to the first bolt hole on the connection plate is constant). The contract documents should allow the contractor to use diaphragms or cross frames fabricated as a rectangle (as opposed to a skewed parallelogram). In this case, the drops vary across the bridge.

The following identifies typical NDOT practices on the selection of diaphragms and cross frames:

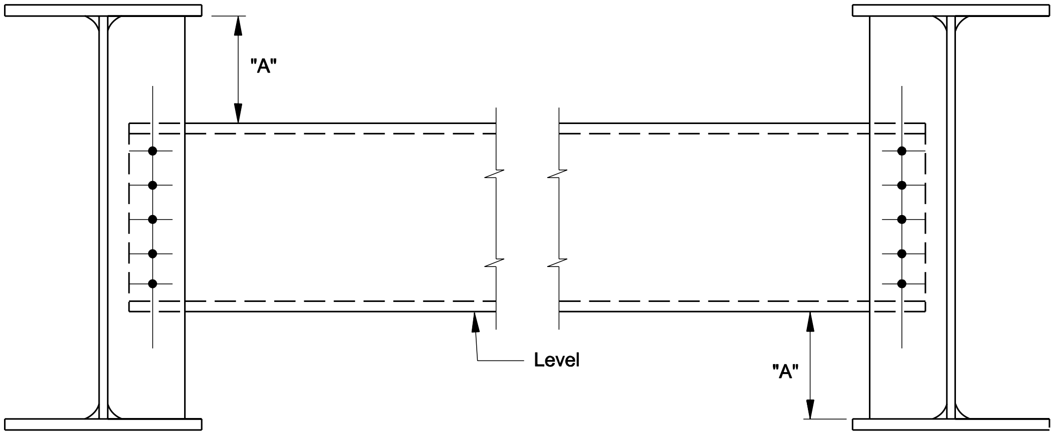
1. Solid Diaphragms. These are preferred for rolled beams. For rolled-beam bridges with seat abutments, the end diaphragms shall be full depth to provide sufficient lateral restraint.

2. K-Frames. These are preferred for plate girder bridges.

3. X-Frames. In the case of relatively narrow girder spacings relative to the girder depth, an X-frame may be more appropriate than a K-frame.

15.5.4.2 Diaphragm Details

On spans composed of rolled beams, diaphragms at interior span points may be detailed as illustrated in Figure 15.5-A. Figure 15.5-B illustrates the typical abutment support diaphragm connection details for rolled beams. Plate girders with web depths of 48 in or less should have similar diaphragm details. For plate girder webs more than 48 in deep, use cross frames as detailed in Figures 15.5-C and 15.5-D.

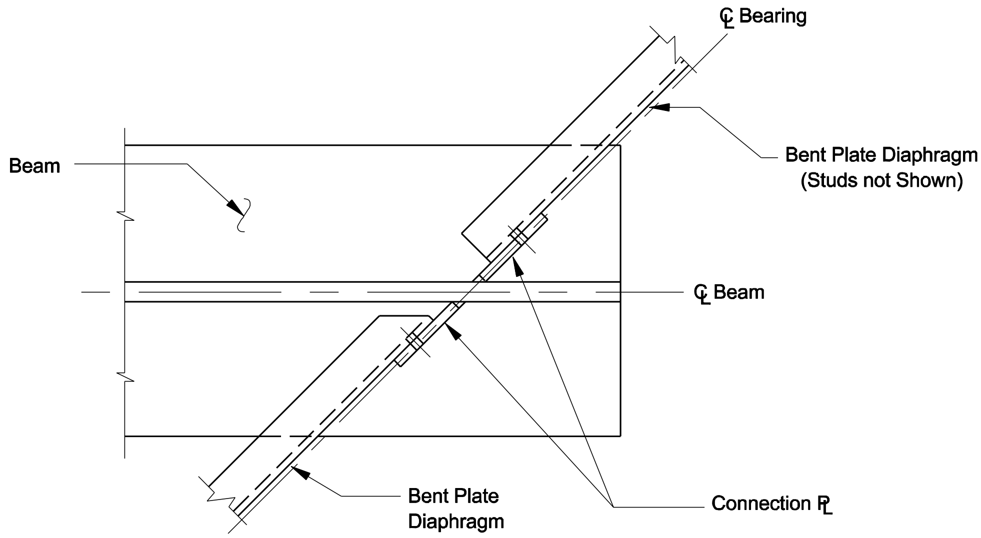


ELEVATION

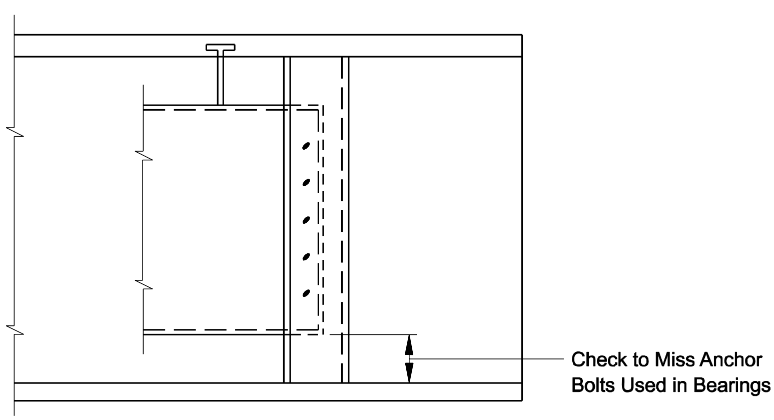
Note: Select a channel depth approximately ½ of the girder depth.

TYPICAL PIER AND INTERMEDIATE DIAPHRAGM CONNECTION
(Rolled Beams)

Figure 15.5-A



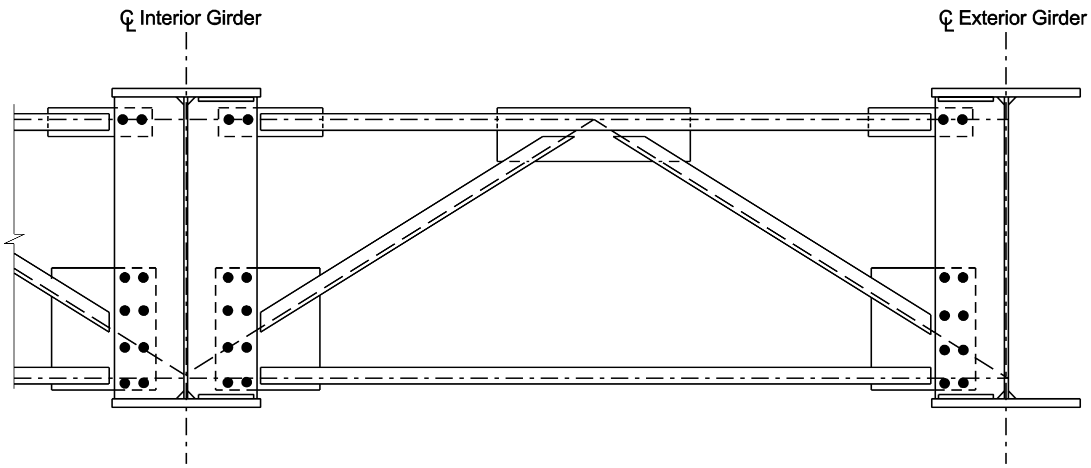
PLAN



ELEVATION

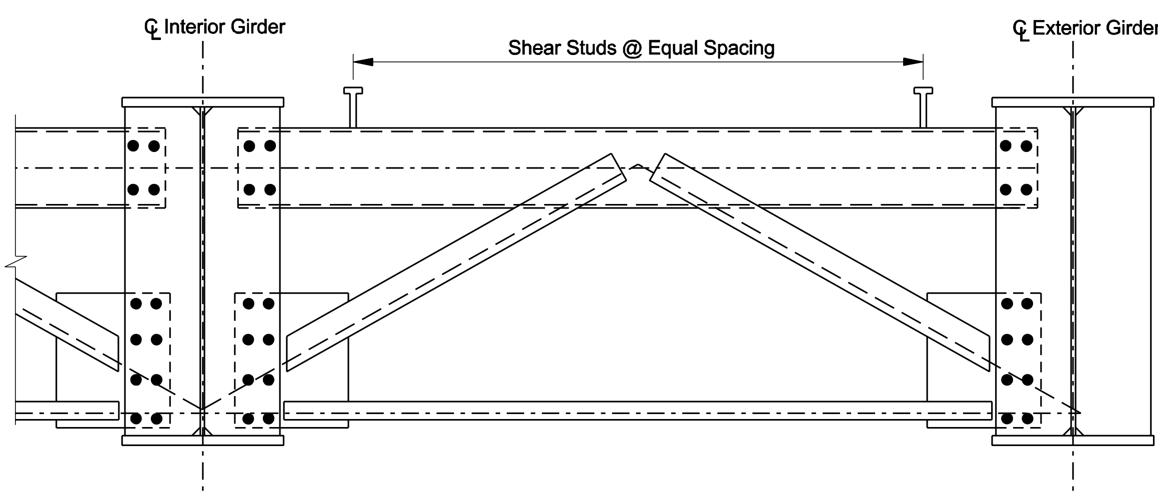
TYPICAL ABUTMENT DIAPHRAGM CONNECTION
(Rolled Beams)

Figure 15.5-B



TYPICAL PIER AND INTERMEDIATE CROSS FRAMES
(Plate Girder Web > 48 in)

Figure 15.5-C



TYPICAL ABUTMENT CROSS FRAMES
(Plate Girder Web > 48 in)

Figure 15.5-D

Pier and intermediate diaphragms for rolled-beam spans shall be detailed with a 3-in minimum clearance between the top of the diaphragm and the bottom of the top beam flange. For bridges having a normal roadway crown, the diaphragms shall be level. For bridges having a superelevated roadway, the diaphragms shall be placed parallel to the slab.

Intermediate diaphragms should be designed and detailed as non-load bearing. Diaphragms at points of support should be designed as a jacking frame as specified in Section 15.5.5.

15.5.4.3 Cross Frame Details

Figure 15.5-C illustrates typical pier and intermediate cross frame details for plate girder webs more than 48 in deep. In general, an X-frame is more cost effective than a K-frame; however, with a relatively wide girder spacing, the X-frame becomes shallow and less effective. The K-frame should be used instead of the X-frame when the girder spacing becomes much greater than the girder depth (e.g., where the girder spacing is greater than 1.75 of the girder depth) and the “X” becomes too shallow. A solid bent-plate diaphragm with a depth equal to 75% of the girder depth is a good option for plate girders less than 48 in deep.

Figure 15.5-D illustrates the typical abutment cross frame connection details for plate girder webs more than 48 in deep.

The rolled angles that comprise the cross frames are minimum sizes based upon the limiting slenderness ratios of LRFD Articles 6.8.4 and 6.9.3.

Current NDOT practice requires that cross frame transverse connection plates be welded or bolted to the compression flange and bolted to the tension flange. The welded and bolted connections to the flanges should be designed to transfer the cross frame forces into the flanges.

The width of connection plates should be sized to use bar stock and be not less than 5 in. When the connection plate also acts as a transverse stiffener, it shall meet the requirements of LRFD Article 6.10.8.1.

15.5.5 <u>Jacking</u>

Reference: LRFD Article 3.4.3

The contract documents shall include a jacking plan for all bearing supported structures. The bridge designer should include live load in the jacking plan for bridges with moderate to high traffic volumes or those with no readily available detour. Contact the District Office for concurrence on any jacking plan that does not include live load. The bearing type shall determine the level of detail shown for the jacking plan.

Include only bearing stiffeners at all points of jacking for plain or reinforced elastomeric bearings. Provide a conceptual jacking plan showing the jack location and clearances, required factored reactions and modifications to cross frames and diaphragms. Also, show conceptual requirements for falsework and jacking frames if required.

Include a complete jacking plan for high-load multi-rotational, isolation or other specialty bearings. The jacking plan must include necessary bearing stiffeners, jack locations and clearances, factored reactions and additional modifications to cross frames and diaphragms.

Also, include a detailed design of the jacking frame if required, but do not include its fabrication as part of the contract documents. Provide only conceptual falsework requirements.

In general, jacking frames will not be required at the supports unless there is insufficient clearance between the bottom of girder and top of cap to place a jack. If less than 7 in of clearance for the jack, the designer must decide whether the jack can be supported by temporary falsework. If temporary falsework is not feasible, provide details for a jacking frame or widen the cap and place the bearings on pedestals to provide sufficient space for a jack to be placed under the girder. Other locations where jacking may be required are:

- at supports under expansion joints where joint leakage could deteriorate the bearing areas of the girders; and

- at expansion bearings with large displacements where deformation-induced wear-and-tear is possible.

If no jacking frame is provided, the cross frame at the support must transfer lateral wind and seismic forces to the bearings.

15.5.6 Lateral Bracing

Reference: LRFD Article 6.7.5

The *LRFD Specifications* requires that the need for lateral bracing be investigated for all stages of assumed construction procedures. If the bracing is included in the structural model used to determine force effects, then it should be designed for all applicable limit states.

In general, lateral bracing is not required in the vast majority of steel I-girder bridges (short through medium spans); however, it must be checked by the designer. Typical diaphragms and cross frames will transfer lateral loads adequately to eliminate the need for lateral bracing. For tub girders, internal top lateral bracing is more typical. Tub girders can rack as much as 0.5 ft in one day due to the thermal effects of the sun. To counteract this effect, provide temporary lateral bracing between adjacent boxes at ¼ points of spans. Remove after the deck has been placed.

LRFD Article 4.6.2.7 provides various alternatives relative to lateral wind distribution in multi-girder bridges.

15.5.7 Inspection Access (Tub Girders)

All new steel tub girder bridges shall be detailed with access openings to allow inspection of the girder interior. Do not locate access openings over travel lanes or railroad tracks and, preferably, not over shoulders or maintenance roads. They should be located such that the general public cannot gain easy entrance.

Provide access openings in the bottom flange plate of all steel tub girders. Provide one access opening at each end of the bridge when the total span length is 100 ft or more.

Access plates shall be connected to the bottom flange with high-strength bolts. If the general public has access to the openings, provide bolts with special head configurations. Contact the Assistant Chief Structures Engineer – Inventory/Inspection for specifications on these special heads. The dimensions of the access opening should be a minimum 2 ft by 2 ft square.

15.6 I-SECTIONS IN FLEXURE

Reference: LRFD Article 6.10

15.6.1 General

Reference: LRFD Article 6.10.1

15.6.1.1 Positive-Moment Region Maximum-Moment Section

For a compositely designed girder, the positive-moment region maximum-moment section may also be considered compact in the final condition. The cured concrete deck in the positive-moment region provides a large compression flange and it laterally braces the top flange. Very little, if any, of the web is in compression.

15.6.1.1.1 *Top Flange*

In the final condition after the deck has cured, the top flange adds little to the resistance of the cross section. During curing of the concrete deck, however, the top flange is very important. The Strength limit state during construction when the concrete is not fully cured governs the design of the top flange in the positive-moment region as specified in LRFD Article 6.10.3.4.

15.6.1.1.2 *Bottom Flange (Tension Flange)*

The bottom flange, if properly proportioned, is not governed by the construction phase. The bottom flange is governed by the final condition. The Service II load combination permanent deformation provisions of LRFD Article 6.10.4.2 govern.

15.6.1.2 Negative-Moment Region Pier Section

The negative-moment region pier section will most likely be a non-compact section during all conditions. The concrete deck over the pier is in tension in the negative-moment region and, thus, considered cracked and ineffective at the nominal resistance (i.e., ultimate). Thus, a good portion of the steel cross section is in compression. To qualify as compact, the web usually needs to be too thick to be cost effective. Thus, the cost-effective section will typically be a non-compact section.

Both top and bottom flanges in the negative moment region are governed by the Strength limit state in the final condition. Furthermore, the bottom flange in compression is governed by the location of the first intermediate diaphragm off of the pier because it provides the discrete bracing for the flange.

15.6.1.3 Negative Flexural Deck Reinforcement

Reference: LRFD Article 6.10.1.7

In the negative-moment region where the longitudinal tensile stress in the slab, due to factored construction loads or the Service II load combination, exceeds the factored modulus of rupture, LRFD Article 6.10.1.7 specifies a minimum area of steel. The total cross sectional area of the longitudinal steel should not be less than 1% of the total cross sectional area of the deck slab (excluding the wearing surface) in these regions. However, the designer shall also ensure that sufficient negative-moment steel is provided for the applied loads.

15.6.1.4 Rigidity in Negative-Moment Regions

Reference: LRFD Articles 6.10.1.5 and 6.10.1.7

LRFD Article 6.10.1.5 permits the assumption of uncracked concrete in the negative-moment regions for member stiffness. This stiffness is used to obtain continuity moments due to live load, future wearing surface and barrier weights placed on the composite section.

For the Service limit state control of permanent deflections under LRFD Article 6.10.4.2 and the Fatigue limit state under LRFD Article 6.6.1.2, the concrete slab may be considered fully effective for both positive and negative moments for members with shear connectors throughout their full lengths and satisfying LRFD Article 6.10.1.7.

15.6.2 Shear Connectors

Reference: LRFD Article 6.10.10

The preferred size for shear studs for use on the flanges of girders and girders shall be ⅞ in diameter by 5 in; the minimum is ¾ in diameter by 5 in. The minimum number of studs in a group shall consist of three in a single transverse row. Skew the studs parallel to the bottom slab reinforcing steel. Increase the stud length in 1-in increments when necessary to maintain a 2-in minimum penetration of the stud into the deck slab. Studs placed on relatively thin elements such as girder webs should be detailed as ¾-in diameter.

15.6.3 Stiffeners

Reference: LRFD Article 6.10.11

15.6.3.1 Transverse Intermediate Stiffeners

Reference: LRFD Article 6.10.11.1

Straight girders may be designed without intermediate transverse stiffeners, if economical, or with intermediate transverse stiffeners placed on one side of the web plate. If stiffeners are required, fascia girders should only have stiffeners on the inside face of the web for aesthetics. Due to the labor intensity of welding stiffeners to the web, the unit cost of stiffener by weight is approximately nine times that of the unit cost of the web by weight. It is seldom economical to use the thinnest web plate permitted; therefore, the use of a thicker web and fewer intermediate transverse stiffeners, or no intermediate stiffeners at all, should be investigated. If the bridge designer proceeds with a design that requires stiffeners, the preferred width of the stiffener is one that can be cut from commercially produced bar stock.

Intermediate transverse stiffeners should be welded near side and far side to the compression flange. Transverse stiffeners should not be welded to tension flanges. The distance between the end of the web-to-stiffener weld and the near toe of the web-to-flange fillet weld should be between $4t_w$ and $6t_w$.

Transverse stiffeners, except when used as diaphragm or cross frame connections, should be placed on only one side of the web. The width of the projecting stiffener element, moment of inertia of the transverse stiffener and stiffener area shall satisfy the requirements of LRFD Article 6.10.11.1.

Orient transverse intermediate stiffeners normal to the web. However, where the angle of crossing is between 70° and 90°, the stiffeners may be skewed so that the diaphragms of cross frames may be connected directly to the stiffeners.

Longitudinal stiffeners should be avoided but, if used in conjunction with transverse stiffeners on spans with deeper webs, should preferably be placed on the opposite side of the web from the transverse stiffener. Where this is not practical (e.g., at intersections with cross frame connection plates), the longitudinal stiffener should be continuous and not be interrupted for the transverse stiffener.

15.6.3.2 Bearing Stiffeners

Reference: LRFD Article 6.10.11.2

Provide bearing stiffeners for all plate girders to prevent the possibility of web buckling at temporary supports. They only require placement on one side and, on the fascia girders, they shall be placed on the inside.

Bearing stiffeners are required at the bearing points of rolled beams and plate girders. Bearing stiffeners at integral abutments may be designed for dead and construction loads only.

Design the bearing stiffeners as columns and extend the stiffeners to the outer edges of the bottom flange plates. The *LRFD Specifications* does not specify an effective column length for the design of bearing stiffeners. Because the reaction load applied at one end of the stiffener pair is resisted by forces distributed to the web instead of by a force concentrated at the opposite end, as in columns, it is not necessary to consider the stiffeners as an end-hinged column even where the flanges are free to rotate. Use an effective column length of ¾ of the web depth.

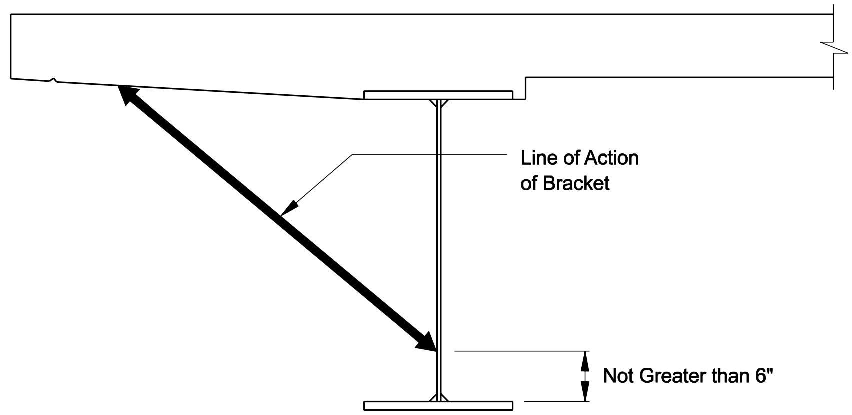
The weld connecting the bearing stiffener to the web should be designed to transmit the full bearing force from the stiffener to the web due to the factored loads.

Detail bearing stiffeners with the stiffener ends bearing on the loaded flange being milled to bear, or weld with a full penetration butt weld. The opposite end will be tight fit only to the flange. Where bearing stiffeners are also used as diaphragm or cross frame connection plates, the stiffeners shall also be fillet welded to the girder flanges if they are milled to bear or tight fit.

15.6.4 <u>Deck-Overhang Cantilever Brackets</u>

Reference: LRFD Article 6.10.3

During construction, the deck overhang brackets may induce twist in the exterior girder. Include in the contract documents the requirement for the contractor to check the twist of the exterior girder and bearing of the overhand bracket on the web. See Figure 15.6-A.



SCHEMATIC OF LOCATION FOR DECK OVERHANG BRACKET

Figure 15.6-A

15.7 CONNECTIONS AND SPLICES

Reference: LRFD Article 6.13

15.7.1 Bolted Connections

Reference: LRFD Article 6.13.2

The following applies to bolted connections:

1. Type. For painted steel, ⅞-in A325 (Type1) bolts should be used. For unpainted weathering steel, A325 (Type 3) bolts should be used.

2. Design. Design all bolted connections as slip-critical at the Service II limit state, except for secondary bracing members.

3. Slip Resistance. LRFD Table 6.13.2.8-3 provides values for the surface condition. Use Class B surface condition for the design of slip-critical connections. Class B is applicable to unpainted, blast-cleaned surfaces and to blast-cleaned surfaces with a Class B coating. All specified coatings must be tested to ensure a slip resistance equal to or exceeding Class B. NDOT policy is to paint the faying surfaces of all slip critical connections with the prime coat of the approved paint systems shown in the Qualified Products List (QPL). Systems on the QPL must meet the minimum requirements of the Research Council on Structural Connections, June 2004 version of "Testing Method to Determine the Slip Coefficient for Coatings Used in Bolted Joints."

15.7.2 Welded Connections

Reference: LRFD Article 6.13.3

15.7.2.1 Welding Process

The governing specification for welding is the ANSI/AASHTO/AWS *Bridge Welding Code D1.5*. However, this specification does not provide control over all of the welding issues that may arise on a project. Additional reference specifications that may need to be consulted are:

- AWS D1.1 for welding of tubular members and strengthening or repair of existing structures, and

- AWS D1.4 if the welding of reinforcing steel must be covered by a specification.

The *Bridge Welding Code* accepts as *prequalified* (i.e., acceptable without further proof of suitability if applied under specified conditions) four electric arc welding processes:

- shielded metal arc welding (SMAW). This process is also known as stick welding and is what is commonly considered "welding";

- submerged arc welding (SAW);

- gas metal arc welding (GMAW). This process is also called metal inert gas welding or MIG; and
- flux-cored arc welding (FCAW).

Gas metal arc and flux-cored arc welding shall not be used except with written approval of the Chief Structures Engineer. Electro-slag welding and electro-gas welding (a development of electro-slag welding) shall not be permitted due to potential problems with the ductility of electro-slag welds.

GMAW should not be used in windy conditions due to the potential loss of shielding gas. The *Welding Code* states that “GMAW, GTAW, EGW or FCAW-G shall not be done in a draft or wind unless the weld is protected by a shelter. Such shelter shall be of material and shape appropriate to reduce wind velocity in the vicinity of the weld to a maximum of five miles per hour” (AWS D1.1-2000, para. 5.12.1).

SMAW is the principal method for hand welding; the others are automatic or semi-automatic processes. Shop practice on most weldments is automatic, offering the advantages of much higher speed and greater reliability. Hand welding is mostly limited to short production welds or tack welds during the fitting up of components prior to production welding.

Acceptable procedures for using these processes or others require the testing of the welding operations and of welds, using a filler metal that is compatible with the base metal, proper preparation of the joints, controlling the temperature and rate of welding, and control of the welding process.

15.7.2.2 Field Welding

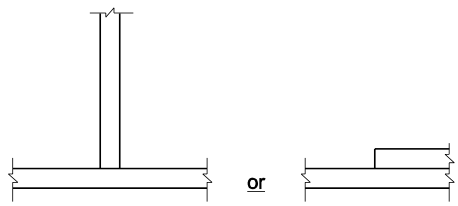
The primary types of welds used in bridge fabrication are fillet welds and groove welds. Figure 15.7-A illustrates a typical cross section where specification of a fillet weld is appropriate, and Figure 15.7-B illustrates a typical cross section where specification of a groove or butt weld is appropriate.

Field welding is prohibited for all but a few special applications. These permissible applications are welded splices for piles, connecting pile tips to piles, bearing plates to bottom flange plates and connector plates between new and existing portions of widened bridges at ends of simply supported spans (though bolted connections are preferred for this application).

Direct welding of stay-in-place (SIP) deck forms to girder flanges is not permitted. Metal forms are welded to a strap that is placed over the flange.

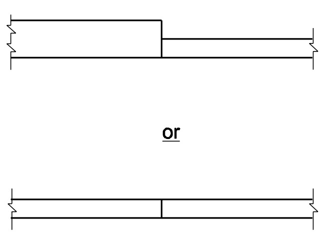
15.7.2.3 Welding Symbols

Welding symbols are used as an instruction on the type, size and other characteristics of the desired weld. The forms of the symbols are precisely defined by AWS A2.4. When these symbols are properly used, the meaning is clear and unambiguous. If not used exactly as prescribed, the meaning may be ambiguous, leading to problems. The *AISC Manual* and most steel design textbooks have examples of welding symbols that, although technically correct, are more complicated than the typical bridge designer needs. With minor modifications, the examples in Figure 15.7-C will suffice for the majority of bridge fabrication circumstances.



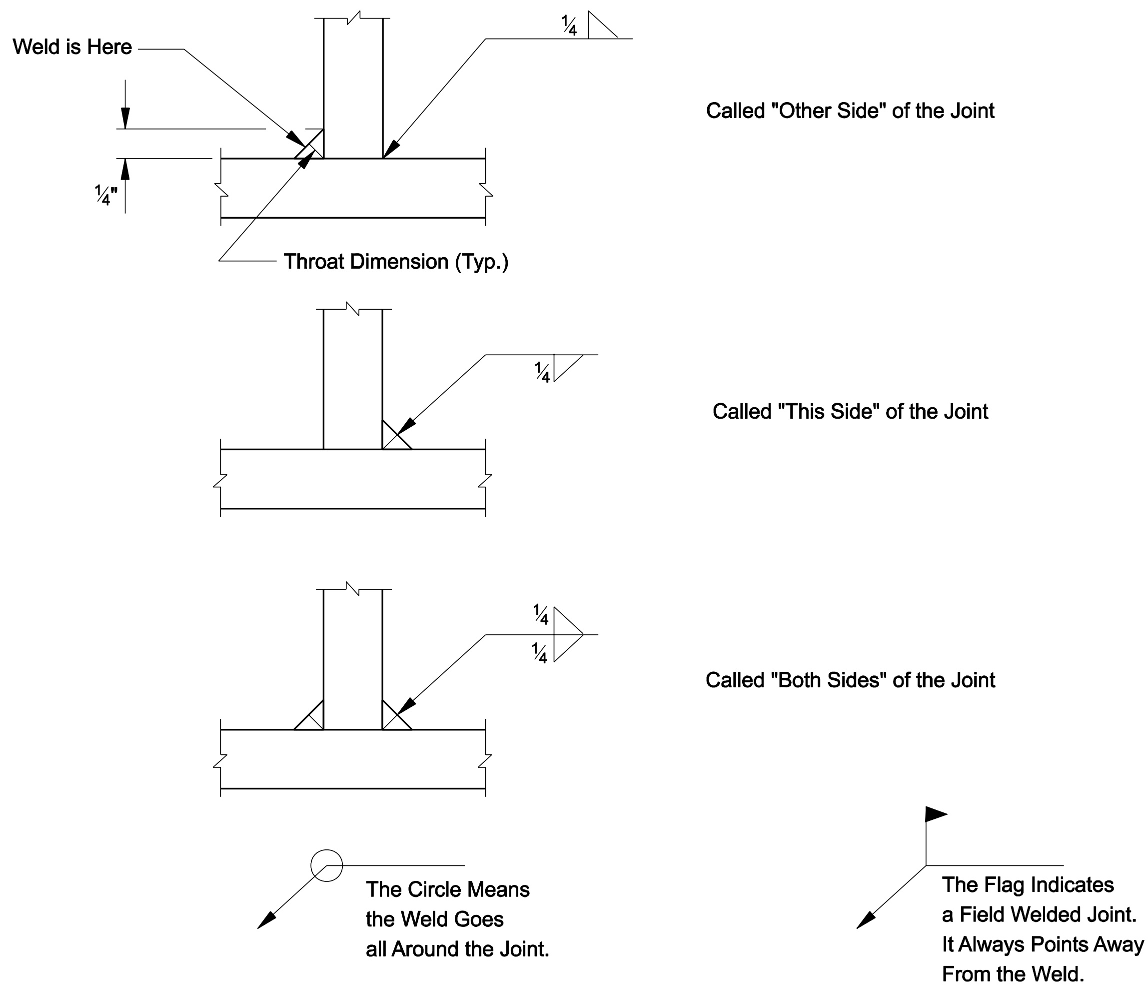
JOINT WITH FILLET WELD

Figure 15.7-A



JOINT WITH GROOVE OR BUTT WELD

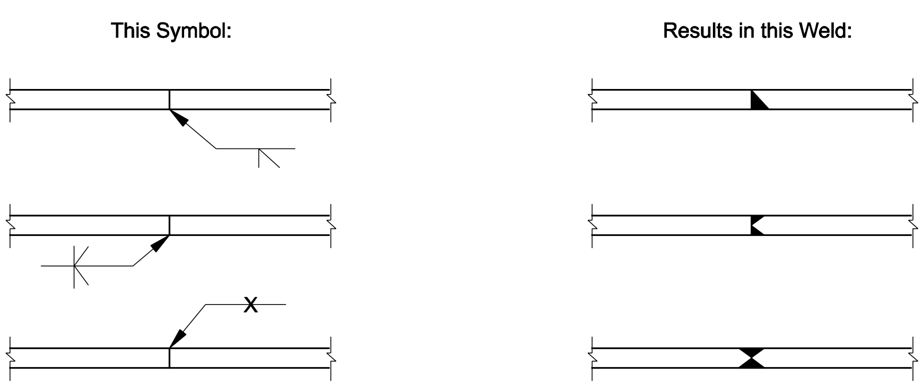
Figure 15.7-B



"This Side" and "Other Side" Welds are the Same Size Unless Specified Otherwise.

Symbols Apply Between Abrupt Changes in Direction of Welding
Unless Governed by the "All-Around Symbol" or Otherwise Dimensioned.

<u>FILLET WELDS</u>



<u>BUTT WELDS</u>

WELDING SYMBOLS

Figure 15.7-C

15.7.2.4 Weld-Metal Strength and Electrode Nomenclature

The strength of the weld filler metal is known from the electrode designation. Figure 15.7-D illustrates the standard nomenclature to identify electrodes. The figure represents more than a bridge designer typically needs to know but, as an illustration, a typical pile weld note may say use E7018 or E7028 series electrodes. This means that electrodes with a weld-metal strength of 70,000 psi and the indicated welding procedures for all positions of welding or only flat and horizontal positions, respectively.

To make a weld of sufficient strength, the designer must consider three variables: weld length, weld throat and weld-metal strength. Because weld strength is a function of these three variables, many possible combinations can yield sufficient weld strength. Full-penetration groove welds in tension require matched welds because the length and throat of the weld also match the dimensions of the base metal. Fillet welds joining higher strength steels are a good application of undermatching. Undermatching weld metal (i.e., specifying a weld-metal strength less than the base metal) can be an attractive alternative when joining higher strength steels (e.g., F_y = 70 ksi and 100 ksi) because undermatching can decrease distortion, residual stresses and cracking tendencies.

15.7.2.5 Design of Welds

The design of fillet welds is integral to LRFD Section 6 on Steel Design. The *LRFD Specifications* addresses topics such as resistance factors for welds, minimum weld size and weld details to reduce fatigue susceptibility.

The weld-strength calculations of LRFD Section 6 assume that the strength of a welded connection is dependent only on the weld metal strength and the area of the weld. Weld metal strength is a fairly self-defining term. The area of the weld that resists load is a product of the theoretical throat multiplied by the length. The theoretical weld throat is the minimum distance from the root of the weld to its theoretical face. See Figure 15.7-C. Fillet welds resist load through shear on the throat, while groove welds resist load through tension, compression or shear depending upon the application.

Often, it is best to only show the type and size of the weld required and leave the details to the fabricator.

When considering design options, note that the most significant factor in the cost of a weld is the volume of the weld material that is deposited. Over specifying a welded joint is unnecessary and uneconomical. A single-pass weld is one made by laying a single weld bead in a single move of the welder along the joint. A multiple-pass weld is one in which several beads are laid one upon the other in multiple moves along the joint. Welds sized to be made in a single pass are preferred because these are most economical and least susceptible to resultant flaws. The maximum weld size for a single-pass fillet weld applicable to all weld types is 5/16 in. The AWS D1.1 *Structural Welding Code*, Table 3.7 provides more specific maximum single-pass fillet-weld sizes for various welding processes and positions of welding. The weld should be designed economically, but its size should not be less than ¼ in and, in no case, less than the requirements of LRFD Article 6.13.3.4 for the thicker of the two parts joined. Show the weld terminations.

These digits indicate the following:	
Exx1z	All positions of welding
Exx2z	Flat and horizontal positions
Exx3z	Flat welding positions only
These digits indicate the following:	
Exx10	DC, reverse polarity
Exx11	AC or DC, reverse polarity
Exx12	DC straight polarity, or AC
Exx13	AC or DC, straight polarity
Exx14	DC, either polarity or AC, iron powder
Exx15	DC, reverse polarity, low hydrogen
Exx16	AC or DC, reverse polarity, low hydrogen
Exx18	AC or DC, reverse polarity, iron powder, low hydrogen
Exx20	DC, either polarity, or AC for horizontal fillet welds; and DC either polarity, or AC for flat position welding
Exx24	DC, either polarity, or AC, iron powder
Exx27	DC, straight polarity, or AC for horizontal fillet welding; and DC, either polarity, or AC for flat position welding, iron powder
Exx28	AC or DC, reverse polarity, iron powder, low hydrogen

The “xx” shown above is a two-digit number indicating the weld metal tensile strength in 1000 psi increments. For example, E7018 is 70,000 psi.

ELECTRODE NOMENCLATURE

Figure 15.7-D

The following types of welds are prohibited:

- field-welded splices,
- intersecting welds,
- intermittent fillet welds (except for the connection of stop bars at expansion joints), and
- partial penetration groove welds (except for the connection of tubular members in hand rails).

Provide careful attention to the accessibility of welded joints. Provide sufficient clearance to enable a welding rod to be placed at the joint. Often, a large-scale sketch or an isometric drawing of the joint will reveal difficulties in welding or where critical weld stresses must be investigated.

15.7.2.6 Inspection and Testing

Indispensable to the reliable use of welding is a systematic program of inspection and testing. Inspection is done at the shop and at the field site. The function of the inspection is to guarantee that specified materials and procedures are used under conditions where proper welding is possible. If the sequence of welding has been specified, the inspector should be able to certify conformance.

Despite careful inspection, weld defects may escape detection unless all or part of the work is subjected to tests. There are two broad categories of testing — destructive testing, which is used very sparingly for big problems or forensic studies, and nondestructive testing, which is used extensively to guarantee the quality of the welds. NDOT uses the following types of non-destructive testing (NDT):

1. Radiographic Testing (RT). Used to find cracks and inclusions after a weld is completed. The process involves placing film on one side of the weld and a source of gamma or x-rays on the other side of the weld. Shadows on the exposed film indicate cracks or inclusions in the welds or adjacent areas. RT is most effective on full-penetration groove joints with ready access to both sides.

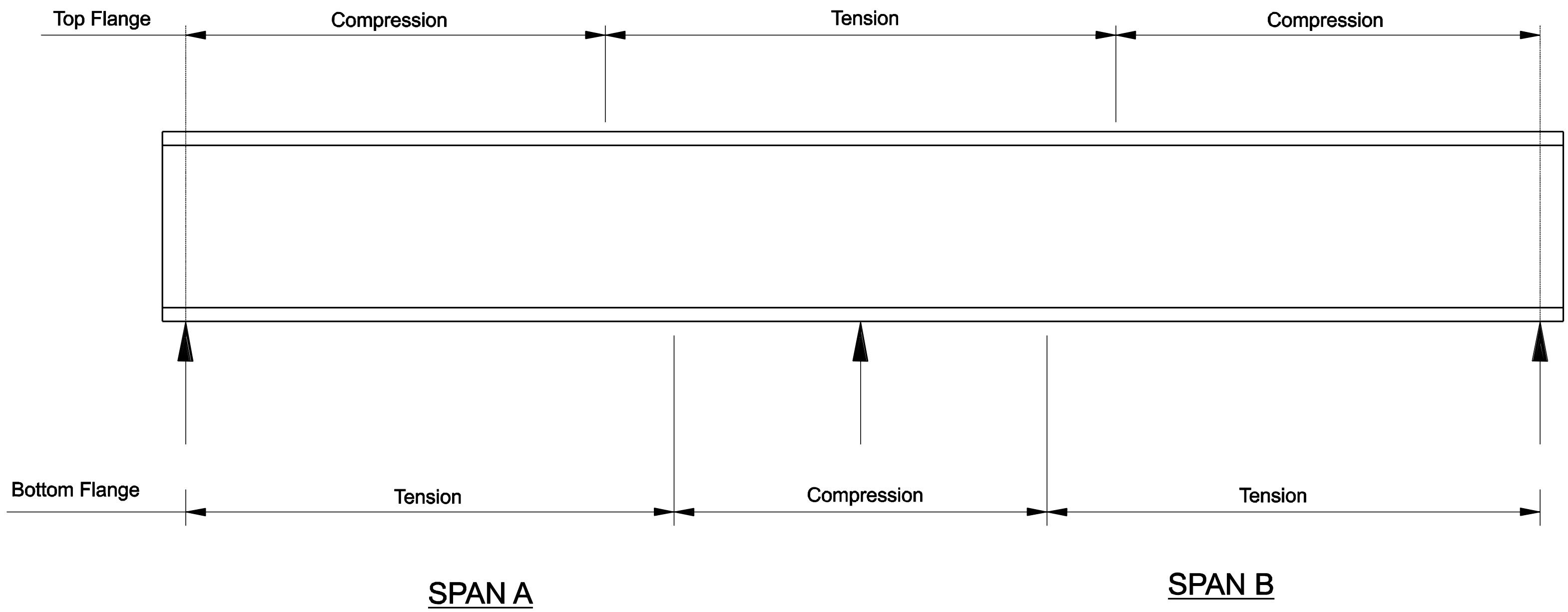
2. Ultrasonic Testing (UT). Relies on the reflection patterns of high-frequency sound waves, which are transmitted at an angle through the work. Cracks and defects interrupt the sound transmission, altering the display on an oscilloscope. UT can reveal many defects that the other methods do not, but it relies very heavily on the interpretative skill of the operator. UT is used on full-penetration groove welds.

3. Magnetic Particle Testing (MT). Performed by covering the surface of a weld with a suspension of ferromagnetic particles and then applying a strong magnetic field. Cracks in the weld interrupt the magnetic force lines, causing the particles to concentrate in the vicinity of the crack in patterns easily interpreted by the inspector. MT is used on fillet welds.

4. Dye Penetrant Testing (DP). Uses a dye in liquid form to detect cracks. Capillary tension in the liquid causes the dye to penetrate into the crack, remaining behind after the surface is cleaned. DP is used to locate surface flaws in and around fillet welds.

5. Eddy Current Testing (ET). Eddy Current testing uses a phenomenon called electromagnetic induction to detect flaws in conductive materials. This form of testing detects flux leakage emanating from a discontinuity in metal when an eddy current is passed through the material. Eddy Current testing can detect very small flaws in or near the surface of the material, the surfaces need minimal preparation, and physically complex geometries can be quickly investigated.

To aid the inspector, the contract documents for continuous structures shall include a sketch showing the location of tension regions along both the top and bottom girder flanges. Show the length of each stress region and reference these regions to the point of support. Figure 15.7-E illustrates the information required.



SCHEMATIC OF FLANGE TENSION REGIONS

Figure 15.7-E

15.7.3 Splices

Reference: LRFD Article 6.13.6

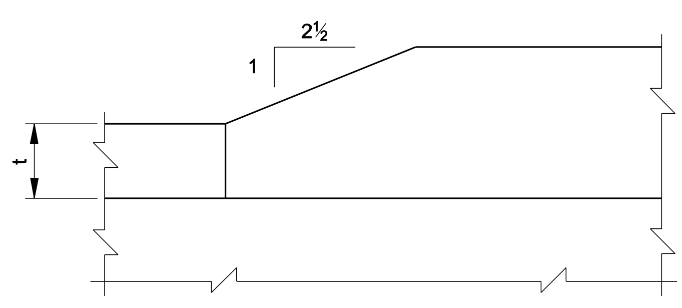
15.7.3.1 Shop Splices

In addition to the provisions of LRFD Article 6.13.6, the following will apply to splices:

1. Location. Numerous groove welds and/or groove welds located in high stress regions are not desirable. Locate flange shop splices away from high moment regions and web splices away from high shear regions. This is simple for flange splices in negative moment regions but more difficult with positive moment regions. In positive moment areas, the magnitude of moment does not change quickly along the girder compared to the negative moment. As such, shop splices on longer span bridges must be located in fairly high positive moment regions.

   The location of shop groove splices is normally dependent upon the length of plate available to the fabricator. This length varies depending upon the rolling process. The maximum length of plates that are normalized, quenched and tempered (70 HPS) is 50 ft. Other plates (e.g., 36 and 50) can be obtained in lengths greater than 80 ft depending on thickness. The cost of adding a shop-welded splice instead of extending a thicker plate should be considered when designing members. Discussions with a fabricator or the NSBA during the design is suggested.

2. Welded Shop Splice. Figure 15.7-F illustrates welded flange splice details. At flange splices, the thinner plate should not be less than one-half the thickness of the thicker plate. See LRFD Article 6.13.6.2 for more information on splicing different thicknesses of material using butt welds.



FLANGE SPLICE DETAILS

TYPICAL WELDED SPLICE DETAILS

Figure 15.7-F

15.7.3.2 Field Splices

In addition to the provisions of LRFD Article 6.13.6, the following will apply to field splices:

1. Location. In general, field splices in main girders should be located at low-stress areas and near the points of dead-load contraflexure for continuous spans. Long spans may require that field splices be located in high moment areas.

2. Bolts. Design loads for bolts shall be calculated by an elastic method of analysis. Provide at least two lines of bolts on each side of the web splice.

3. Composite Girder. If a compositely designed girder is spliced at a section where the moment can be resisted without composite action, the splice may be designed as noncomposite. If composite action is necessary to resist the loads, the splice should be designed for the forces due to composite action.

4. Design. Bolted splices must be designed to satisfy both the slip-critical criteria under Service II loads and the bearing-type connection criteria under the appropriate Strength limit states.

5. Swept Width (or shipping width) for Curved Girders. The swept width is the horizontal sweep in a curved girder plus its flange width. Field splices should be located such that the maximum swept width for a horizontally curved girder is 10 ft within a single field section.

Chapter 16

BRIDGE DECKS

NDOT STRUCTURES MANUAL

September 2008

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Chapter 16
BRIDGE DECKS

Sections 3, 4 and 9 of the *LRFD Bridge Design Specifications* present the AASHTO criteria for the structural design of bridge decks. Section 3 specifies loads for bridge decks, Section 4 specifies their analyses, and Section 9 specifies the resistance of bridge decks. Unless noted otherwise in this Chapter of the *NDOT Structures Manual*, the *LRFD Specifications* applies to the design of bridge decks in Nevada.

This Chapter documents NDOT criteria on the design of bridge decks that are constructed compositely in conjunction with concrete and steel girders and top slabs of cast-in-place, post-tensioned box girders. Chapter 14 discusses the design of CIP concrete slabs.

16.1 GENERAL

16.1.1 Protection of Reinforcing Steel

Reference: LRFD Articles 2.5.2.1 and 5.12

NDOT typical practice is to use epoxy-coated reinforcing steel (except in Clark County) for all reinforcing within 12 in of the riding surface. This includes both layers of deck reinforcing and all reinforcing extending into the deck from precast and cast-in-place construction. The epoxy-coated reinforcing steel is combined with a minimum cover of 2½ in from the top surface of the deck to the top layer. In addition, all concrete for deck slabs, approach slabs and barrier rails shall use a high-performance concrete having a low water/cement ratio and low permeability. A variety of other methods are available to protect the reinforcing steel in new decks and to retard the rate of corrosion. NDOT occasionally uses some of these other methods to protect reinforcing steel in new bridge decks.

16.1.2 Traditional Design Using the "Strip Method"

Reference: LRFD Articles 9.7.3, 4.6.2.1.1, 4.6.2.1.3 and Appendix A4

The application of the strip method to concrete decks is based on the design table for deck slabs in the Appendix to Section 4 of the *LRFD Specifications* (LRFD Table A4-1). An introduction to the LRFD Table discusses its application.

LRFD Table A4-1 shall be used to design the concrete deck reinforcement. LRFD Table A4-1 tabulates the resultant live-load moments per unit width for slab steel design as a function of the girder or web spacings, S. The Table distinguishes between negative moments and positive moments and tabulates these for various design sections as a function of the distance from the girder or web centerline to the design section. LRFD Article 4.6.2.1.6 specifies the design sections to be used. NDOT design practice is to use a 40-kip axle instead of the 32-kip axle specified in the *LRFD Specifications*. Therefore, the bridge designer must multiply the design moments shown in LRFD Table A4-1 by 1.25.

16.1.3 Empirical Design

Reference: LRFD Article 9.7.2

NDOT prohibits the use of the empirical deck design methodology.

16.2 DESIGN DETAILS FOR BRIDGE DECKS

16.2.1 General

The following general criteria apply to bridge decks that are constructed compositely in conjunction with concrete girders, steel girders and the top slabs of cast-in-place, post-tensioned box girders:

1. Thickness. The thickness of reinforced concrete decks shall typically be 8 in but not less than 7½ in for all girder-type bridges. The composite deck over precast side-by-side box girders shall be a 5½-in minimum CIP reinforced concrete slab.

2. Reinforcing Steel Strength. The specified yield strength of reinforcing steel shall be 60 ksi.

3. Exposure Condition. Use a Class 2 exposure factor in LRFD Equation 5.7.3.4-1 for all bridge decks.

4. Reinforcement Cover. The bottom reinforcement cover shall be a minimum of 1½ in. The top reinforcement cover shall be a minimum of 2½ in, which includes a ½-in sacrificial wearing surface. The primary reinforcement in the top and bottom mats shall be the closer reinforcement to the concrete face. See Figure 14.3-B for additional concrete cover criteria.

5. Placement of Top and Bottom Transverse Reinforcing Steel. The top and bottom transverse reinforcing steel shall be offset, preferable at half the spacing, so that the top mat is not placed directly above the bottom mat.

6. Reinforcing Steel Spacing. Maintain a minimum of 1½ in vertical separation between the top and bottom reinforcing mats. Where conduits are present between mats, the 1½ in must be increased. Maintain a minimum horizontal spacing of 5 in on center (with 6 in preferred) between adjacent bars within each mat. The maximum horizontal reinforcing steel spacing is 8 in for primary (transverse) steel. See Figure 14.3-C for additional information on reinforcing steel spacing.

7. Reinforcing Bar Size. The minimum reinforcing steel size used for bridge deck reinforcement is a #4 bar.

8. Sacrificial Wearing Surface. The 2½-in top reinforcement concrete cover includes ½ in that is considered sacrificial. For both the deck and superstructure, its weight shall be included as a dead load, but its structural contribution shall not be included in the structural design.

9. Concrete Strength. The minimum specified 28-day compressive strength of concrete for bridge decks shall be 4.0 ksi in Clark County and 4.5 ksi elsewhere in the State.

10. Length of Reinforcement Steel. For detailing, the maximum length of reinforcing steel in the deck shall be 60 ft.

11. Placement of Transverse Reinforcing Steel on Skewed Bridges. The following applies:

    a. Skews $\leq$ 20°: Place the transverse reinforcing steel parallel to the skew.

b. Skews > 20°: Place the transverse reinforcing steel perpendicular to the longitudinal reinforcement.

See Section 16.2.4 for a definition of skew angle and for structural considerations related to skewed reinforcing steel placement.

12. Splices/Connectors. Use lap splices for deck reinforcement unless special circumstances exist. Mechanical connectors may be used where clearance problems exist or on a phased-construction project that precludes the use of lap splices. See Section 14.3.1.8 for more discussion on splices.

    Transverse slab reinforcement should be lapped, if necessary, as follows: Negative moment steel in the positive-moment region between the slab supports and positive moment steel in the negative-moment region over the slab supports.

13. Shear Connectors For Concrete Girder Bridges. Stirrups shall project from the girders into the slab to provide a composite section. Detail bars to hook around longitudinal deck reinforcement.

14. Post-Tensioning. Post-tensioning is not normally used in cast-in-place concrete decks. It can be used where spans exceed 14 ft and as approved by the Chief Structures Engineer.

16.2.2 Detailing Requirements for Concrete-Deck Haunches

16.2.2.1 General

Haunches, concrete between the top of a steel flange or concrete girder and the bottom of the bridge deck, are provided to account for construction variations and tolerances. The haunch varies across the width of the flange due to cross slope, the length of the girder due to flange thickness, camber variation and profile. In all cases, however, there shall be a minimum of a ½-in haunch.

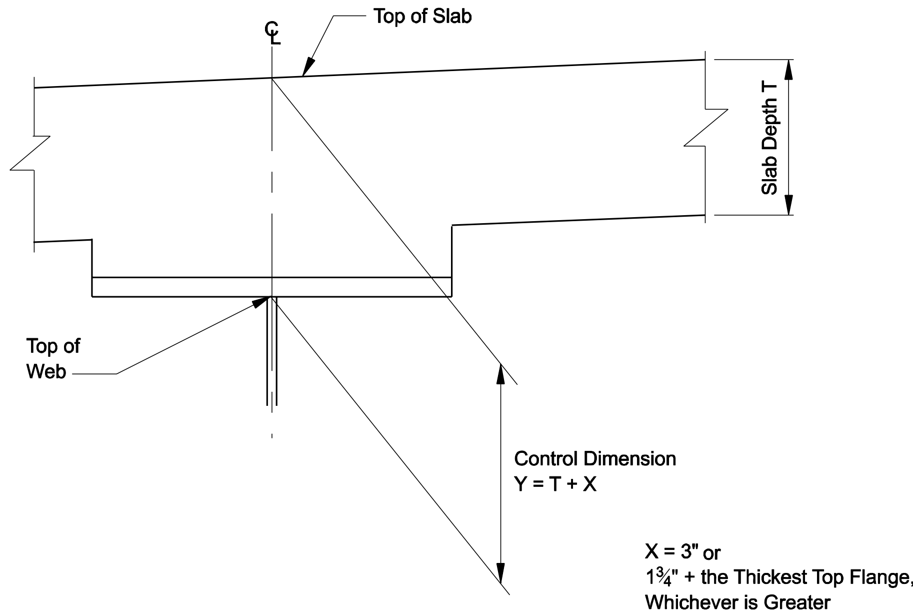
The girder haunch should be included in the load calculations as dead load by applying the maximum haunch dimension throughout the span. The haunch, however, should be ignored in the calculation of the section's resistance.

The Control Dimension "Y" is measured at the centerline of bearing and varies along the span to compensate for variations in camber and superelevation ordinate. In some cases where vertical curve corrections are small, the vertical curve ordinate can be accommodated in the haunch without including it in the girder.

The haunch should be detailed flush with the vertical edge of the top flange.

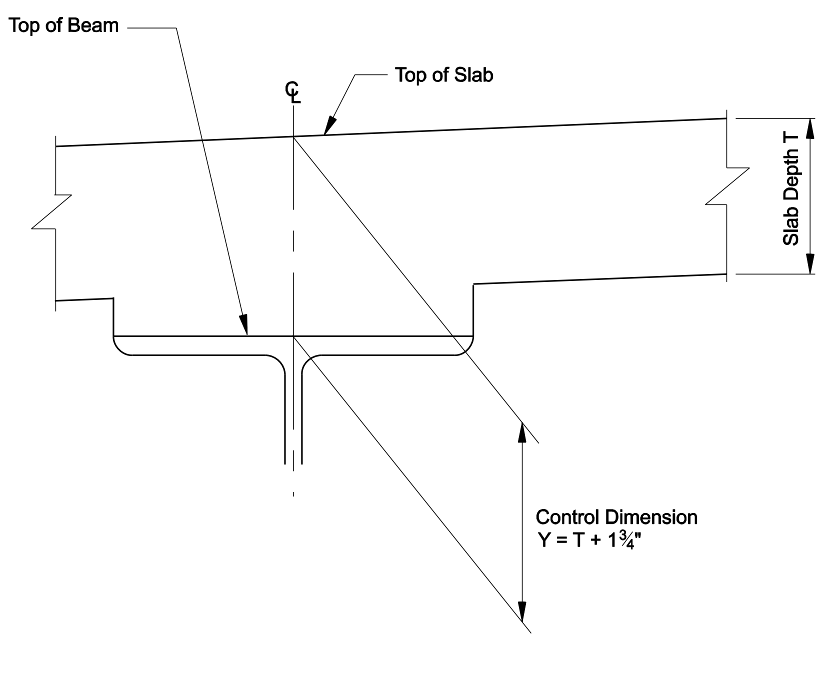
16.2.2.2 Haunch Dimensions for Steel Girders

Figure 16.2-A illustrates the controlling factors used to determine the haunch dimension for steel plate girders. Figure 16.2-B illustrates a steel rolled beam. For plate girders, the control dimension "Y" is the deck thickness "T" plus a dimension "X". "X" is the greater of 1¾ in plus the thickest top flange or 3 in. The 1¾ in dimension represents the maximum positive camber fabrication tolerance allowed by AWS D-1.5 of 1½ in plus a moderate deck cross slope. For rolled beams, the control dimension "Y" includes the deck thickness "T" plus 1¾ in.



HAUNCH DIMENSION FOR STEEL PLATE GIRDERS

Figure 16.2-A



HAUNCH DIMENSION FOR STEEL ROLLED BEAMS

Figure 16.2-B

16.2.2.3 Haunch Dimensions for Precast Concrete Girders

Figure 16.2-C illustrates the controlling factors used to determine the haunch dimension for precast concrete girders. Control dimension "Y" is the deck thickness "T" plus 3 in. The 3-in dimension is used to account for camber growth in the girder at the center of span. The amount of camber growth can vary even between girders cast at the same time.

16.2.2.4 Reinforcement for Deep Haunches

Provide additional reinforcement in haunches greater than 4 in deep. The additional reinforcement shall consist of a minimum of #4 U-shaped reinforcing bars spaced at a maximum of 12 in. These reinforcing bars shall be properly developed into the bridge deck.

16.2.3 Stay-in-Place Forms

Steel stay-in-place forms are typically used with steel I-girders. Although not typical, steel stay-in-place forms can be used with precast concrete girders. Stay-in-place forms are not allowed in bays having longitudinal joints nor in deck overhangs.

Design loads for stay-in-place forms shall be applied for all girder bridges and consist of 0.015 ksf for the metal forms and form corrugation fill applied over the areas of the forms. If the Contractor elects not to use stay-in-place forms, the bridge designer should consider revising the camber calculations. NDOT prohibits field welding of the stay-in-place forms to steel flanges.

16.2.4 Skewed Decks

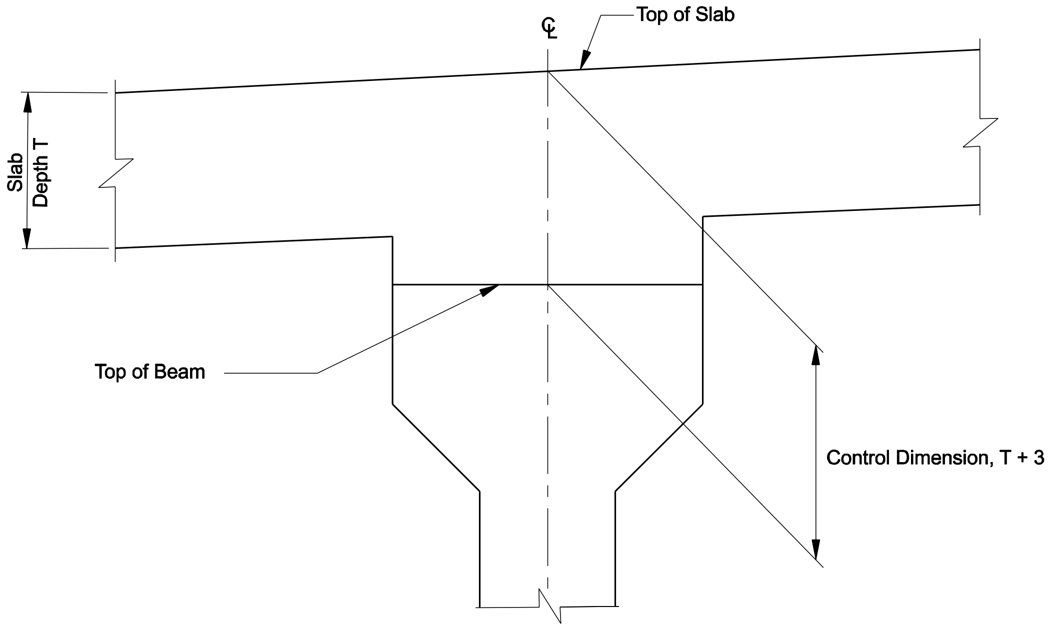
Reference: LRFD Article 9.7.1.3

Skew is defined by the angle between the centerline of support and the normal drawn to the longitudinal centerline of the bridge at that point. See Figure 16.2-D. The support skews can be different. In addition to skew, the behavior of the superstructure is also affected by the span-length-to-bridge-width ratio.

The *LRFD Specifications* suggests that the effects of skew angles not exceeding 25° can be neglected for concrete decks, but the *LRFD Specifications* assumes the typical case of bridges with relatively large span-length-to-bridge-width ratios. Further, the Commentary indicates that the 25° limit is "somewhat arbitrary." Therefore, the traditional NDOT 20° threshold is acceptable for use.

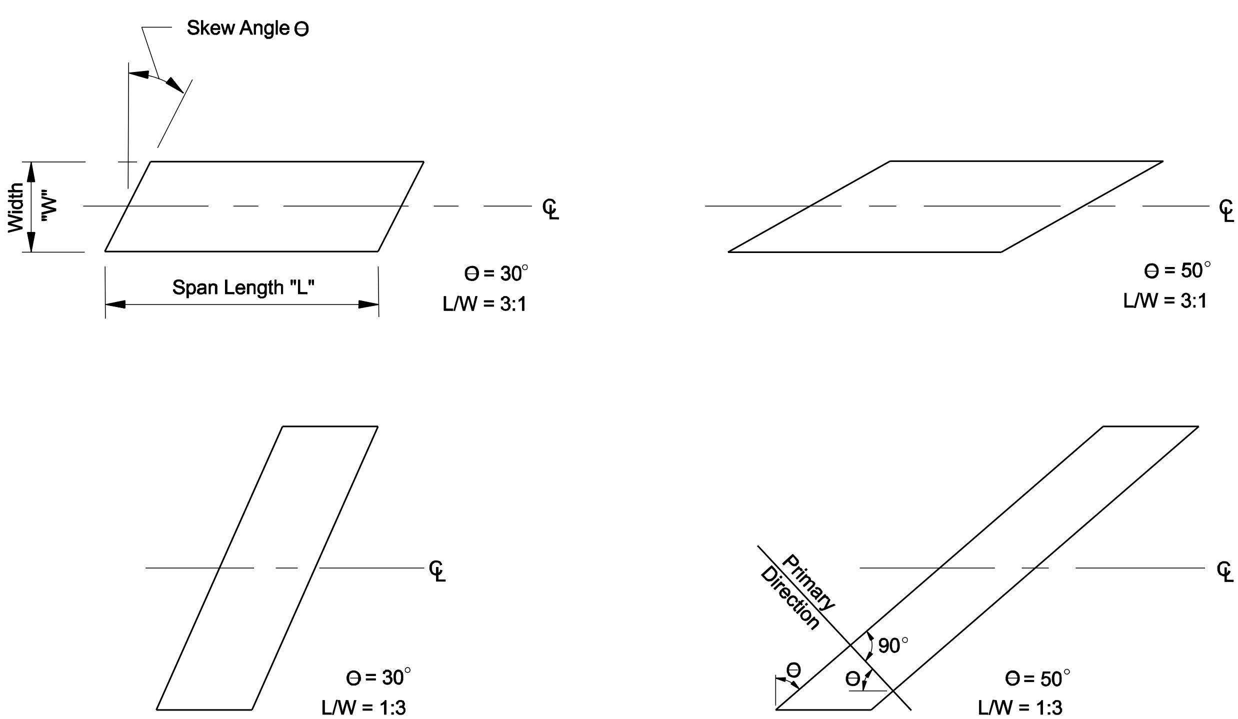
Figure 16.2-D illustrates four combinations of skew angles 30° and 50° and length-to-width ratios of 3:1 and 1:3. Both the 50° skew and the 1:3 length-to-width ratio may be considered extreme values for bridges, but this often occurs where the deck constitutes the top slab of a box culvert.

Both combinations with 30° skew may be orthogonally modeled for design with the skew ignored.



HAUNCH DIMENSION FOR CONCRETE I-GIRDERS

Figure 16.2-C



SKEW ANGLE AND LENGTH/BRIDGE WIDTH RATIOS

Figure 16.2-D

The combinations with 50° skew may require additional thought. Consider, for example, the combination of 50° skew and L/W = 1:3. If the deck is a cast-in-place concrete slab without girders, the primary direction of structural action is perpendicular to the span not in the direction of the span. In this case, consider running the primary reinforcement in that direction and fanning it as appropriate in the side zone. With this arrangement, the secondary reinforcement could then be run parallel to the skew, thus regaining the orthogonality of the reinforcement as appropriate for this layout.

16.2.5 Deck Pouring Sequence for Bridge Decks that are Constructed Compositely in Conjunction with Concrete And Steel Girders

Reference: LRFD Article 2.5.3

16.2.5.1 NDOT Typical Practice

The designer determines the need for a bridge deck pouring sequence based on factors such as size of pour, configuration of the bridge, potential placement restrictions, direction of placement, deck tensile stresses and any other special circumstances that might affect the bridge deck placement. In addition, provide a deck pouring schedule for bridges that have any of the following features:

- continuous bridges,
- bridges with curved or non-parallel deck edges, or
- wide or long single span bridges.

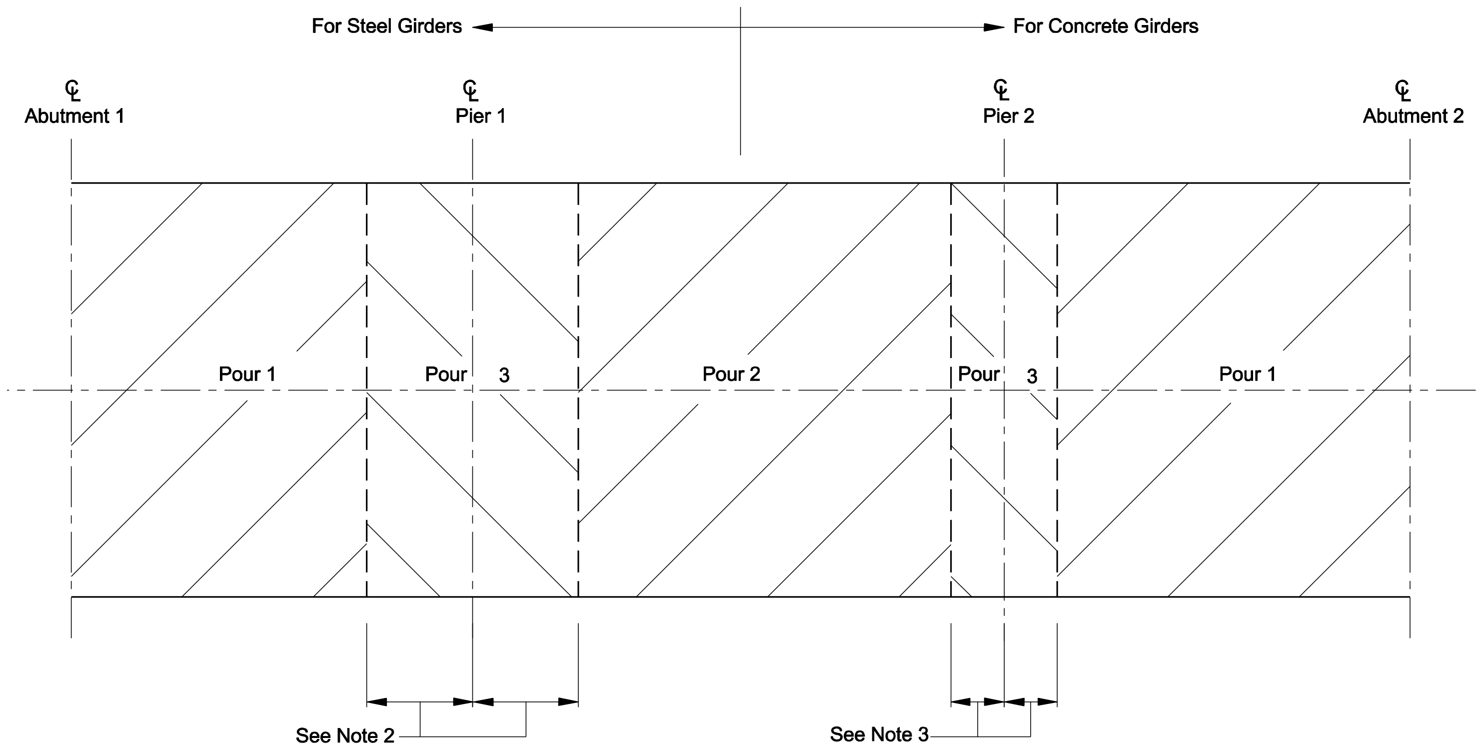
Where required, the bridge designer will present in the contract documents the sequence of placing concrete in various sections (separated by transverse construction joints) of deck slabs on continuous spans. The designated sequence should avoid or minimize the dead-load tensile stresses in the slab during concrete setting to minimize cracking, and the sequence should be arranged to cause the least disturbance to the portions placed previously. In addition, for longer span steel girder bridges, the pouring sequence can lock-in stresses far different than those associated with the instantaneous placement typically assumed in design. Therefore, for these bridges, the designer shall consider the pouring sequence in the design of the girders.

Deck placement shall be uniform and continuous over the full width of the superstructure. The first pours shall include the positive-moment regions in all spans. For all deck pours on a longitudinal gradient of 3% or greater, the direction of pouring should be uphill.

Figure 16.2-E illustrates a sample pour sequence diagram for a continuous girder bridge. For precast concrete girders, the cast-in-place diaphragm over the abutment is cast integrally at the same time as the deck above it. The negative-moment regions for steel girders extend between the points of beam dead load contraflexure. For precast concrete girders, the designer should use a minimum of 3 ft on each side of the center of support or 5% of the span length, whichever is greater.

For simple spans, it is desirable to pour the entire deck at once. If this is not practical, the deck may be poured in a series of longitudinal strips with closure pours as needed. For steel bridges, the designer should carefully investigate potential differential deflections.

Precast concrete girders made continuous for live load and superimposed dead load shall be treated as a special case. The deck segment and diaphragm over supports shall be poured after the mid-span regions of the deck have been poured as simple-span loads.



Notes:
1. The direction of pour should be shown for each pour.
2. Pour 3 limits for steel girders are at the points of beam dead load contraflexure.
3. Pour 3 limits for precast girders are 5% of span length or 3' minimum.

TYPICAL POUR DIAGRAM

(Continuous Steel and Precast Girders)
Figure 16.2-E

End wall concrete in integral abutments will usually be cast concurrently with applicable portions of the superstructure (e.g., bottom slab, web/diaphragm, deck). The contract documents shall indicate the requirements for a special placement sequence.

16.2.5.2 Transverse Construction Joints

Place a transverse construction joint in the end span of bridge decks on steel superstructures where uplift is a possibility during the deck pour. Where used, transverse construction joints should be placed parallel to the transverse reinforcing steel. They shall not be placed over field splices.

A bridge with a relatively short end span (60% or less) when compared to the adjacent interior span is most likely to produce this form of uplift. Uplift during the deck pour can also occur at the end supports of curved decks and in superstructures with severe skews. If analysis using the appropriate permanent load factors of LRFD Article 3.4.1 demonstrates that uplift occurs during deck placement, require a construction joint in the end span and require placing a portion of the deck first to act as a counterweight.

16.2.6 <u>Longitudinal Construction Joints</u>

Longitudinal construction joints in bridge decks can create planes of weakness that can lead to maintenance problems. In general, NDOT discourages the use of construction joints, although they cannot be avoided under certain circumstances (e.g., widenings, phased construction). The following will apply to longitudinal construction joints:

1. <u>Usage</u>. Longitudinal construction joints need not be used on decks having a constant cross section where the width is less than or equal to approximately 120 ft. For deck widths greater than 120 ft (i.e., where the finishing machine span width must exceed 120 ft), the designer shall make provisions to permit placing the deck in practical widths. The designer shall detail either a longitudinal joint or a longitudinal closure pour, preferably not less than 3 ft in width. Lap splices in the transverse reinforcing steel shall be located within the longitudinal closure pour. Such a joint should remain open as long as the construction schedule permits to allow transverse shrinkage of the deck concrete. The designer should consider the deflections of the bridge on either side of the closure pour to ensure proper transverse fit up.

2. <u>Location</u>. If a longitudinal construction joint is necessary, do not locate it underneath a wheel line. Preferably, a construction joint should be located outside the girder flange and in a shoulder or median area.

3. <u>Closure Pours</u>. For staged construction projects, a closure pour shall be used to connect the slab between stages. A closure pour serves two useful purposes: It defers final connection of the stages until after the deflection from deck slab weight has occurred, and it provides the width needed to make a smooth transition between differences in final grades that result from construction tolerances. The closure width should relate to the amount of relative dead-load deflection that is expected to occur across the pour after the closure is placed. A minimum closure width of 3 ft is recommended. Greater closure widths may be required when larger relative dead-load deflections are anticipated. The required width can be estimated by considering the closure pour to be a fixed-fixed beam and by limiting the stresses in the concrete to the cracking stress. When a closure pour is used, the following apply:

- Stay-in-place forms shall not be used under the closure pour.

- Diaphragms/cross frames in the staging bay of structural steel girders shall not be rigidly connected until after the adjacent stages of the deck have been poured. Construct concrete diaphragms in the staging bay of prestressed concrete girders after adjacent portions of the bridge are complete. The diaphragms may be poured as part of the closure.

- Reinforcing steel between different stages shall not be tied or coupled until after the adjacent stages of the deck have been poured.

- Support the finishing machine on an overhang jack that is connected to the girder loaded by the deck pour. Do not place the finishing machine on a previously poured deck. The bridge designer must indicate in the contract documents that this method of constructing the closure pour is not allowed. See Figure 16.2-F.

16.2.7 Longitudinal Deck Joints

Reference: LRFD Article 14.5.1.1

Open longitudinal joints, used on slab-on-girder bridges, are not typically needed except on the widest of bridges. The requirement for open longitudinal joints in bridges is based on the bridge width, skew and span configuration. NDOT has not adopted a specified maximum bridge width that can be used without a longitudinal open joint although, as an approximate guide, widths up to 120 ft without a joint are usually acceptable. Open longitudinal joints may be needed where the width of the bridge exceeds 120 ft or on multiple-span bridges with large skews.

The following applies:

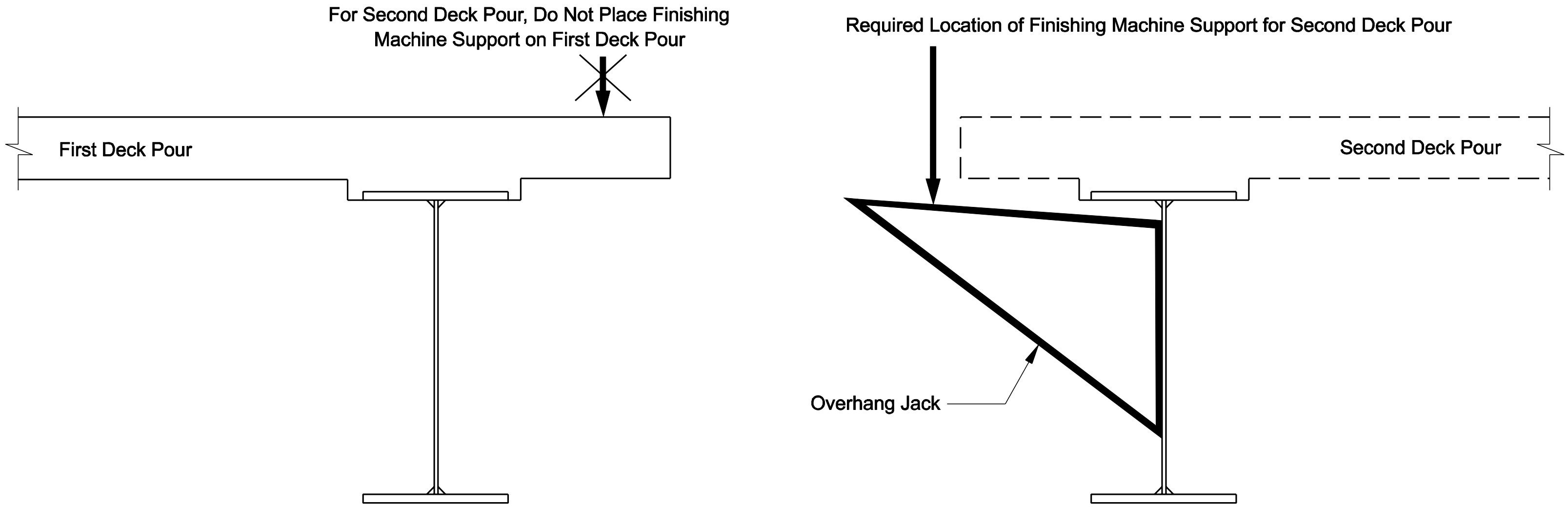
1. Column Design. A longitudinal open joint should be used where transverse temperature controls the column design. Desirably, the column design will be controlled by seismic loads and not other load combinations.

2. Location. Longitudinal open joints shall not be placed over a girder flange. If a longitudinal joint is used, it should be placed in both the superstructure and substructure.

16.2.8 Transverse Edge Beam for Steel Girder Bridges

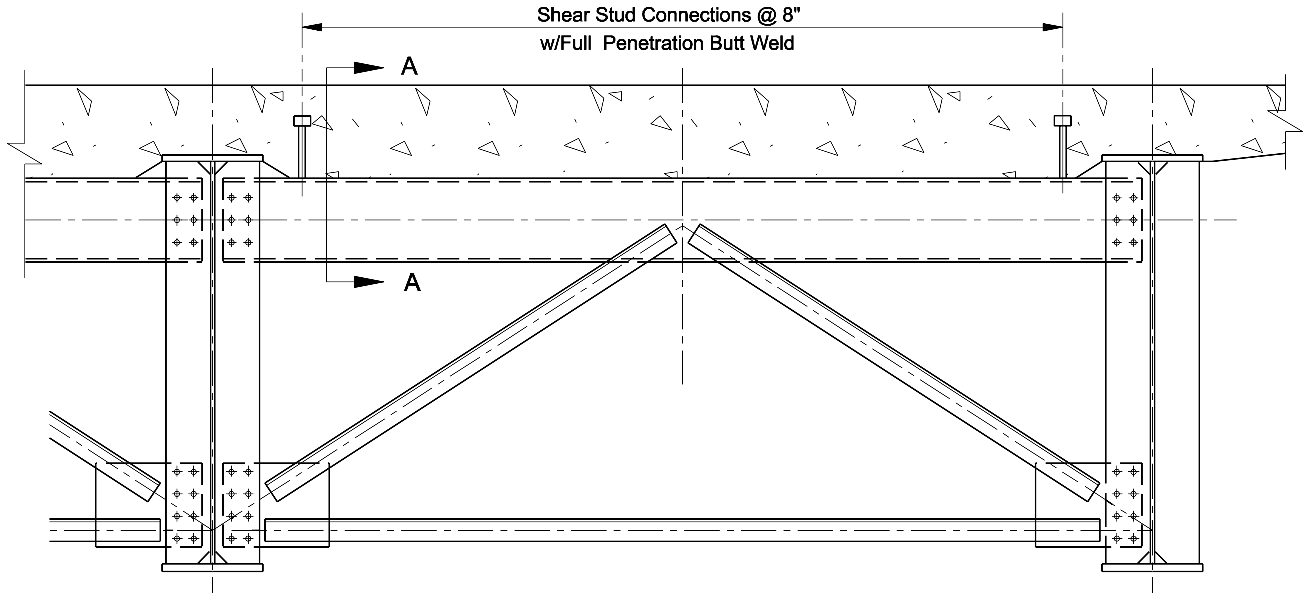
Reference: LRFD Article 9.7.1.4

NDOT practice is to provide a transverse edge beam to support wheel loads near the transverse edge of the deck in conjunction with an end diaphragm for steel girder bridges. See Figure 16.2-G.

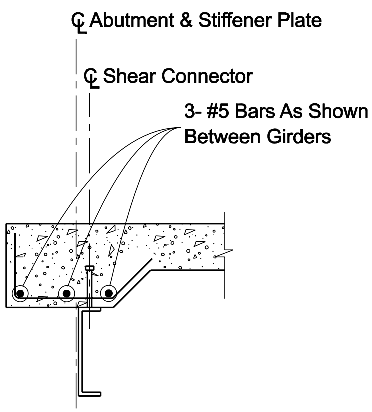


SUPPORT FOR FINISHING MACHINE

Figure 16.2-F



ELEVATION
ABUTMENT CROSSFRAME DETAIL



SECTION A-A

TRANSVERSE EDGE BEAM

Figure 16.2-G

16.2.9 <u>Deck Overhang/Bridge Rail</u>

Reference: LRFD Article 9.7.1.5

16.2.9.1 Overhang Width and Thickness

Bridge deck overhang is defined as the distance between the centerline of the exterior girder to the outside edge of the deck. Typically, NDOT practice is that the overhang width will not be more than 40% of the girder spacing. The thickness of the overhang at the outside edge of deck should be the same as the interior deck thickness. The thickness of the overhang at outside edge of girder should be the deck thickness plus the haunch depth.

16.2.9.2 Construction

Typical NDOT practice is to construct the exterior overhang of the bridge deck slab using an overhang jack for steel and precast concrete girders or falsework for CIP concrete bridges. Overhang jacks are connected to the girder at their top and braced against the web or bottom flange on the bottom. Large overhang widths can cause excessive lateral distortion of the bottom flange and web of the girder. See Section 15.6.4, which requires that the contractor check the twist of the exterior girder and bearing of the overhang bracket on the web. See Figure 16.2-H for typical overhang construction forming on concrete and steel girders and Figure 16.2-I for cast-in-place concrete.

16.2.9.3 Structural/Performance Design

Reference: LRFD Articles 13.6.1, 13.6.2 and 13.7.2

All combination bridge rail/deck overhang designs shall meet the structural design requirements to sustain rail collision forces in LRFD Article A13.2. All bridge rails shall meet the performance requirements of LRFD Article 13.7.2; see Section 16.5. Use a Class 2 exposure factor in LRFD Equation 5.7.3.4-1 for all bridge rails and deck overhang designs.

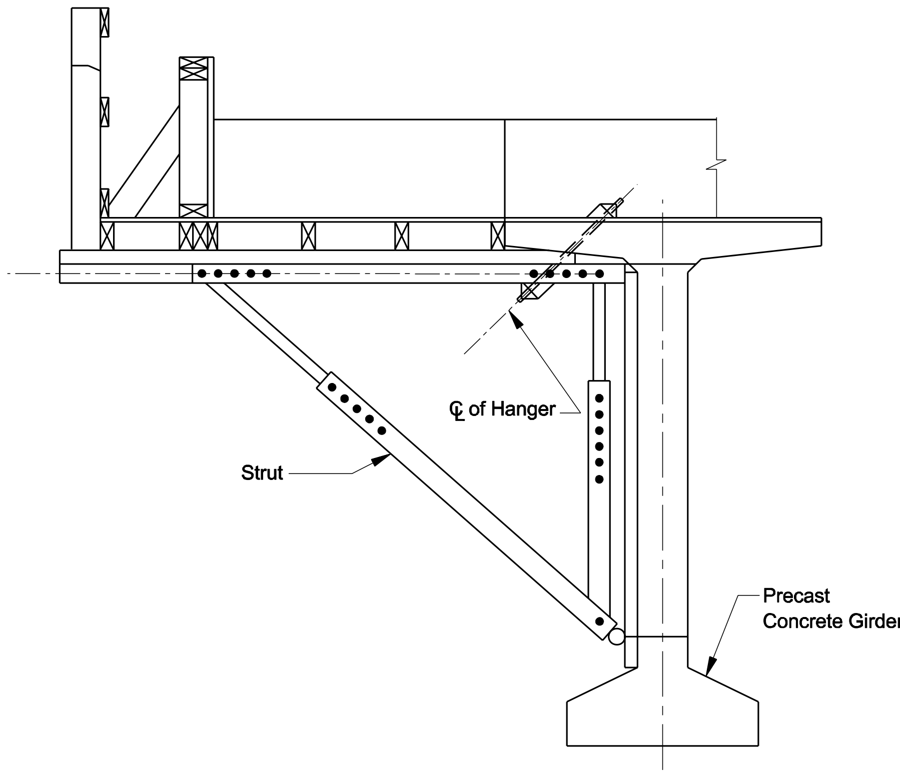
When designing the deck overhang for Extreme Event II, include a vertical wheel load located 12 in from the face of bridge rail in conjunction with transverse and longitudinal bridge rail loads; do not apply the wheel load in combination with vertical rail loads. Design the deck overhang using the rail resistance instead of the rail load. This ensures failure in the rail before the deck overhang.

Sidewalks, when used, are placed on the outside edge of bridge decks adjacent to rails. Assume the point of fixity for the design of the rail at the deck level and not the top of sidewalk.

16.2.9.4 Bridge Rail Joints

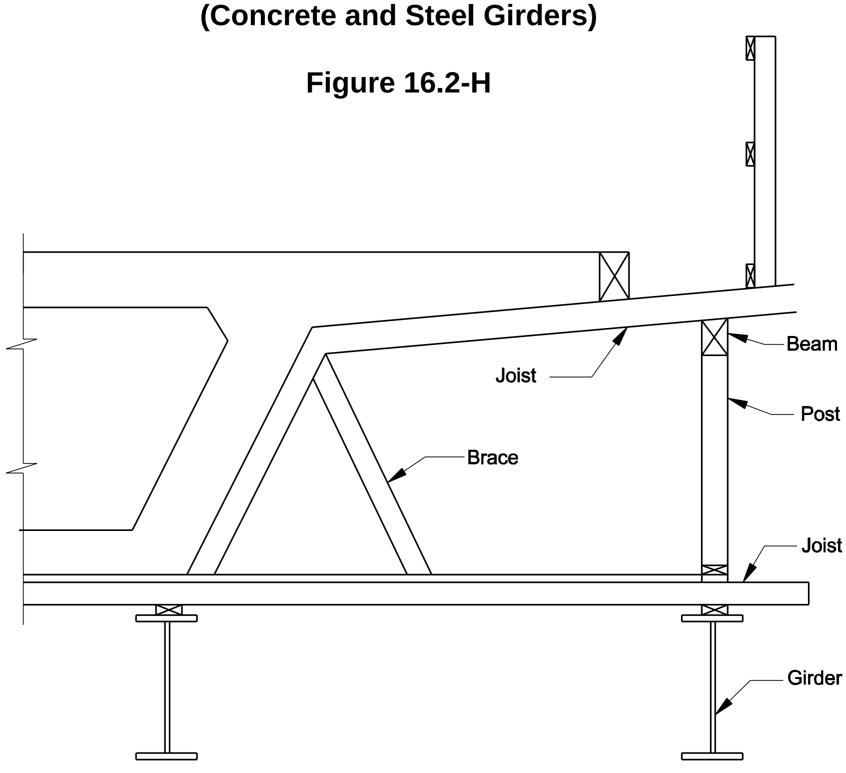
The following applies to bridge rail joints:

1. <u>Concrete Bridge Rail Joints</u>. Joints on concrete bridge rails shall be provided at all locations of expansion in the bridge; i.e., the joints on the bridge deck and barrier will



TYPICAL OVERHANG FORMING SYSTEM
(Concrete and Steel Girders)

Figure 16.2-H



TYPICAL OVERHANG FORMING SYSTEM
(Cast-in-Place Box Girder)

Figure 16.2-I

match. In addition, 2-in open joints in the barrier, extending from the top of the barrier downward 2 ft, shall be provided at the mid-span of each span and over supports. Additional open joints should be considered on longer spans. Open joints shall be designed as discontinuities.

2. Barrier Rail Connection. In Clark County, the expansion joint should extend up into the barrier rail at least 6 in; in the remainder of the State, the extension should be 12 in up the face of the rail.

16.3 APPROACH SLABS

16.3.1 Usage

Approach slabs are required on all bridges. See Section 11.4.7.

16.3.2 Design Criteria

See the *NDOT Drafting Guidelines* and *Standard Plans* for the typical approach slab details. The roadway ends of approach slabs should be designed parallel to the bridge ends except when the skew is more than 20° on approaching roadways with concrete pavement. The following design criteria apply to approach slabs:

1. Materials. Concrete shall be 4 ksi in Clark County and 4.5 ksi in the remainder of the State. The class of concrete used in the approach slabs shall be consistent with the class used in the deck to which they are attached. Grade 60 reinforcing bars shall be used in the design of all approach slabs. In areas outside Clark County, all reinforcing steel in the approach slab shall be epoxy coated.

2. Profile Grade with Asphalt Pavements. For asphalt pavements, NDOT practice is to place the profile grade at the top of the dense graded mix. The actual riding surface on asphalt paving is higher than the profile grade due to the placement of an open grade asphaltic concrete layer. Therefore, bridge decks and the approach slabs must be constructed higher than the profile grade on the approaching roadway. Confirm the thickness of the open grade AC layer with the roadway designer (typically ¾ in, but sometimes other thicknesses are used), and provide a note in the contract documents that indicates this elevation adjustment has been reflected in the plans.

3. Profile Grade with Concrete Pavements. For concrete pavements, NDOT practice is to place the profile grade at the riding surface. Therefore, if the approaching roadway pavement is concrete, bridge decks and the approach slabs will be constructed at the elevation of the profile grade.

4. Analysis. If a special design is used, the approach slab shall be modeled as a simple span.

5. Bridge Approach Joints. Provide a terminal joint or pavement relief joint at the end of the roadway at the bridge approach slab, if the approaching roadway is concrete.

6. Skews. When concrete pavement and skews of 20° or greater exist, a redesign of the typical NDOT approach slab may be necessary. See the *NDOT Standard Plans* for guidance on the layout.

7. Deep End Spans. The minimum approach slab length should be the larger of two times the structure depth plus 3 ft, or 24 ft. Approach slabs longer than 24 ft shall be designed as a longitudinally reinforced slab. The design shall assume that the approach slab is a simple span supported by the bridge on one end and by 3 ft of competent soil at the approach roadway end.

8. Wingwalls. The design forces for wingwalls are due to earth pressure only. It is NDOT practice to extend the approach slabs over the wingwall, which eliminates the live load surcharge in the design of the wingwall. Seismic forces from the soil behind the wingwall must also be considered in the design of wingwalls.

16.3.3 Construction

Approach slabs shall be finished and cured by the same methods used to construct bridge decks.

16.4 DECK DRAINAGE

Reference: LRFD Article 2.6.6

16.4.1 Importance of Bridge Deck Drainage

The bridge deck drainage system includes the bridge deck, sidewalks, railings, gutters, inlets and, for a closed drainage system, the underdeck closed pipe system. The primary objective of the drainage system is to remove runoff from the bridge deck before it collects in the gutter to a point that exceeds the allowable design spread. Proper bridge deck drainage provides many other benefits, including:

- Efficiently removing water from the bridge deck enhances public safety by decreasing the risk of hydroplaning.
- Long-term maintenance of the bridge is enhanced.
- The structural integrity of the bridge is preserved.
- Aesthetics are enhanced (e.g., the avoidance of staining substructure and superstructure members).
- Erosion on bridge end slopes is reduced.

16.4.2 Responsibilities

The Hydraulics Section calculates the flow of water and recommends the location, type and spacing of the deck drain. The bridge designer uses this information to design the deck drains, pipes, cleanouts, support system and outlets. The bridge designer should work with the Hydraulics Section to locate inlets near piers or abutments. The Environmental Services Division determines if the outflow can be placed directly into another drainage facility or if it must be first treated before release to a natural waterway.

16.4.3 Open vs Closed Drainage

The first option, as determined by the Hydraulics Section, is to avoid placing deck drains on the bridge deck. Where this option is not practical, NDOT policy is to use a closed drainage system. An open, free-falling drainage system may be used if all of the following are satisfied:

- The Environmental Services Division determines that an open, free-falling drainage system is acceptable for the proposed location.
- The free-falling discharge is not over or likely to be over travel lanes, shoulders, bicycle facilities or sidewalks beneath a bridge.
- A free-falling discharge is not allowed on railroad right-of-way.
- The free-falling discharge does not have the potential to erode earth slopes or natural ground.

- The free-falling discharge will not flow onto substructure elements. Downspouts should extend at least 2 in below the bottom of the girder. Downspouts should not be located within 5 ft of the end of any substructure units or where water could easily blow over and run down a substructure element. Downspouts should not be located such that a 45° cone of splash beneath the downspout will touch any structural component. Downspouts shall not encroach upon the required vertical or horizontal clearances.

16.4.4 <u>Design of Deck Drainage</u>

16.4.4.1 Deck Slope

To provide proper bridge deck drainage, the absolute minimum profile grade of the bridge should be 0.5% but, preferably, not be less than 1%. The transverse slope of the bridge deck must be accommodated by providing a suitable roadway cross slope, typically 2%.

16.4.4.2 Pipes and Cleanouts

Pipes and cleanouts shall be of a rigid steel pipe, either galvanized or painted, with a diameter not less than 6 in but preferably 8 in. Pipes shall have a minimum grade of not less than 6%. Provide cleanouts at each turn in the pipe and every 100 ft where practical.

16.4.4.3 Structural Considerations

The primary structural considerations in drainage system design are:

1. <u>Decks</u>. Inlet sizing and placement must be compatible with the structural reinforcement and other components of a bridge deck. For example, inlets for reinforced concrete bridge decks must fit within the reinforcing bar design. Thickening of the deck and additional reinforcement may be required to maintain clearances and deck resistance.

2. <u>Caps, Columns and Other Concrete Elements</u>. Pipes entering or exiting caps, columns or other concrete elements must do so where relocating reinforcing steel will not have an adverse effect on the resistance of the element. For example, pipes entering or exiting columns must do so outside the plastic hinge zone. Either relocate the pipe so that it enters or exits outside the plastic hinge zone or place the pipe external to the column. Do not cut reinforcement to accommodate drainage pipes.

3. <u>Corrosion and Erosion</u>. The drainage system should be designed to deter runoff (and the associated corrosives) from contacting vulnerable structural members and to minimize the potential for eroding embankments. To avoid corrosion and erosion, the design must include the proper placement of outfalls. Water running to the end of a bridge must be directed away from the end of wingwalls to prevent erosion.

4. <u>Expansion, Deflection and Rotation</u>. Special attention is required where drainage pipes cross points of expansion or where the superstructure is more flexible than the substructure. Where horizontal pipes cross a point of expansion, the pipe must have an expansion device capable of expanding, contracting and deflecting with thermal and seismic movements while maintaining a closed system. When the superstructure is flexible compared to the substructure, a vertical pipe needs to have some flexibility to account for rotation.

5. <u>Pipe Supports</u>. Pipe runs must be supported by pipe hangers connected to the deck or cross frames. The pipe hanger should have a roller on the bottom to facilitate erection of the pipe and an adjustment screw to set the proper pipe grade. The maximum spacing of hangers shall be 25 ft but no longer than that required by design. Assume that the pipe is full of water when designing hanger spacings.

16.4.4.4 Maintenance Considerations

The drainage system will not function properly if it becomes clogged with debris. Therefore, it is important that maintenance requirements be considered in the design. The bridge designer should avoid drainage designs that provide inadequate room for maintenance personnel on the bridge deck or access beneath the bridge or that provide unsafe working areas for maintenance personnel. Outlets should daylight above the ground to provide access for backflushing, rodding or air-pressure cleaning equipment.

Downspouts for free-falling drainage systems should be located so that the maintenance crews can access them from underneath the bridge and preferably from the ground.

16.5 BRIDGE DECK APPURTENANCES

16.5.1 <u>Bridge Rails</u>

Reference: LRFD Article 13.7

16.5.1.1 Test Levels

Reference: LRFD Article 13.7.2

LRFD Article 13.7.2 identifies six test levels for bridge rails, which have been adopted from NCHRP 350 *Recommended Procedures for the Safety Performance Evaluation of Highway Features*. Test Levels One and Two have no application in Nevada. The following identifies the general test level applications:

1. <u>TL-3 (Test Level 3)</u>. A TL-3 bridge rail is the minimum acceptable performance level for all bridges in Nevada except on NHS facilities. TL-3 is generally acceptable for a wide range of high-speed arterial highways with very low mixtures of heavy vehicles and with favorable site conditions. Performance crash testing is at 60 mph with a 1.55-kip passenger car and a 4.5-kip pickup truck.

2. <u>TL-4 (Test Level 4)</u>. A TL-4 bridge rail is the minimum performance level for bridges on the National Highway System (NHS). TL-4 is generally acceptable for the majority of applications on high-speed highways, freeways and expressways and Interstate highways with a mixture of trucks and other heavy vehicles. Performance crash testing is at 60 mph with a 1.55-kip passenger car and a 4.5-kip pickup truck plus an 18-kip single-unit truck at 50 mph.

3. <u>TL-5 (Test Level 5)</u>. TL-5 is for a special case where large trucks make up a significant portion of the vehicular mix. A TL-5 rail can only be used when approved by the Chief Structures Engineer.

4. <u>TL-6 (Test Level 6)</u>. TL-6 is for a special case where alignment geometry may require the use of an extra height rail. A TL-6 rail can only be used when approved by the Chief Structures Engineer.

16.5.1.2 Bridge Rail Types/Usage

The *NDOT Bridge Drafting Guidelines* presents details for those bridge rail types used by NDOT. The following identifies typical NDOT usage for bridge rails:

1. <u>32-in Concrete F-Shape Bridge Rail</u>. NDOT typically uses this bridge rail on all bridges for which the 42-in F-shape and 42-in vertical wall are not applicable; see Items #2 and #3 below. The 32-in F-shape bridge rail meets the height criteria for a TL-4. The concrete bridge rail's advantages when compared to a metal beam rail include its superior performance when impacted by large vehicles, its relatively low maintenance costs and its better compatibility with the bridge deck system (i.e., the concrete rail can be constructed integrally with the bridge deck). The concrete bridge rail's disadvantages include its higher dead weight.

2. <u>42-in Concrete F-Shape Bridge Rail</u>. NDOT typically uses this bridge rail:

- if the roadway approach barrier is 42 in,
- across railroads,
- across multiple-use (pedestrian, bicycle) facilities, or
- curved structures with high degree of curvature.

The 42-in concrete F-shape bridge rail meets the TL-5 height criteria.

3. 42-in Vertical Concrete Wall. NDOT typically uses this rail where sidewalks are present on the bridge and where the bridge rail is located between the sidewalk and roadway. Its height conforms to the LRFD requirements for pedestrian rails; therefore, its use where sidewalks are present avoids the need to extend the height of a 32-in concrete bridge rail to meet the height requirements of a pedestrian rail or bicycle rail. The 42-in vertical concrete wall meets the TL-5 height criteria.

4. Metal Beam Bridge Rail. NDOT generally discourages the use of any metal beam bridge rail system. Its use may only be considered where aesthetics or other special conditions are important. The Chief Structures Engineer must approve the use of any metal beam bridge rail. When compared to a concrete bridge rail, a metal beam rail's advantages include lower dead weight and providing a more open view of the surrounding scenery. The comparative disadvantages include a lesser ability to contain heavier vehicles, higher maintenance costs, and a more complex structural connection to the bridge deck system.

5. TL-6 Rail. This special rail may be considered on bridges where extra protection for semi-trucks is warranted because:

- The road is a high-speed facility.
- There are a significant number of trucks using the facility.
- The alignment has a sharp degree of curvature.
- The potential consequences of rail penetration would be catastrophic.

The advantage of this system is the extra protection it provides to higher profile vehicles (e.g., a tanker truck). The disadvantages include the poor aesthetics due to its height (90 in) and the design of the bridge deck and superstructure must include the extra weight of the rail and impact loads. The impact loads are also not clearly defined.

16.5.1.3 Guardrail-To-Bridge-Rail Transitions

Standard Plan details are used for most applications of guardrail-to-bridge-rail transitions. Special designs, if required for unusual circumstances, are developed by the bridge designer with input from the Roadway Design Division.

16.5.1.4 Bridge Rail/Sidewalk

Reference: LRFD Articles 13.4 and 13.7.1.1

As discussed in Section 11.9.4.4, the Roadway Design Division determines the warrants for a sidewalk on a bridge. At lower speeds, a raised sidewalk is separated from the adjacent roadway by a vertical curb, which is typically 6 in high. The 42-in bridge rail is located on the outside edge of the sidewalk. However, at higher speeds, the vertical curb is not considered to be adequate protection; the 42-in bridge rail is located between the roadway and sidewalk. For this arrangement, a raised sidewalk is typically not used. A 42-in pedestrian railing is used at the outside edge of the sidewalk.

16.5.2 <u>Bicycle Rails</u>

Reference: LRFD Article 13.9

As discussed in Section 11.9.4.5, the Statewide Bicycle/Pedestrian Coordinator determines if bicycle accommodation is required across a bridge. Where required, a bicycle rail that meets the geometric and loading requirements of LRFD Article 13.9 must be provided. The required height of the bicycle rail is 42 in.

Bicycle paths are bikeways that are physically separated from motorized vehicular traffic by an open space or barrier and may be either within the highway right-of-way or within an independent right-of-way. Bridges that are part of a bicycle path require a 42-in bicycle rail.

16.5.3 <u>Protective Fencing</u>

Protective fencing is used across bridges when protection to facilities adjacent to or beneath the structure is warranted. Fencing is typically required for:

- all overpasses in urban areas, and
- all overpasses over railroads.

Protective fencing may be considered at other locations on a case-by-case basis.

Protective fencing is designated as "pedestrian railing" in the *NDOT Standard Plans* and contract bid items. Standard NDOT pedestrian railings includes:

1. <u>Type M Pedestrian Rail</u>. Type M pedestrian rails are used adjacent to sidewalks where there is no bridge rail on the outside edge of bridge. The Type M pedestrian rail is mounted on a short concrete pedestal. The lower portion is vertical with the upper portion extending back over the sidewalk.

2. <u>Type M Modified Pedestrian Rail</u>. Type M Modified pedestrian rails are used adjacent to sidewalks where there is a bridge rail on the outside edge of bridge. The Type M Modified pedestrian rail is a short version of the Type M pedestrian rail and is mounted on top of the exterior bridge rail.

3. <u>Type V Pedestrian Rail</u>. Type V pedestrian rails are used on bridges where no sidewalks are present. They are located adjacent to traffic and are mounted on the top of the exterior bridge rail. Type V pedestrian rails are vertical.

16.5.4 <u>Utility Attachments</u>

The Structures Division will coordinate with the Utilities Section within the Right-of-Way Division and with the Roadway Design Division for any utility attachments proposed on the bridge.

Utility companies frequently request approval from NDOT to attach utility lines or pipes to bridges. The Structures Division's concern is that the structural performance and function of the bridge not be compromised; that the safety of the individuals using the bridge not be compromised; and that NDOT maintenance of the bridge is not unduly complicated. On new or replaced bridges, the bridge designer should consider detailing two 2-in diameter conduits in each concrete bridge rail (or other similar accommodations) for the future use of NDOT or Utility Companies.

The following guidelines apply to attaching utilities to NDOT-owned structures:

1. Utility lines cannot be attached to the outside edge of new or replaced bridges where the structure crosses another highway or where aesthetics are a concern. The attachment shall be within box cells or between girders, preferably in the outside bays. On existing bridges, it is acceptable but not preferred to mount utility lines on the outside edge.

2. Utility lines cannot hang below the bottom of the girders or below the bottom of the deck on CIP concrete bridges.

3. No field welding is allowed on steel girders. Field drilling may be allowed on concrete girders only at approved locations.

4. All attachments to concrete shall be made with permanent-type, approved epoxy-resin anchors. Attachment hardware shall be galvanized or stainless steel. Some epoxies creep when subjected to permanent tension loads; therefore, provide special attention to the attachment to ensure that appropriate bonding materials are used under these circumstances.

5. Provide expansion couplings at the bridge's points of expansion.

6. New utility lines will not be allowed on bridges with a limiting load posting.

7. Utility attachments must be made in accordance with Nevada State laws. The Utility cannot unilaterally hook up to a bridge because it is convenient without notifying NDOT.

8. Utility attachments are inspected as part of the Nevada Bridge Inspection Program. The utility owner will be notified of any necessary maintenance or repair work. The utility owner is required to secure necessary approvals and permits from NDOT prior to completing repairs. See Section 28.2.6.11.

9. To ensure a safe and structurally adequate installation, NDOT requires an engineered attachment plan from the Utility.

10. If the bridge cannot safely accommodate the traffic loads and the utility, the Utility will not be permitted on the bridge. Also, no attachment will be permitted that impairs NDOT inspection and maintenance activities.

11. A utility attachment that reduces the vertical clearance or freeboard will not be permitted.

12. To ensure a safe installation for the utility, NDOT requires all attachments on the downstream side of the bridge because, during floods, trees and other drift will occasionally strike the girders.

13. NDOT does not allow a utility to pass through an abutment or wing wall without specific approval; they must exit from underneath the roadway as soon as possible.

14. The Utility will not be allowed to bolt through the deck or girders.

15. Because NDOT frequently has maintenance work on bridge rails, bridge rail mounting is not preferred.

16. Trenching operations that are so close to the bridge footings such that there may be undercutting or sloughing will not be allowed.

17. NDOT is not the final approval authority for attachments to historic bridges; these must also be cleared with other agencies.

18. The Utility is responsible for any damage resulting from the presence of the utility on the bridge.

19. Installation of the utility should not interfere with the NDOT contractor constructing the bridge.

20. For a pipeline containing fluids or gases, the installation must be cased the full length of the bridge and extend a minimum of 50 ft beyond the end of abutment or 10 ft beyond the end of the approach slab. Trenches close to footings or piles may also require casing on a case-by-case basis.

21. The utility attachment shall be designed to prevent discharge of the pipe product into the stream or river in case of pipe failure.

22. Use of bridge members to resist forces caused by moving fluids will not be permitted.

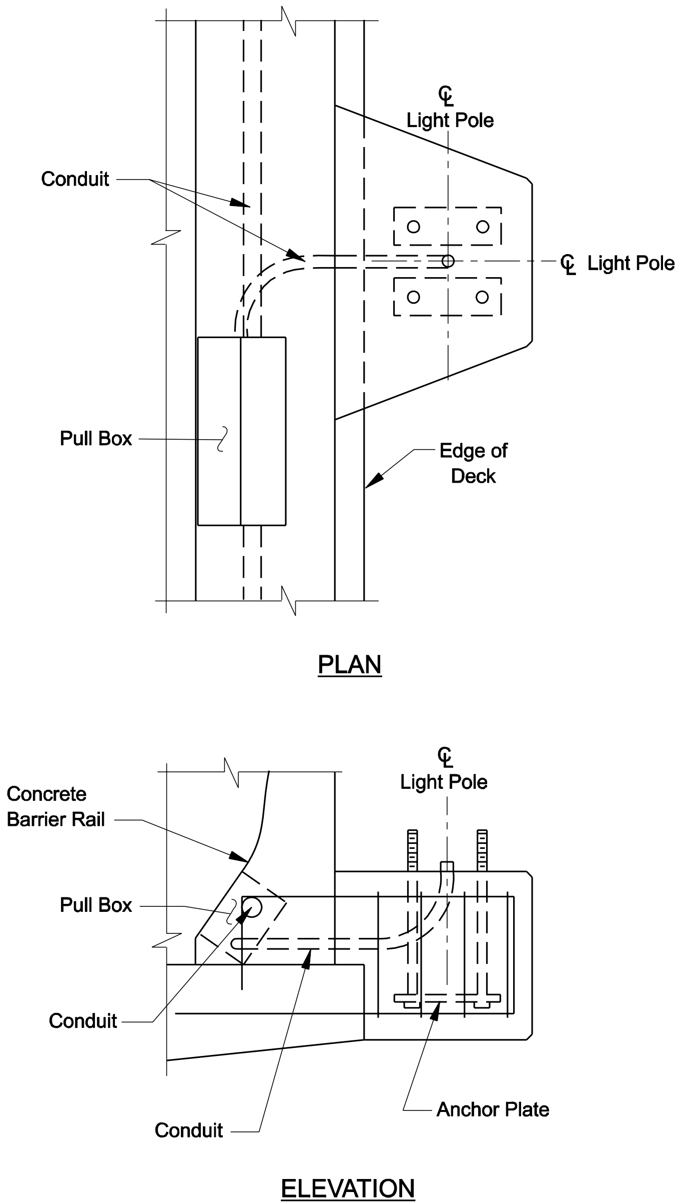
23. Utility attachments will require that an expansion-deflection device be installed where the conduit or casing crosses a bridge expansion joint.

16.5.5 Sign Attachments

If the Traffic Engineering Section proposes to attach a sign to a bridge, the Section must coordinate with the Structures Division. The Structures Division will assess the structural impact on the bridge and, if the sign attachment is approved, the Structures Division will design the attachment details. Signs cannot decrease the vertical clearance.

16.5.6 Luminaire/Traffic Signal Attachments

The Traffic Engineering Section determines the warrants for highway lighting and traffic signals, and the Section performs the design work to determine, for example, the spacing of the luminaries and the provision of electricity. Lighting will often be included on bridges that are located in urban areas; traffic signal warrants are determined on a case-by-case basis. Where attached to a bridge, the Structures Division will design the structural support details for the luminaire and/or traffic signal attachments to the bridge. Locate soffit lighting such that it will not adversely affect vertical clearances over traffic lanes or shoulders. The design is not standardized as situations vary greatly. Figure 16.5-A presents an example of a viable solution.



EXAMPLE OF A LUMINAIRE SUPPORT

Figure 16.5-A

Chapter 17

FOUNDATIONS

NDOT STRUCTURES MANUAL

September 2008

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FOUNDATIONS

A critical consideration for the satisfactory performance of any structure is the proper selection and design of a foundation that will provide adequate support and addresses constructibility considerations. This Chapter discusses NDOT-specific criteria that are supplementary to Section 10 of the *LRFD Specifications* for the design of spread footings, driven piles and drilled shafts. Section 11.7 presents NDOT criteria for the selection of an appropriate foundation type within the context of structure-type selection.

17.1 GENERAL

17.1.1 Chapter Scope

Chapter 17 has been prepared primarily for use by the bridge designer and as a reference for the geotechnical engineer and hydraulics engineer. The *NDOT Geotechnical Policies and Procedures Manual*, which is the responsibility of the Geotechnical Section, discusses the geotechnical considerations for the design of bridge foundations. The *NDOT Drainage Manual*, which is the responsibility of the Hydraulics Section, discusses the evaluation of hydraulic scour for bridge foundations.

17.1.2 Design Methodology

This Chapter is based upon the load and resistance factor design (LRFD) methodology. The following summarizes the concepts in the *LRFD Specifications* for the design of foundations.

Considering basic design principles for foundations, the *LRFD Bridge Design Specifications* implemented a major change when compared to the traditional principles of the *Standard Specifications for Highway Bridges*. The *LRFD Specifications* distinguishes between the strength of the in-situ materials (soils and rock strata) supporting the bridge and the strength of the structural components transmitting force effects to these materials. The distinction is emphasized by addressing in-situ materials in Section 10 “Foundations” and structural components in Section 11 “Abutments, Piers and Walls,” which is necessitated by the substantial difference in the reliability of in-situ materials and man-made structures. The foundation provisions of the *LRFD Specifications* are essentially strength design provisions with a primary objective to ensure equal, or close to equal, safety levels in all similar components against structural failure.

Sections 5 and 6 of the *LRFD Specifications* specify requirements for concrete and steel components. The appropriate provisions from these Sections are applied in the structural design of footings, steel and concrete piles and drilled shafts.

The target safety levels for each type of foundation are selected to achieve a level of safety comparable to that inherent in those foundations designed with the *Standard Specifications*. This approach differs from that for superstructures, where a common safety level has been selected for all superstructure types.

Historically, the primary cause of bridge collapse has been the scouring of in-situ materials. Accordingly, the *LRFD Specifications* introduced a variety of strict provisions for scour protection, which may result in deeper foundations.

17.1.3 Bridge Foundation Design Process

The selection of a foundation type involves an evaluation of the load/structural considerations for the superstructure and substructure, the geotechnical factors pertaining to the native soils, and site conditions. The following summarizes the NDOT procedure for selecting and designing a bridge foundation type:

1. Preliminary Structure Layout. The bridge designer obtains preliminary soils information from the Geotechnical Section to assist with the selection of support locations and span lengths. Preliminary foundation loads are calculated and provided to the Geotechnical Section.

2. Scour Potential. For bridges over waterways, the Hydraulics Section evaluates the proposed bridge site and preliminary structure layout to identify the predicted hydraulic scour based on material properties provided by the Geotechnical Section. This analysis is provided to both the Structures Division and the Geotechnical Section.

   As part of the specific subsurface site investigation, the Geotechnical Section will provide a geologic or historic elevation for scour. The Hydraulics Section will calculate an anticipated hydraulic scour depth. The bridge designer in conjunction with the Geotechnical Section and Hydraulics Section will determine a "design" scour for the design of the foundation.

3. Geotechnical Data. For all sites, the Geotechnical Section conducts a site-specific subsurface investigation and prepares a Geotechnical Report. The Geotechnical Section provides this Report to the Structures Division.

4. Foundation Type Selection. Based on information provided by the bridge designer (e.g., structure layout, vertical and lateral loads, settlement criteria), the Geotechnical Section provides the foundation-type recommendation to the Structures Division in the Geotechnical Report. In the absence of mitigating circumstances, including the evaluation of the estimated construction costs, the Structures Division typically accepts this foundation-type recommendation. Environmental considerations may not allow the use of driven piles.

5. Detailed Structural Design. The bridge designer performs the detailed structural design of the foundation based on Section 10 of the *LRFD Bridge Design Specifications* as modified by Chapter 17 of the *NDOT Structures Manual* in conjunction with the structural requirements of Sections 5 and 6 of the *LRFD Specifications.*

17.1.4 Bridge Design/Geotechnical Design Interaction

17.1.4.1 Overview

Prior to the design of the foundation, the bridge designer must have knowledge of the environmental, climatic and loading conditions expected during the life of the proposed unit. The primary function of the foundation is to spread concentrated loads over a sufficient zone, to provide adequate bearing resistance and limitation of movement and, when necessary, to transfer loads through unsuitable foundation strata to suitable strata. Therefore, knowledge of the subsurface soil conditions, ground water conditions and scour is necessary.

The Geotechnical Section is responsible for developing a subsurface exploration program and preparing preliminary geotechnical design recommendations and a Final Geotechnical Report. The Structures Division uses this information to design bridge foundations and other structures. The successful integration of the geotechnical design recommendations into the bridge design will require close coordination between the Geotechnical Section and the Structures Division.

17.1.4.2 Preliminary Geotechnical Design Recommendations

The preliminary geotechnical design recommendations provide general geotechnical recommendations based on existing soil information and any preliminary subsurface investigation that may have been conducted for the project. These general geotechnical recommendations are used to select the bridge foundation and initiate the preliminary structure design. The geotechnical recommendations are used in conjunction with the input of the Hydraulics Section (as applicable) to establish support locations. Prior to beginning work on preliminary bridge design, the bridge designer will review the preliminary geotechnical design recommendations to gain knowledge of the anticipated soil conditions at the bridge site and the recommended general foundation types. The preliminary geotechnical design recommendations provide a preliminary footing elevation and an expected allowable bearing pressure when spread footings are recommended. For deep foundations, the recommendations will include the use of a driven pile or drilled shafts. Driven piles will include pile capacity and type. This preliminary geotechnical information is used to estimate sizes of foundation members and prepare the preliminary bridge design.

17.1.4.3 Final Geotechnical Report

17.1.4.3.1 *Subsurface Exploration*

After a field review has been conducted, a detailed subsurface exploration is performed based on the bridge abutment and pier locations and anticipated foundation type as shown on the Preliminary Front Sheet. The Geotechnical Section determines the proposed boring locations. Typically, the structural modeling and analysis of the bridge proceed based on the preliminary geotechnical design recommendations while the geotechnical subsurface exploration is conducted. During this time, the Structures Division assumes a preliminary point-of-fixity or preliminary footing elevation to model the substructure. The Structures Division determines, verifies and provides foundation loads or calculated bearing pressures to the Geotechnical Section. The Structures Division provides the design loads (vertical and horizontal) at the bottom of substructure units. The Structures Division also provides the elevation at which the loads or bearing pressures are applied. When the geotechnical subsurface exploration has been completed, the Geotechnical Section will perform laboratory testing and geotechnical design. They will issue a Final Geotechnical Report based on the field exploration, laboratory testing, geotechnical design, the preliminary bridge design and the loads provided by the Structures Division.

17.1.4.3.2 *Foundation Design*

The Final Geotechnical Report is used to design foundations for bridges and bridge-related structures. For deep foundations, the Final Geotechnical Report provides tip elevations and p-y soil models of the subsurface that are used to perform foundation lateral soil-structure interaction analyses. The Structures Division then performs the lateral soil-structure interaction analysis with computer programs such as StrainWedge, LPile Plus or COM624P. The Structures Division uses this information to compute lateral displacements and to analyze the structural adequacy of the columns and foundations. The lateral soil-structure interaction analysis is also used to select the appropriate method (point-of-fixity, stiffness matrix, linear stiffness springs or p-y nonlinear springs) to model the bridge foundation in the structural design software. For spread footings, the Final Geotechnical Report provides the estimated footing elevation, allowable bearing pressure, and estimates on settlements and lateral displacements. The Structures Division uses this information to finalize the design of the footing and verify that members are not overstressed. The Final Geotechnical Report may also include notes and tables to be incorporated in the contract documents.

17.1.4.3.3 *Seismic Design*

For bridges on deep foundations requiring rigorous seismic analysis, the Structures Division performs lateral soil-structure interaction analyses using Extreme Event I loadings. If soil liquefaction is anticipated, the Geotechnical Section will provide the Structures Division with foundation downdrag loads due to liquefaction for use in developing the Extreme Event I load combination. The Geotechnical Section will also provide any lateral soil forces that act on the foundation as a result of seismically induced stability movements of earth retaining structures (e.g., embankments, retaining walls) or lateral soil movements attributable to lateral spread. These additional lateral loads should be included in the Extreme Event I load combinations when evaluating lateral soil-structure interaction. The Geotechnical Section will generate the p-y soil model of the subsurface that accounts for cyclic loadings and any liquefied soil conditions. The Structures Division then performs the lateral soil-structure interaction analysis with computer programs such as StrainWedge, LPILE Plus or COM624P. The Structures Division uses this information to calibrate the seismic model of the structure.

17.1.4.3.4 *Foundation Redesign*

If structural members are overstressed due to anticipated deformations or if the deformations exceed acceptable limits from any loading combination, then a redesign of the foundation is required. Redesign may include the adjustment of support member spacing or modification of member sizes. When a redesign of the foundation is required, the Structures Division must resubmit the redesign information (e.g., new foundation layout, sizes, foundation load combinations) to the Geotechnical Section. The Geotechnical Section will analyze the new foundation and resubmit any necessary information to the Structures Division.

17.1.5 <u>Bridge Design/Hydraulic Design Interaction</u>

Bridges and other structures exposed to stream flow may be subject to local and/or contraction scour. The bridge designer must work closely with the Hydraulics Section to determine the extent of scour. This may require an interactive design process. When a redesign of the foundation is required, the Structures Division must resubmit the redesign information (e.g., new foundation layout, sizes, foundation load combinations) to both the Hydraulics Section and the Geotechnical Section. The Geotechnical Section will analyze the new foundation and resubmit the necessary information to the Structures Division. The Hydraulics Section will analyze the new foundation and confirm that the new design adequately accommodates local and contraction scour.

17.2 SPREAD FOOTINGS AND PILE CAPS

Reference: LRFD Article 10.7

This discussion applies to both spread footings supported on soil and to pile caps. Pile caps distribute loads among two or more driven piles or drilled shafts that support a single column, group of columns or pier wall.

17.2.1 Usage

As noted in Section 11.7.2, spread footings are NDOT's preferred foundation type if soils and estimated settlements allow their use. Spread footings are thick, reinforced concrete members sized to meet the structural and geotechnical loading requirements for the proposed structural system. A factor affecting the size of the footing is the structural loading versus the ability of the soil to resist the applied loads. Spread footings are prohibited:

- at stream crossings where they may be susceptible to scour, and
- on MSE fills.

17.2.2 Dynamic Load Allowance (IM)

If a significant portion of the footing is above ground, dynamic load allowance (IM), traditionally termed impact, shall be applied to the proportioning of footings.

17.2.3 Thickness

Reference: LRFD Articles 5.13.3.6 and 5.13.3.7

The footing thickness may be governed by the development length of the column or wall reinforcement, or by shear requirements. Generally, shear reinforcement in footings should be avoided. If shear governs the thickness, it is usually more economical to use a thicker footing without shear reinforcement instead of a thinner footing with shear reinforcement.

Use a minimum footing thickness of 2 ft for bridge abutments and piers.

17.2.4 Depth

Reference: LRFD Articles 5.8.3, 5.13.3.6 and 5.13.3.8

The following will apply:

1. In Waterways. On soil, the top of spread footing must be located below the design scour depth. On rock, the bottom of the footing must be 1 ft below the surface of the scour-resistant rock.

2. Minimum Embedment and Bench Depth. Spread footings shall be embedded a sufficient depth to provide the greatest of the following:

- adequate bearing, scour and frost heave protection;
- 3 ft to the bottom of footing; or
- 2 ft of cover over the footing.

Pile caps may be located above the lowest anticipated scour level provided that the piles are designed for this condition. Footings shall be constructed so as to neither pose an obstacle to water traffic nor be exposed to view during low flow. Footings shall be constructed so as to pose minimum obstruction to water and debris flow if exposed during high flows.

Abutment footings shall be constructed so as to be stable if scour or meandering causes a loss of approach fill.

17.2.5 Bearing Resistance and Eccentricity

Reference: LRFD Article 10.6.3

The required nominal bearing and the geotechnical resistance factor shall be shown in the contract documents. See Chapter 5.

17.2.5.1 Soils Under Footings

Reference: LRFD Article 10.6.3.1.5

In contrast to the approach in the *Standard Specifications for Highway Bridges*, a reduced effective footing area based upon the calculated eccentricity is used to include the effects of bearing resistance and eccentricity. Uniform design bearing pressure is assumed over the effective area. Figure 17.2-B provides an example.

The location of the resultant of the center of pressure based upon factored loads should be within the middle ½ of the base.

17.2.5.2 Rock

Reference: LRFD Article 10.6.3.2.5

Following the traditional approach, a triangular or trapezoidal pressure distribution is assumed for footings on rock. This model acknowledges the linear-elastic response of rock.

The location of the resultant center of pressure based upon factored loads should be within the middle ¾ of the base.

17.2.6 Sliding Resistance

Reference: LRFD Article 10.6.3.3

Use the coefficients of friction in the *LRFD Specifications* for sliding resistance.

Keys in footings to develop passive pressure against sliding are not commonly used for bridges. When it becomes necessary to use a key, the bridge designer should consult with the Geotechnical Section.

17.2.7 <u>Differential Settlement</u>

Reference: LRFD Articles 3.12.6, 10.6.2.2 and 10.7.2.3

17.2.7.1 NDOT Practice

Differential settlement (SE) is considered a superstructure load in the *LRFD Specifications*. Differential settlement is defined as the difference between the settlements of two adjacent foundations. Generally, due to the methods used by NDOT to proportion foundations, settlements are within a tolerable range and, therefore, force effects due to differential settlement need not be investigated. The following presents general NDOT practices on the acceptable limits for settlement:

1. <u>Estimated Differential Settlement</u>. If the Geotechnical Section estimates that the differential settlement is ½ in or less, the bridge designer may usually ignore the effects of differential settlement in the structural design of the bridge.

2. <u>Angular Distortion</u>. Angular distortion is the differential settlement divided by the distance between the adjacent foundations. LRFD Article C10.5.2.2 states that angular distortions between adjacent foundations greater than 0.008 radians in simple spans and 0.004 radians in continuous spans should not be ordinarily permitted, and the Article suggests that other considerations may govern. NDOT does not use the LRFD limits for design, which are related to structural distress, because these angular distortions yield unacceptable impacts on rideability and aesthetics. Typically, meeting the requirements of Comment No. 1 on differential settlement should preclude exceeding the angular distortions allowed by the *LRFD Specifications*.

3. <u>Piers</u>. Continuous footings or deep foundations should be considered where differential settlement is a concern between columns within a pier.

17.2.7.2 Effects of Foundation Settlement

If varying conditions exist, settlement will be addressed in the Final Geotechnical Report, and the following effects should be considered:

1. <u>Structural</u>. The differential settlement of substructures causes the development of force effects in continuous superstructures. These force effects are directly proportional to structural depth and inversely proportional to span length, indicating a preference for shallow, long-span structures. They are normally smaller than expected and tend to be reduced in the inelastic phase. Nevertheless, they may be considered in design if deemed significant, especially those negative movements that may either cause or enlarge existing cracking in concrete deck slabs.

2. <u>Joint Movements</u>. A change in bridge geometry due to settlement causes movement in deck joints that should be considered in their detailing, especially for deep superstructures.

3. <u>Profile Distortion</u>. Excessive differential settlement may cause a distortion of the roadway profile that may be undesirable for vehicles traveling at high speed.

4. <u>Appearance</u>. Viewing excessive differential settlement may create a feeling of lack of safety.

5. Mitigation. Ground modification techniques may be used to improve the soil to address differential settlement concerns. These techniques include but are not limited to:

- chemical grouting,
- over-excavation and replacement,
- surcharging,
- the construction of stone columns, and
- compaction grouting.

17.2.8 Reinforcement

Reference: LRFD Articles 5.10.8 and 5.13.3

Section 14.3 discusses NDOT practices for the reinforcement of structural concrete. The design of spread footings shall meet all applicable requirements in Section 14.3. Unless other design considerations govern, the reinforcement in footings should be as follows:

1. Steel in Top of Footing. For pile caps, the anchorage of piles or drilled shafts into footings requires tension reinforcement in the top of the footing to resist the potential negative bending under seismic action. The minimum reinforcement in the top of pile caps and spread footings shall be as required by design but, in no case, less than #6 bars at 12-in spacing.

2. Embedment Length. Vertical steel extending out of the footing shall extend down to the bottom pile cap or spread footing steel and shall be hooked on the bottom end regardless of the footing thickness.

3. Spacing. The minimum spacing of reinforcing steel in either direction is 6 in on center; the maximum spacing is 12 in on center.

4. Vertical Footing Reinforcement. In addition to the provisions of LRFD Article 5.8.3, the following shall apply: The minimum vertical reinforcement for spread footings and pile caps shall be #5 bars at 36-in spacing in each direction. Additionally, the minimum vertical reinforcement for column spread footings and pile caps shall be #5 bars at 12-in spacing in each direction in a band between "d" of the footing from the column face and beginning 6 in maximum from the column reinforcement. Vertical bars shall be hooked around the top and bottom flexure reinforcement in the footing or cap using alternating 90° and 135° hooks. See the *NDOT Bridge Drafting Guidelines* for typical detailing. These vertical bars enhance seismic performance and are not necessarily for shear resistance.

5. Tremie Seal. Where a tremie seal is used and there are no piles, the bottom footing reinforcement shall be 6 in above the bottom of footing. Where a tremie seal is used and there are piles extending through the tremie, the reinforcement shall be placed above the top of piling.

6. Other Reinforcement Considerations. LRFD Article 5.13.3 specifically addresses concrete footings. For items not included, the other relevant provisions of Section 5 should govern. For narrow footings, to which the load is transmitted by walls or wall-like bents, the critical moment section shall be taken at the face of the wall or bent stem; the critical shear section is a distance equal to the larger of "d_v" (d_v is the effective shear depth of the footing) or "$0.5d_v \cot \theta$" (θ is the angle of inclination of diagonal compressive stresses as defined in LRFD Article 5.8.3.4) from the face of the wall or bent stem where

the load introduces compression in the top of the footing section. For other cases, either LRFD Article 5.13.3 is followed, or a two-dimensional analysis may be used for greater economy of the footing.

17.2.9 Miscellaneous

17.2.9.1 Joints

Footings do not generally require construction joints. Where used, footing construction joints should be offset 2 ft from expansion joints or construction joints in walls and should be constructed with 3-in deep keyways.

17.2.9.2 Stepped Footings

Stepped footings may be used occasionally. Where used, the difference in elevation of adjacent stepped footings should not be less than 2 ft. The lower footing should extend at least 2 ft under the adjacent higher footing.

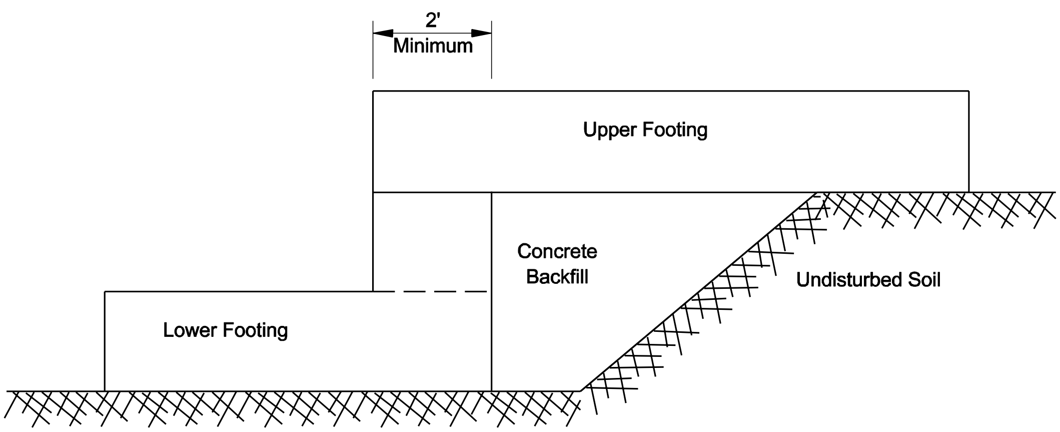
If high bearing pressures under a spread footing are present, use concrete backfill instead of granular backfill for support under the upper step. See Figure 17.2-A.

17.2.10 Example Analysis of a Spread Footing on Competent Soil

Figure 17.2-B presents a schematic example of a spread footing on soil.

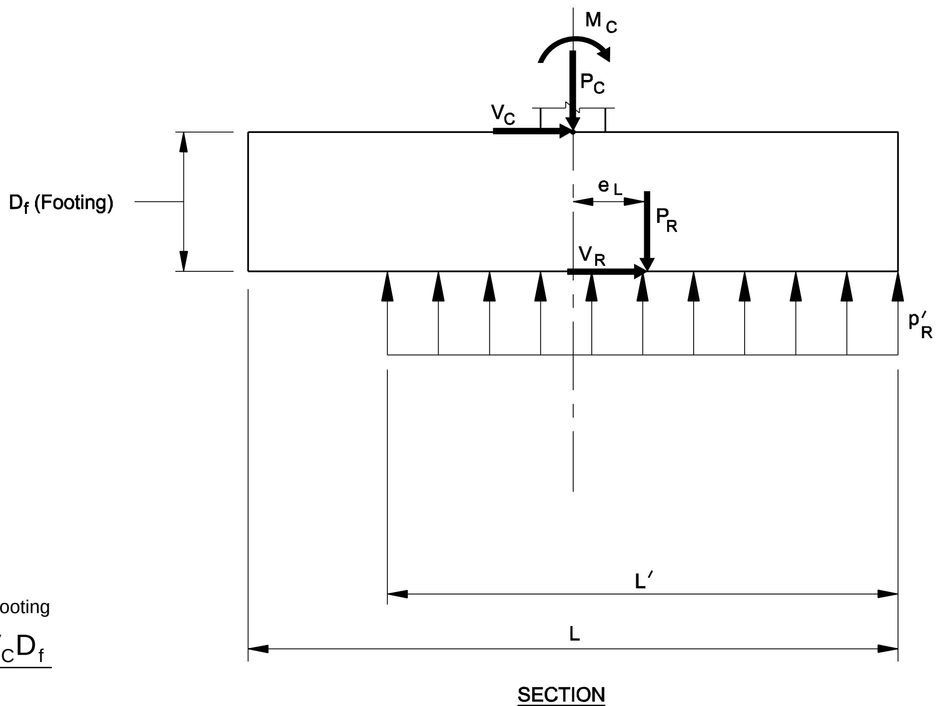
17.2.11 Example Analysis of Pile Caps

Figure 17.2-C presents a schematic example of the analysis of a pile cap to support a pier .



CONCRETE BACKFILL UNDER STEPPED FOOTING WITH HIGH BEARING PRESSURES

Figure 17.2-A



SECTION

$$P_R = P_C + P_{Footing}$$

$$e_L = \frac{M_C + V_C D_f}{P_R}$$

$$V_R = V_C$$

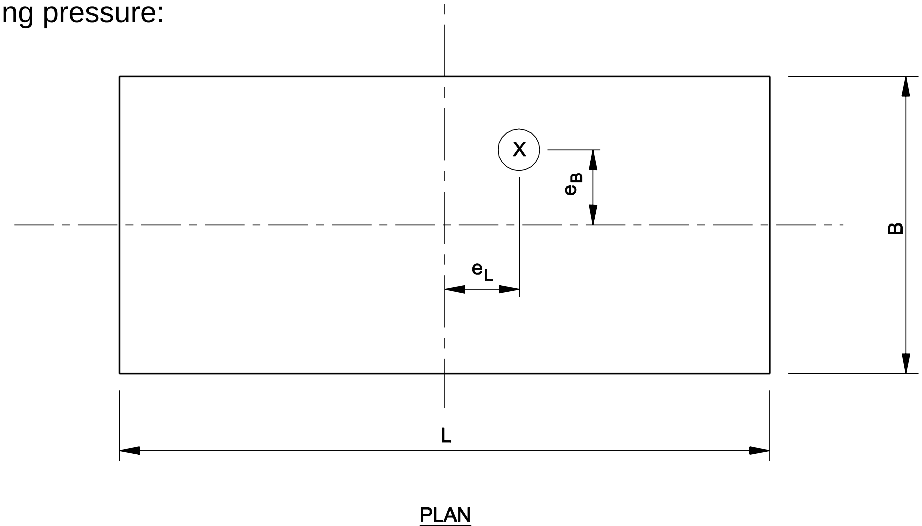
$$L' = L - 2e_L$$

In two dimensions, bearing pressure:

$$p'_R = \frac{P_R}{(L')(B')}$$

Where:

$L' = L - 2e_L$
$B' = B - 2e_B$

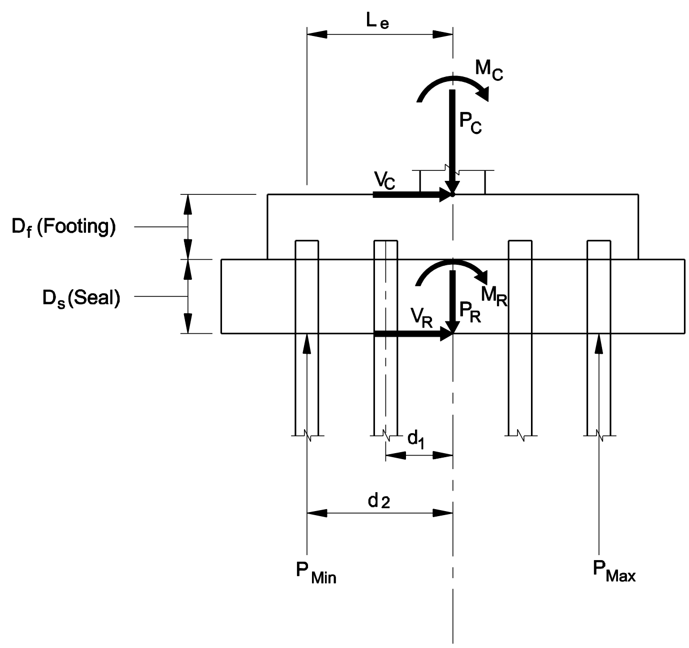


PLAN

Note: See LRFD Article 10.6.3.1.5.

EXAMPLE ANALYSIS OF SPREAD FOOTING ON COMPETENT SOIL

Figure 17.2-B



P_R = $P_c + P_{footing} + P_{seal}$ – Buoyancy

Assumptions: Pile footing is rigid (footing is considered rigid if $L_e/D_f \leq 2.2$). Pile connections are pinned, or shear force in pile is small.

V_R = $V_c - V_{\text{passive soil pressure on footing and seal}}$ *Note: Passive soil pressure is typically ignored.*

M_R = $M_c + V_c(D_f + D_s)$

Pile Loads:

$$P_{max} = \frac{P_R}{\text{\# of piles}} + \frac{M_R\ d_2}{\Sigma d_i^2}$$

$$P_{min} = \frac{P_R}{\text{\# of piles}} - \frac{M_R\ d_2}{\Sigma d_i^2}$$

EXAMPLE ANALYSIS OF A PILE CAP

Figure 17.2-C

17.3 DRIVEN PILES

Piles serve to transfer loads to deeper suitable strata. Piles may function through skin friction and/or through end bearing.

17.3.1 Pile Types/Selection

See Section 11.7.2 for NDOT practices for selecting driven piles as the foundation type. To limit the number of pile types and sizes used throughout a project, use only one pile type and size, if practical.

17.3.1.1 Steel Pipe Piles

Reference: LRFD Articles 6.9.5 and 6.12.2.3

Steel pipe piles are the most common type of driven pile used by NDOT. A typical use of steel pipe piles is in waterways where the predicted scour is deep and driving conditions are favorable. The following applies:

1. Diameter. NDOT uses pipe pile diameters of 12 in to 24 in. The wall thickness typically varies between ⅜ in to ⅝ in, depending on the pile size and driving conditions.

2. Interior Filler. Steel pipe piles are typically filled with concrete and reinforced with 1% of the concrete area or as required by design to develop the pile loads.

17.3.1.2 Steel H-Piles

NDOT occasionally uses steel H-piles where deep scour is not anticipated and a bearing condition is anticipated. The steel H-piles typically used by NDOT are:

- HP10
- HP12

On large projects, where a significant savings may be realized by using non-typical sizes or where the design dictates, other standard AISC sizes may be used.

17.3.1.3 Prestressed Concrete Piles

NDOT rarely uses prestressed concrete piles. Where prestressed concrete piles are used, typical sizes are 12 in to 18 in square or octagonal sections. Spiral reinforcement is permitted in prestressed concrete piles.

17.3.1.4 Pile Selection

The Geotechnical Section ultimately determines the selected type of pile. Figure 17.3-A provides guidance in selecting pile types based on their typical usage by NDOT.

Pile Type	Soil Conditions and Structural Requirements
Steel pipe pile (closed or open end)	Loose to medium dense soils or clays where skin friction is the primary resistance and lateral stiffness in both directions is desirable, especially in rivers where deep scour is anticipated and high lateral stiffness is needed. Primarily used as a friction pile.
Steel H-pile	Rock or dense soil where end bearing is desirable and lateral flexibility in one direction is not critical. Primarily used for end bearing.
Prestressed concrete pile	Loose to medium dense soils or clays where skin friction is the primary resistance.

DRIVEN PILE SELECTION GUIDE

Figure 17.3-A

17.3.2 Design Details

Reference: LRFD Article 10.7.1

17.3.2.1 Pile Length

Reference: LRFD Articles 10.7.1.10, 10.7.1.11 and 10.7.1.12

Pile length will be determined on a project-by-project basis. All piles for a specific pier or abutment should be the same length where practical. Pile lengths should be shown in whole foot increments.

The design and minimum pile tip elevations shall be shown on the drawing of the structural element in the contract documents. Design pile tip elevations shall reflect the elevation where the required ultimate pile capacity is anticipated to be obtained. Minimum pile tip elevations shall reflect the penetration required, considering scour and liquefaction, to support both axial and lateral loads.

Piles placed at abutment embankments that are more than 5 ft in depth require pre-drilling. The size of the pre-drilled hole shall be 2 in larger than the diameter or largest dimension of the pile.

The Final Geotechnical Report will provide project-specific recommendations for the pile embedment, socketing and special construction requirements.

17.3.2.2 Reinforced Pile Tips

Where hard layers are anticipated, use reinforced pile tips to minimize damage for all steel piles. Where rock is anticipated, the pile tips shall be equipped with teeth designed to penetrate into the rock.

The Geotechnical Section recommends the type of pile tip to be used. The bridge designer must show this in the contract documents.

17.3.2.3 Battered Piles

Vertical piles are preferred. Battered piles, typically 1H:3V, may be considered where there is inadequate horizontal resistance. If battered piles are used, a refined analysis is recommended; a two-dimensional analysis is a minimum.

17.3.2.4 Spacing

Spacing of piles is specified in LRFD Article 10.7.1.2. Center-to-center spacing should not be less than the greater of 30 in or 2½ times the pile diameter/width of pile. The distance from the side of any pile to the nearest edge of footing shall not be less than 9 in.

Pile spacing should not normally exceed 10 ft.

17.3.2.5 Orientation

The orientation of steel H-piles (strong versus weak axis) is a design consideration, and it is preferable that all piles be oriented the same. For diaphragm-with-pile integral abutments, typically use a single row of steel H-piles or steel pipe piles, driven vertically, with the strong axis parallel to the diaphragm centerline.

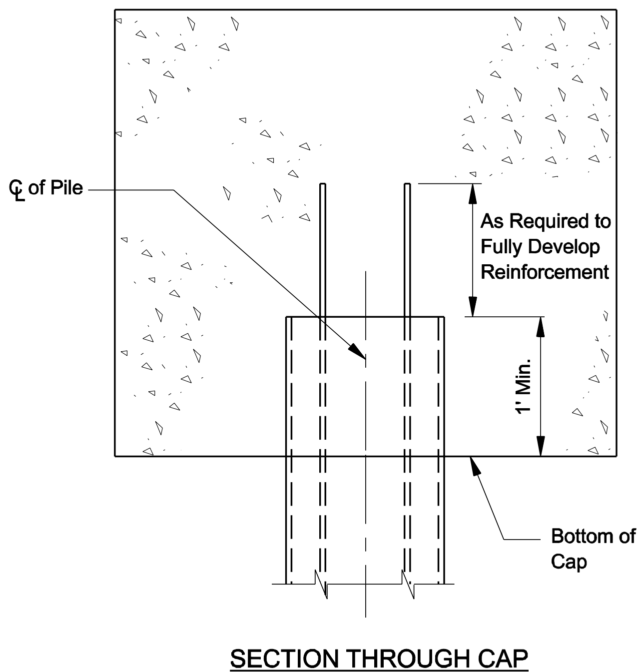
17.3.2.6 Pile Connection Details

The following applies to the connection of piles to pile caps or to bent caps unless seismic analysis dictates otherwise:

1. Steel Pipe Piles. The reinforcing steel must be extended into the pile cap and fully developed through adequate development length or standard hooks. The reinforcing steel extends to the minimum tip elevation of the pile. See Figure 17.3-B.

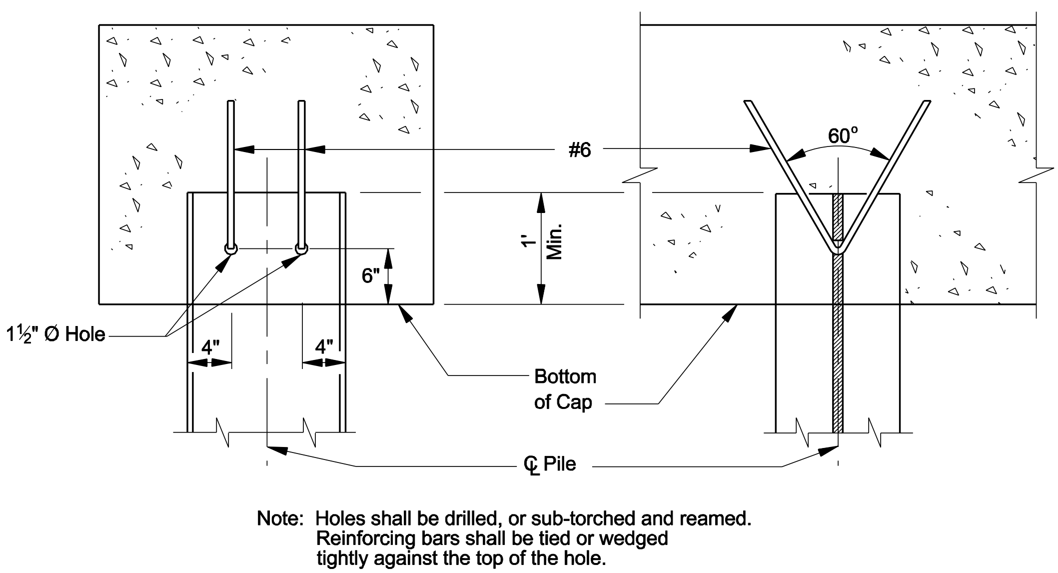
2. Steel H-Piles. Two V-shaped #6 reinforcing bars should be used to anchor steel piles to pile-supported footings or caps. The diameter of the hole should be 1½ in. The reinforcing bars shall be tied or wedged tightly against the top of the hole to reduce the possibility of slip between the reinforcing bar anchor and the pile. The reinforcing bars should extend into the cap or footing a minimum of 1′-8″ beyond the bottom mat of reinforcement. See Figure 17.3-C.

3. Prestressed Concrete Piles. The piles may be connected to the caps or footings by simply being embedded the larger of 1 ft or an equivalent of one pile width. No roughening of the pile is required. However, the pile surface to be embedded shall be clean and free of any laitance prior to placement of the cap or footing concrete.



STEEL PIPE PILE CONNECTION

Figure 17.3-B



STEEL H-PILE CONNECTION

Figure 17.3-C

17.3.3 Force Effects

17.3.3.1 Downdrag (DD) Loads

When a pile penetrates a soft layer subject to settlement, the designer must evaluate the force effects of downdrag or negative loading on the foundations. Downdrag acts as an additional permanent axial load on the pile and may cause additional settlement. If the force is of sufficient magnitude, structural failure of the pile or a bearing failure at the tip is possible. For piles that derive their resistance mostly from end bearing, the structural resistance of the pile must be adequate to resist the factored loads including downdrag.

Downdrag forces can be mitigated by the following methods:

- provide friction-reducing material around the piles;
- construct embankments a sufficient amount of time in advance of the pile driving for the fill to settle; or
- prebore and backfill the space around the installed pile with pea gravel (may be less effective if the adjacent soil continues to settle).

17.3.3.2 Uplift Forces

Uplift forces can be caused by lateral loads, buoyancy or expansive soils. Piles intended to resist uplift forces should be checked for resistance to pullout and structural resistance to tensile loads. The connection of the pile to the cap or footing must also be checked.

17.3.3.3 Laterally Loaded Piles

The resistance of laterally loaded piles must be estimated according to approved methods. Several methods exist for including the effects of piles and surrounding soil into the structural model for lateral loadings including seismic loads. NDOT's preferred method is discussed in Section 17.5.

17.3.3.4 Group Effect

Minimum spacing requirements are not related to group effect. Group effects are specified in LRFD Articles 10.7.3.7.3 and 10.7.3.10.

17.3.4 Pile Loads

17.3.4.1 Contract Documents

Applicable pile loads shall be shown in the contract documents. See Chapter 5. This information will help ensure that pile driving efforts during construction will result in a foundation adequate to support the design loads.

17.3.4.2 Static Load Tests

Reference: LRFD Article 10.7.3.8.2

NDOT occasionally performs static load tests on driven piles. The Geotechnical Section will determine the number and location of the static load tests. Test locations and sizes should be shown in the contract documents.

17.3.4.3 Dynamic Pile Monitoring

Reference: LRFD Article 10.7.3.8.3

During the installation of production piles, dynamic pile monitoring ensures that driving occurs in accordance with the established criterion. It provides information on soil resistance at the time of monitoring and on driving performance. Dynamic pile monitoring also reveals driving stresses, which helps prevent pile damage. If damage is imminent, the monitoring provides an alert early enough to save the pile from complete destruction.

Data obtained during pile-driving monitoring is used to verify pile resistance with CAPWAP.

17.4 DRILLED SHAFTS

Reference: LRFD Article 10.8

17.4.1 Usage

Section 11.7.2 presents NDOT practices for selecting drilled shafts as the foundation type. Drilled shafts should also be considered to resist large lateral or uplift loads where deformation tolerances are relatively small.

Drilled shafts derive load resistance either as end-bearing shafts transferring load by tip resistance or as friction shafts transferring load by side resistance or a combination of both. Friction-only shafts are the most desirable but may not be the most economical. Drilled shafts are typically good for seismic applications.

17.4.2 Drilled Shaft Axial Compressive Resistance at the Strength Limit State

The *LRFD Specifications* provides procedures to estimate the axial resistance of drilled shafts in cohesive soils and cohesionless soils in LRFD Articles 10.8.3.5.1 and 10.8.3.5.2. In both cases, the resistance is the sum of the shaft and tip resistances. LRFD Article 10.8.3.5.4 discusses the determination of axial resistance of drilled shafts in rock.

17.4.3 Structural Design

The following will apply to the design of drilled shafts:

1. Column Design. Because even soft soils provide sufficient support to prevent lateral buckling of the shaft, drilled shafts surrounded by soil may be designed according to the criteria for short columns in LRFD Article 5.7.4.4 when soil liquefaction is not anticipated. If the drilled shaft is extended above ground to form a pier, it should be analyzed and designed as a column. Similarly, the effects of scour around the shafts must be considered in the analysis.

2. Casing*.* A casing may be used to maintain the excavation, especially when placing a shaft within the water table. This casing, if left in place after construction, shall not be considered in the determination of the structural resistance of the shaft. However, it should be considered when evaluating the seismic response of the foundation because the casing will provide additional resistance.

3. Lateral Loading. Section 17.5 discusses the analysis of drilled shafts for lateral loading and resistance.

17.4.4 Design Details

The following details apply to the design of drilled shafts:

1. Location of Top of Shaft. Drilled shafts are normally terminated 1 ft to 2 ft below finished grade.

2. Reinforcement. Section 14.3 discusses NDOT practices for the reinforcement of structural concrete. The design of drilled shafts shall meet all applicable requirements in Section 14.3. Additional reinforcement criteria include:

   - The shaft will have a minimum reinforcement of 1% of the gross concrete area and the reinforcement will extend from the bottom of the shaft into the footing.

   - For confinement reinforcement, use spirals (up to #7) or butt-welded hoops.

   - The design and detailing of drilled shafts must conform to the clearances for reinforced steel cages as specified in the *NDOT Standard Specifications*:

     + 4 in for drilled shafts having a diameter of less than 5 ft, or
     + 6 in for drilled shafts having a diameter of 5 ft or more.

     Non-corrosive rollers will ensure that the annular space around the cage is maintained.

   - Detail drilled shafts and columns to accommodate concrete placement considering the multiple layers of reinforcing steel including lap splices. Maximize lateral reinforcement spacing. Consider recommendations from the Association of Drilled Shaft Contractors.

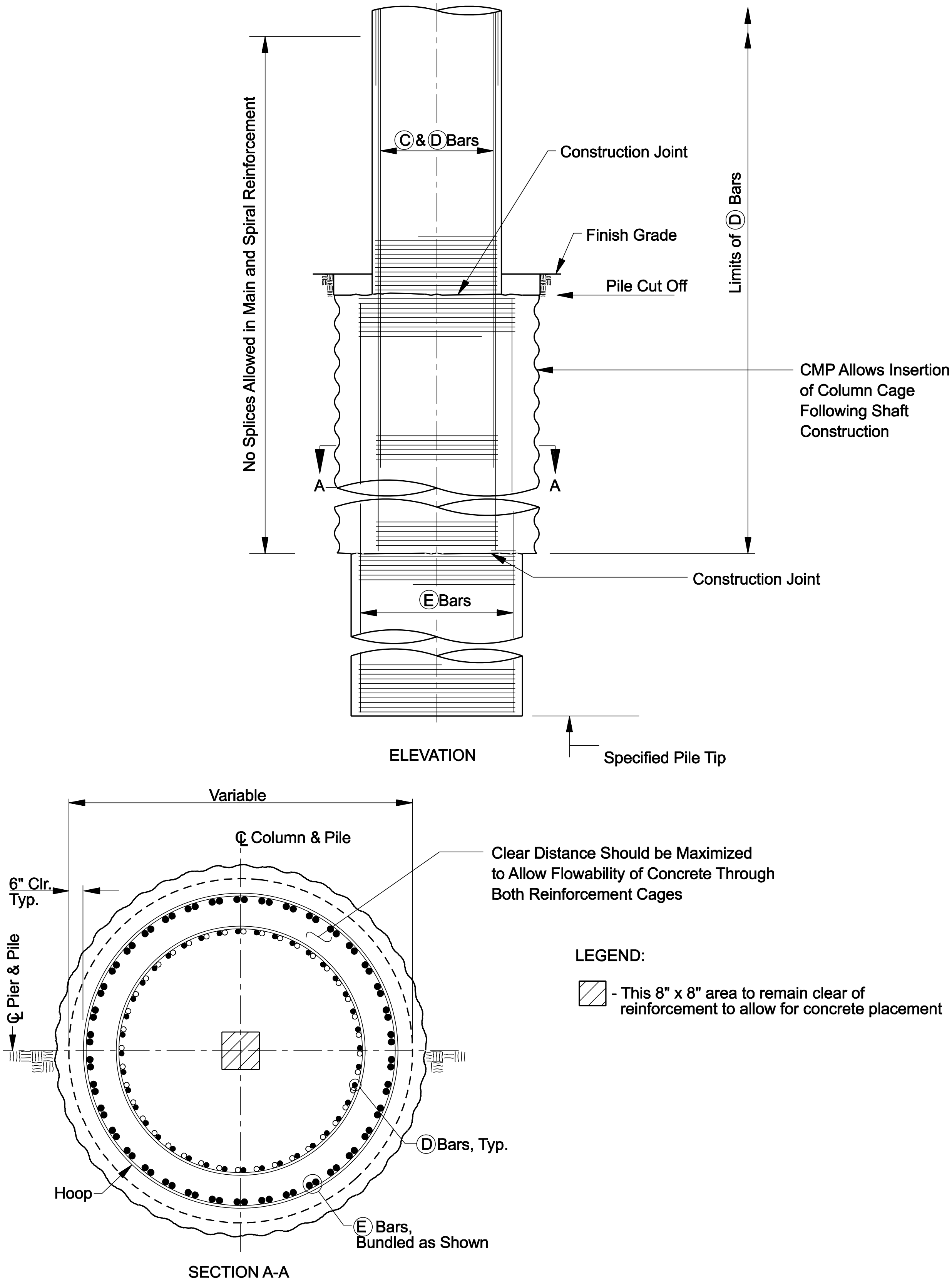
   Figure 17.4-A illustrates the typical drilled shaft and column longitudinal and transverse reinforcement.

3. Construction Joints. Do not use keys in the design of construction joints for drilled shafts.

4. Diameter. The diameter of a drilled shaft supporting a single column shall be at least 1½ ft greater than the greatest dimension of the column cross section.

5. Constructibility. Detail drilled shafts and columns to accommodate concrete placement through the layers of reinforcing steel. Limit lap splices in the drilled shaft locations and provide adequate openings.

6. Casing. A permanent casing (typically CMP) is often used to facilitate insertion of the column cage into the upper portion of the shaft after the shaft concrete has been placed up to the first construction joint. See Figure 17.4-A.



DRILLED SHAFT DETAIL

Figure 17.4-A

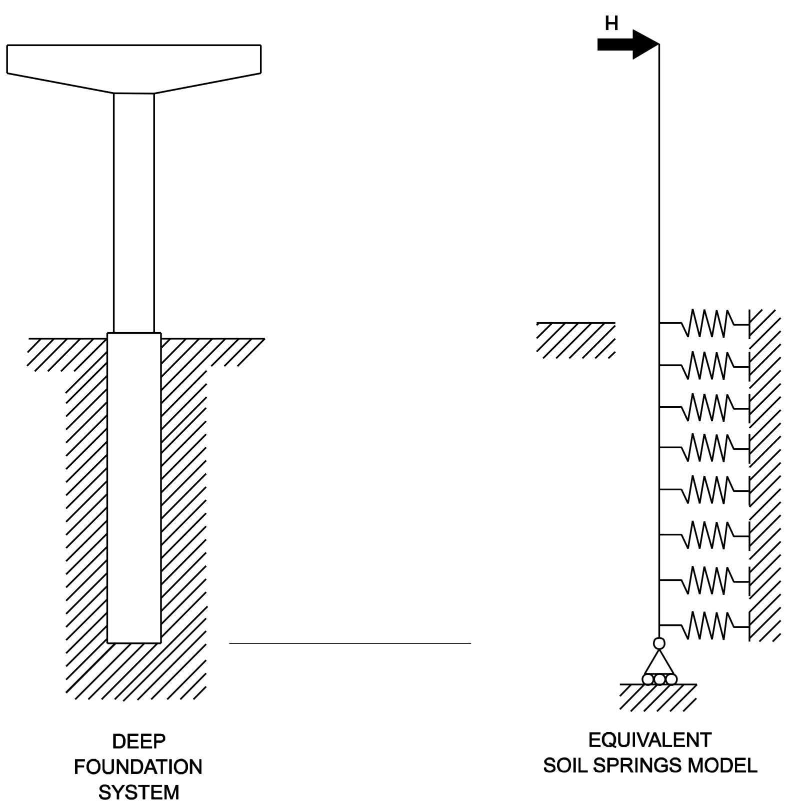
17.5 MODELING FOR LATERAL LOADING

In the initial stages of design, when using driven piles or drilled shafts, estimate the preliminary point-of-fixity at the top of the pile (bottom of the column).

For final design, a structural model with site-specific p-y curves is used to represent the soil and determine the lateral resistance of piles or shafts. The soil surrounding the pile is modeled as a set of equivalent non-linear soil "springs," as represented in Figure 17.5-A. Figure 17.5-B shows a set of typical p-y curves. The soil resistance "p" is a non-linear function of the corresponding horizontal pile deflection "y." The solution's accuracy is a function of the spacing between nodes used to attach the soil springs to the pile (the closer the spacing, the better the accuracy), and the pile itself. Simple girder column elements are usually adequate for modeling pile behavior.

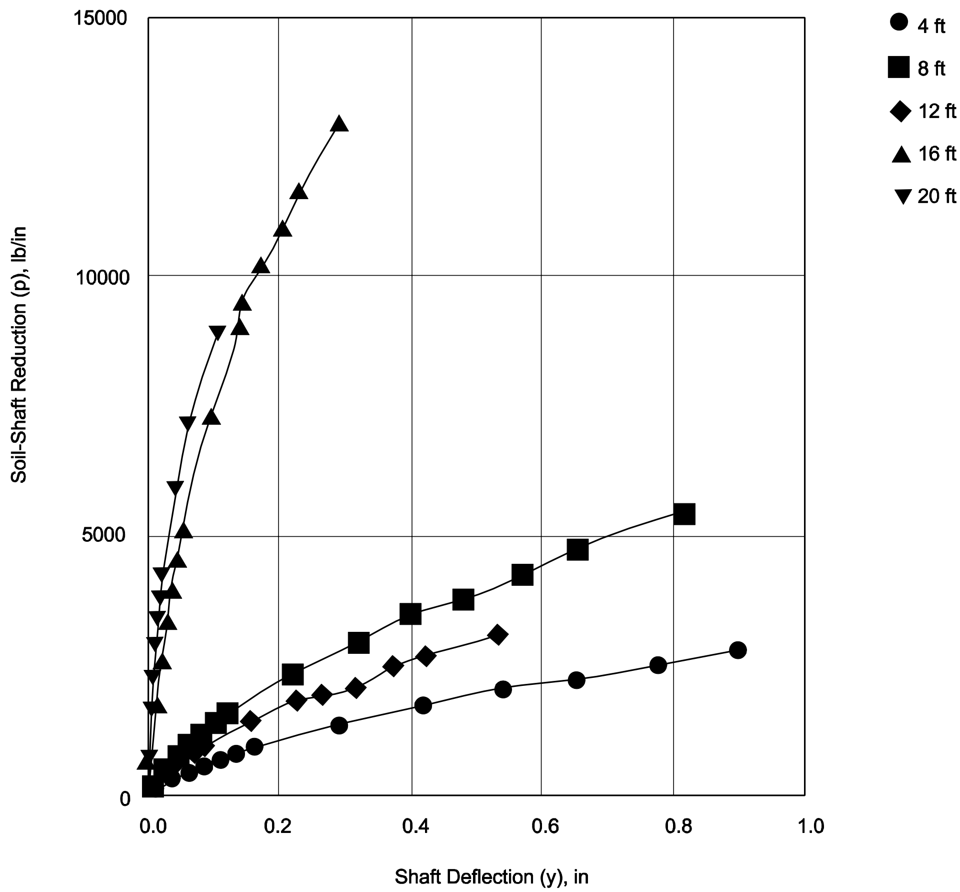
The node placement for springs should model the soil layers. Generally, the upper ⅓ of the pile in stiff soils has the most significant contribution to the lateral soil reaction. Springs in this region should be spaced at no more than 3 ft apart. Springs for the lower ⅔ of the pile may transition to a much larger spacing. Stiff foundations in weak soils will transfer loads much deeper in the soil, and the use of more springs is advised.

NDOT uses computer software (e.g., StrainWedge, LPILE Plus, COM624P) to model soil-structure interaction. The interaction between the Structures Division and the Geotechnical Section is discussed in Sections 17.1.4.3.2 and 17.1.4.3.3.



METHOD OF MODELING DEEP FOUNDATION STIFFNESS

Figure 17.5-A



EXAMPLE p-y CURVE

Figure 17.5-B

Chapter 18

SUBSTRUCTURES

NDOT STRUCTURES MANUAL

September 2008

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Chapter 18
SUBSTRUCTURES

Section 11 of the *LRFD Bridge Design Specifications* discusses design and detailing requirements for abutments, piers and walls. This Chapter presents NDOT supplementary information on the design of these structural elements. Section 11.6 of the *NDOT Structures Manual* presents NDOT criteria for the selection of substructure components within the context of structure-type selection.

18.1 ABUTMENTS/WINGWALLS

18.1.1 General

See Section 11.6.2 for NDOT practices on the selection of an abutment type.

An abutment includes an end diaphragm, a stem wall and wingwalls. A stem wall or diaphragm functions as a wall providing lateral support for fill material on which the roadway rests immediately adjacent to the bridge. Abutments shall generally be of the cast-in-place, reinforced concrete type and shall be founded on spread footings, drilled shafts or driven pile footings.

Do not use MSE wall abutments except when approved by the Chief Structures Engineer. When an MSE wall abutment is allowed, the abutment shall be founded on piles and not on a spread footing. The piles should be isolated from the MSE backfill to eliminate downdrag and should be founded in the soils below the MSE wall. Section 23.2 discusses the use and design of MSE walls in more detail.

18.1.2 Static and Quasi-Static Loads

Reference: LRFD Articles 3.11, 11.6.1.1 and 11.6.1.3

The static earth pressure shall be determined in accordance with LRFD Article 3.11.

18.1.3 Seismic Analysis and Design

Reference: LRFD Article 11.6.5

18.1.3.1 General

NDOT uses a dynamic analysis for multi-span structures. A dynamic analysis is not required for single-span bridges.

For single-span structures, the longitudinal seismic design force at the abutments is the structure weight multiplied by the acceleration coefficient. Because only one abutment resists this force at any time, each abutment is designed to resist the entire seismic force.

For single-span structures, the transverse seismic design force at the abutment is one-half the structure weight multiplied by the acceleration coefficient. Concrete shear keys located on the outside edges of the bridge resist the seismic forces. When seismic forces are high or the abutment is too wide for the external keys to resist all the force, internal shear keys may be provided. Internal shear keys are undesirable because they are extremely difficult to repair after a seismic event.

Soil springs are used to model foundation stiffness. The foundation springs are included in both the longitudinal and transverse directions. Because no soil resistance is mobilized when the structure moves away from the soil, one-half of the total longitudinal stiffness is modeled at each abutment. As a result, the resulting forces at each abutment should be added together to determine the abutment design force.

An abutment must resist the seismic active soil pressure and any force-transmitted from the superstructure elastically. Longitudinal and transverse shear keys are conservatively designed to resist total seismic abutment forces. In addition, the backwalls are assumed to fail, and the columns are conservatively designed to take all of the longitudinal seismic force.

In general, typical NDOT practice in seismically active areas is to design abutments and wingwalls for reduced seismic pressures corresponding to 2.0 in to 4.0 in of displacement. However, the amount of tolerable deformation will depend on the nature of the wall, what it supports, and what is in front of the wall.

NDOT has adopted a soil stiffness of 70 ksf per ft of movement for dynamic modeling. This stiffness is based on test results for large movements and is applicable for displacements in the range of 1 in to 3 in. Computations of abutment stiffness with soil pressures can be found in the FHWA *Seismic Design of Highway Bridges Training Course Workbook*. The procedures described therein may be used with the 70-ksf soil stiffness value.

18.1.3.2 Integral and Dozer Abutments

Typical NDOT diaphragm-with-footing abutments are constructed with a free-sliding diaphragm on a spread footing or pile cap. The abutment is constructed with a constant width gap (normally 1 in), filled with expansion material, between the abutment diaphragm and footing shear keys. The frictional force between the diaphragm-footing interface at the opposite abutment is conservatively neglected. An analysis is performed using an initial estimate of foundation stiffness equal to the stiffness of the backfill soil. The resulting abutment displacements are checked and compared to the width of the gap. If the predicted displacements are equal to or less than the gap width, the model is deemed satisfactory and the overall design proceeds. If the predicted displacements exceed the gap width, the shear keys are engaged and the structure is reanalyzed with increased foundation stiffness. This process is repeated until the displacements are close to the gap width. Resulting soil pressures behind the abutment wall are checked against the recommended ultimate soil capacity of 5 ksf for granular backfill material. If the calculated soil pressure exceeds the ultimate capacity and the shear keys have not been engaged, a reduced effective stiffness equal to the ultimate capacity divided by the displacement is used.

Diaphragm-with-pile and dozer abutments are modeled in a similar manner with the exception that multiple iterations are not necessary, provided that the maximum values of abutment displacement and backfill soil pressure are not exceeded. Pile stiffness is added to the soil stiffness to compute the foundation stiffness of diaphragm-with-pile abutments.

The assumption is made that, for diaphragm-with-footing abutments, the expansion joint material used to fill the gap between the abutment diaphragm and footing shear keys crushes under load to one quarter of its original thickness. The gap width between the shear keys and abutment diaphragm should be set at twice the expected thermal displacement with a 1-in minimum width at the back face of the diaphragm and ½-in minimum at the sides. The effects of long-term elastic shortening must be considered for setting the gap width on prestressed concrete structures.

Reinforcing steel in the front face of diaphragm-with-footing stem wall is designed to resist a minimum frictional force at the diaphragm-footing interface where the diaphragm slides away from the abutment fill. A minimum force equivalent to 15% of the dead-load reaction at the top of footing is assumed. This represents the force needed to overcome the friction between the stem wall and footing caused by temperature or seismic movements. Serviceability requirements are also checked for this loading.

Reinforcing is provided in the back face of the integral abutment stem walls and dozer abutment extensions to resist static and dynamic forces from the abutment backfill. For the static load combinations, the diaphragm is designed for the passive soil pressure behind the abutment. The minimum frictional force described above must also be included for diaphragm-with-footing abutments. Design loading for the dynamic load combination consists of the resulting soil pressure due to the seismic abutment shear force. The design force may be conservatively applied at the bottom of the stem wall. Although integral abutments are designed using elastic methods, stirrups are provided in the upper half of the stem walls below the superstructure to resist potential plastic hinging forces. Longitudinal and transverse shear keys are conservatively designed to resist total seismic abutment forces. For footings supported on piles, shear keys should have a resistance that is less that 75% of the pile resistance.

18.1.3.3 Seat Abutments

For the dynamic analysis of continuous structures, abutment spring constants are used to model the movement needed to engage shear keys and restrainers only; otherwise, the abutment is considered a fixed, pinned or free-support, depending on the degree of freedom being considered.

The stem wall and back wall of a seat abutment are designed for a minimum equivalent fluid pressure of 36 pcf and for lateral seismic pressures.

18.1.4 <u>General Abutment/Wingwall Design and Detailing Criteria</u>

The following applies to the design and detailing of abutments and wingwalls:

1. <u>Granular Backfill</u>. See the *NDOT Standard Specifications for Road and Bridge Construction*.

2. <u>Expansion Joints</u>. Vertical expansion joints should be considered for wall lengths exceeding 125 ft in length. In this case, a water stop or other means of control shall be used to prevent leakage.

3. <u>Abutment Top Surfaces</u>. Abutment seats at bearing locations shall be level. For seat abutments, the remaining exposed top surfaces shall be transversely sloped to provide adequate drainage.

4. <u>Approach Slab Seat</u>. Provide a minimum 12-in approach-slab seat. Use restrainer units for all approach slabs that are not otherwise connected to the abutment or superstructure.

5. Dead Load. NDOT policy is to include one-half of the dead load of the approach slab as an abutment dead load.

6. Skewed Bridges. For skew angles greater than 30°, detail a 3-in minimum chamfer at acute corners.

7. Soil Reinforcements. Soil reinforcements (such as steel strips and bar mats commonly used in mechanically stabilized earth (MSE) wall construction) shall not be used as attachments to abutment diaphragms or stem walls in an attempt to resist lateral loads applied to these components.

8. Reinforcement. Section 14.3 discusses NDOT practices for the reinforcement of structural concrete (e.g., concrete cover, bar spacing, corrosion protection). The design of abutments and wingwalls shall meet all applicable requirements in Section 14.3.

18.1.5 Integral Abutments

Reference: LRFD Article 11.6.2.1

18.1.5.1 Usage

Section 11.6.2 discusses NDOT practices for the use of integral abutments. In addition, their use should be limited to a maximum height of 9 ft as measured from the bottom of the soffit to the bottom of the wall, or twice the depth of the superstructure, whichever is less.

18.1.5.2 Diaphragm-With-Footing

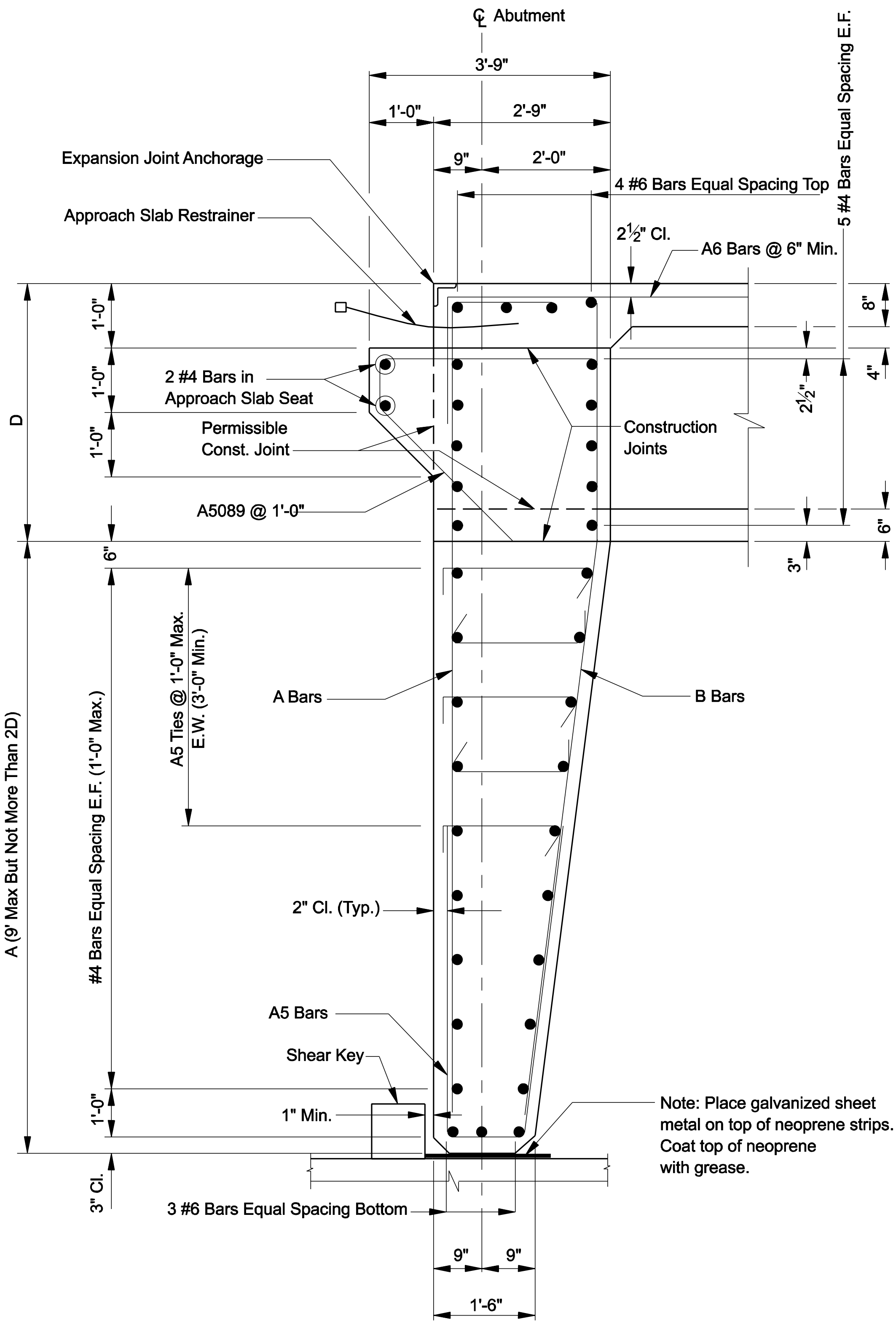
Figure 18.1-A shows a typical diaphragm-with-footing abutment. Diaphragm-with-footing abutment details shall meet the following requirements:

1. Abutment Height. If the extension of the end diaphragm down to the footing (shown as “A” in Figure 18.1-A) exceeds two times the superstructure depth (shown as “D” in the Figure), the bridge should be modeled as a rigid frame.

2. Superstructure Flexural Resistance. Provide flexural resistance at the end of the superstructure equal to 130% of the diaphragm flexural resistance (#6 bars @ 6 in as a minimum).

3. Size and Spacing of A and B Bars. The size and spacing of the A and B bars shown in Figure 18.1-A shall be determined by design.

4. Top and Bottom Slab Reinforcing. Extend the top and bottom slab reinforcing through the abutment diaphragm.



TYPICAL DIAPHRAGM-WITH-FOOTING ABUTMENT

Figure 18.1-A

18.1.5.3 Diaphragm-With-Driven-Pile

Figure 18.1-B shows a typical diaphragm-with-driven-pile abutment, which is only appropriate for driven piles and should not be used with drilled shafts. Diaphragm-with-driven-pile abutment details shall meet the following requirements:

1. Pile Embedment. To provide pile fixity, the pile connection details of Section 17.3.2.6 shall be used.

2. Size and Spacing of A and B Bars. The size and spacing of the A and B bars shown in Figure 18.1-B shall be determined by design.

3. Top and Bottom Slab Reinforcing. Extend the top and bottom slab reinforcing through the abutment diaphragm.

18.1.6 Seat Abutments

Figures 18.1-C and 18.1-D show typical short and high cantilever seat abutments. Seat abutment details shall meet the following requirements:

1. Seat Width. A seat width, N, of 2′-6″ is typical for post-tensioned box girders. Increase as required for anchor head embedments and seismic design requirements.

2. Stem Width. The minimum stem width for T is 2 ft. Increase as required by design.

3. Size and Spacing of Bars. The size and spacing of the A, B, C, D, E, F and G bars shown in Figures 18.1-C and 18.1-D shall be determined by design with a minimum of #5 bars @ 12 in unless noted otherwise.

4. Dimension “a”. The dimension “a” in Figures 18.1-C and 18.1-D shall be as required for the expansion joint with a minimum of 1½ in.

5. Batter. Walls are of constant thickness; battered walls shall not be used except for high cantilever abutments.

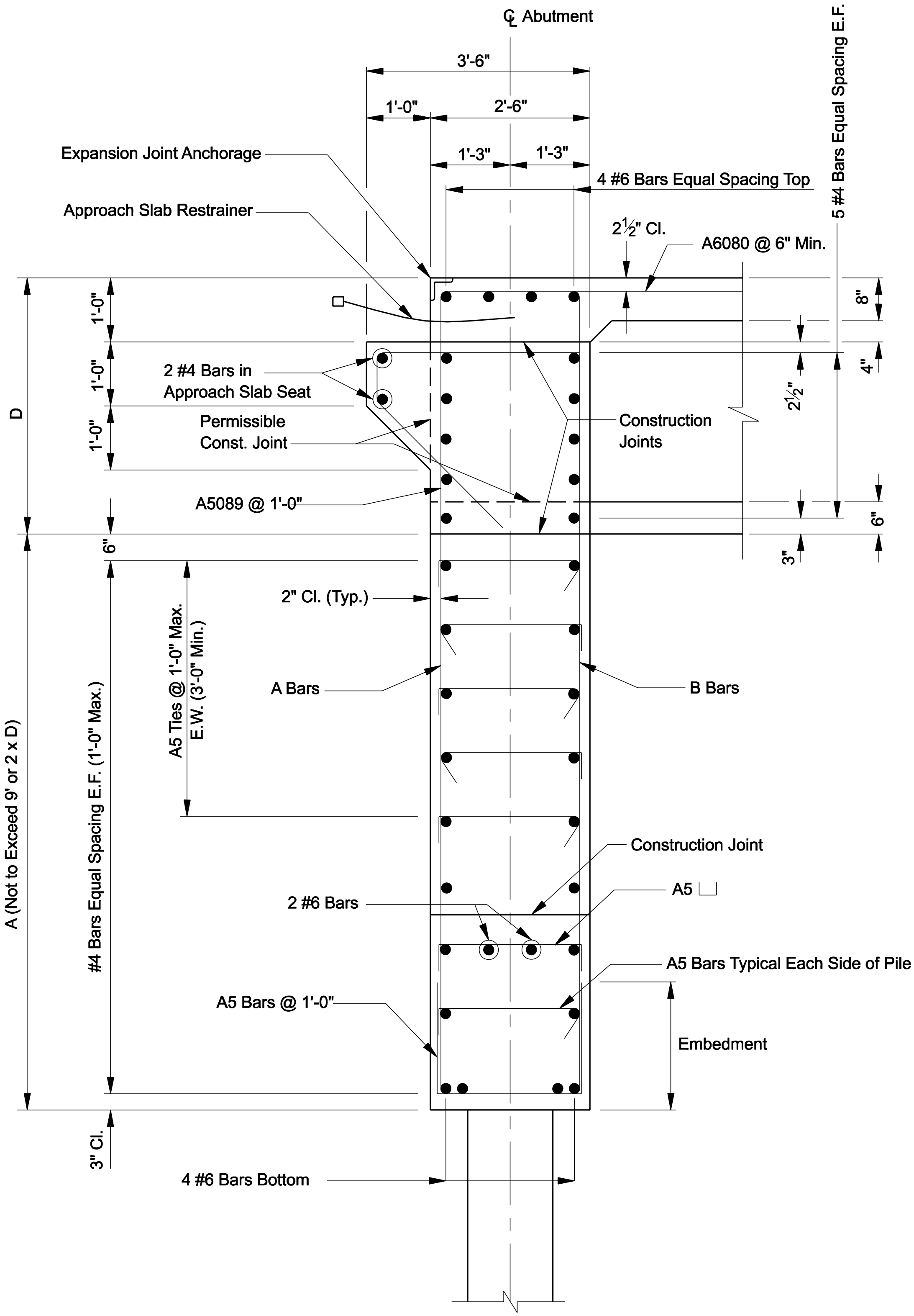
18.1.7 Dozer Abutments

Figure 18.1-E shows a typical dozer abutment. Dozer abutment details shall meet the following requirements:

1. Superstructure Flexural Resistance. Provide flexural resistance at the end of the superstructure equal to 130% of the diaphragm flexural resistance (#6 bars @ 6 in as a minimum).

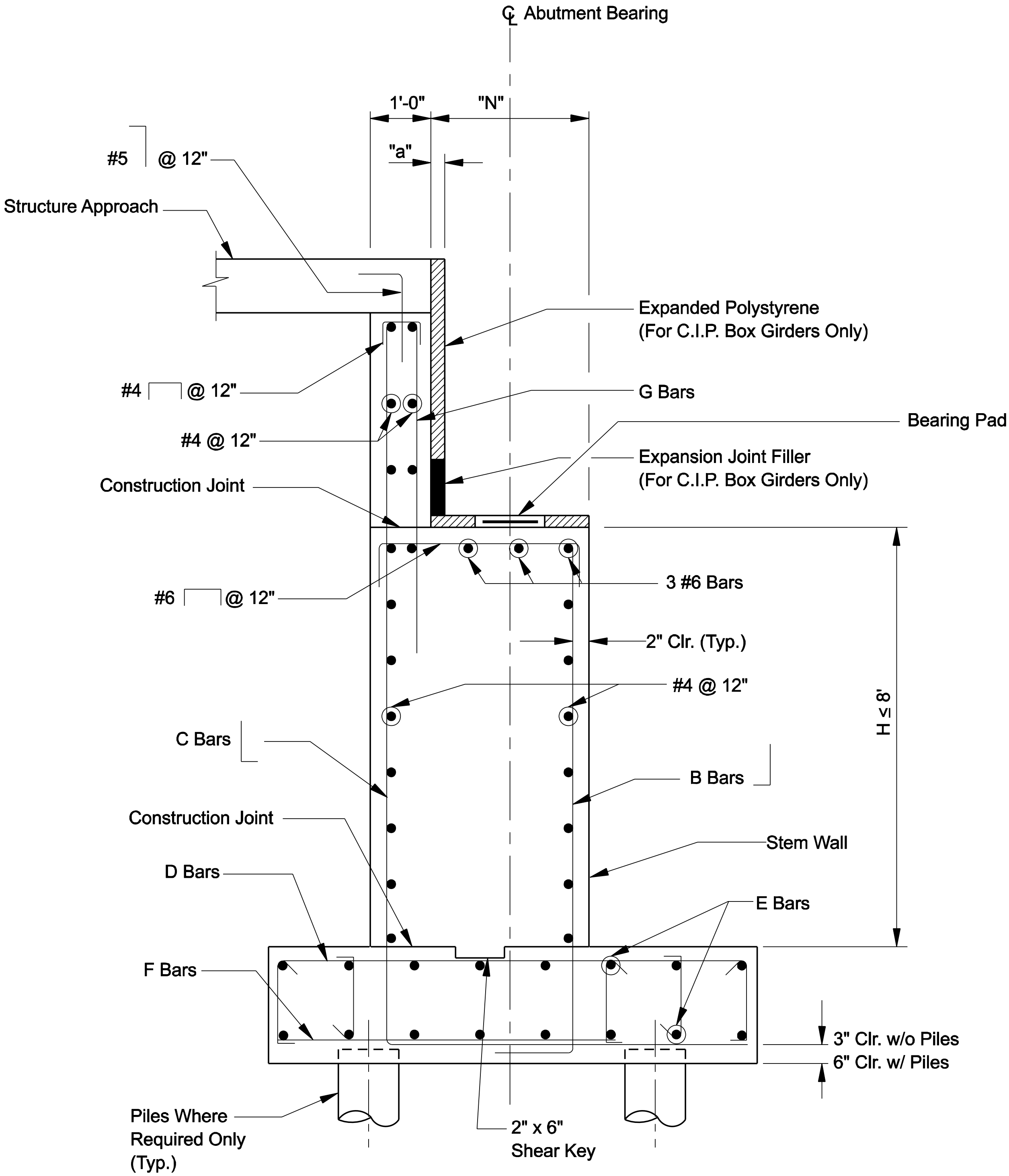
2. Size and Spacing of A and B Bars. The size and spacing of the A and B bars shown in Figure 18.1-E shall be determined by design.

3. Taper. The taper of the diaphragm extension may be omitted where its extension below the superstructure (shown as “A” in Figure 18.1-E) is less than 3 ft. In this case, form a 6-in gap between the extension and the stem wall with expanded polystyrene filler.



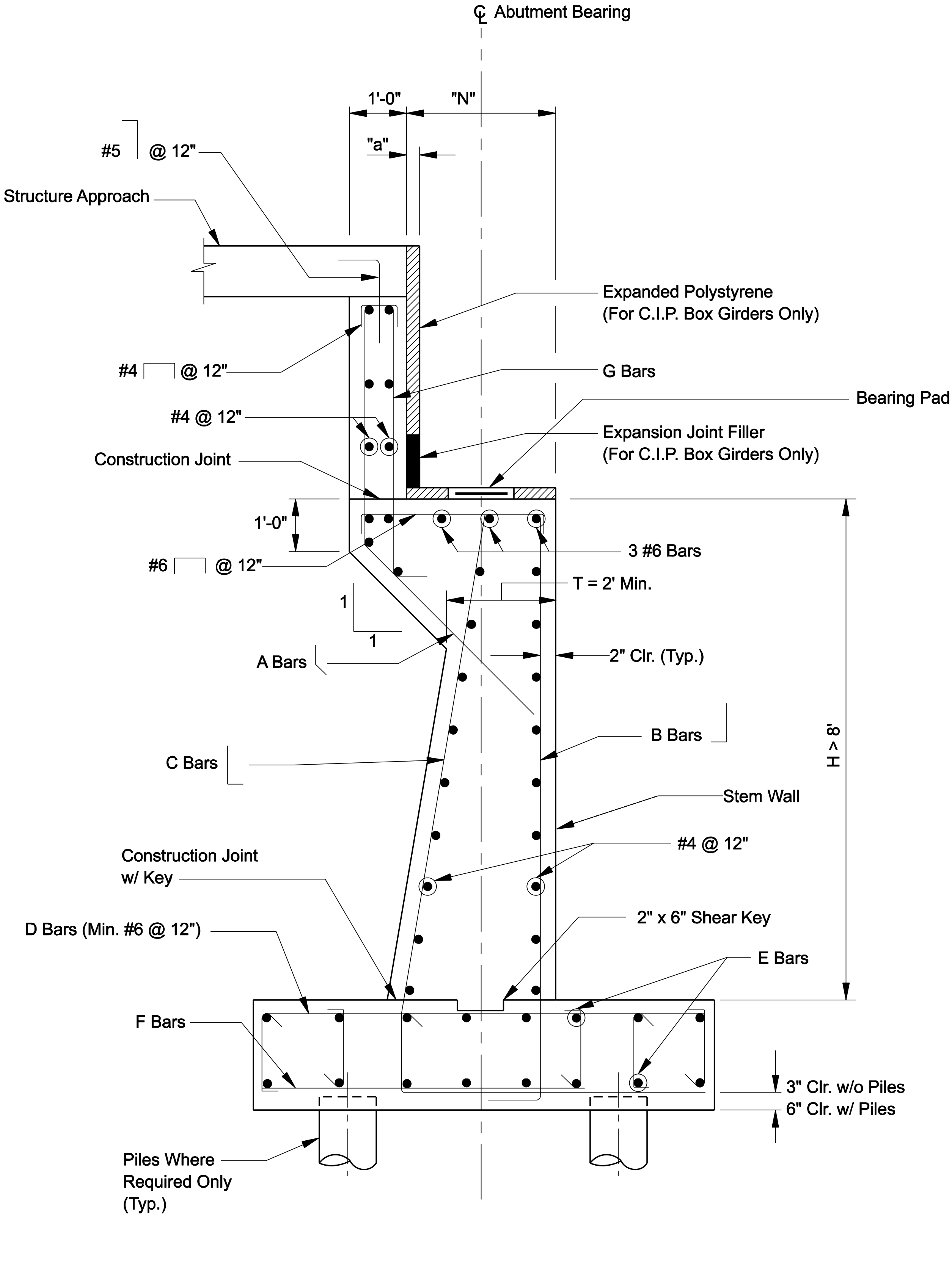
TYPICAL DIAPHRAGM WITH DRIVEN PILE ABUTMENT

Figure 18.1-B



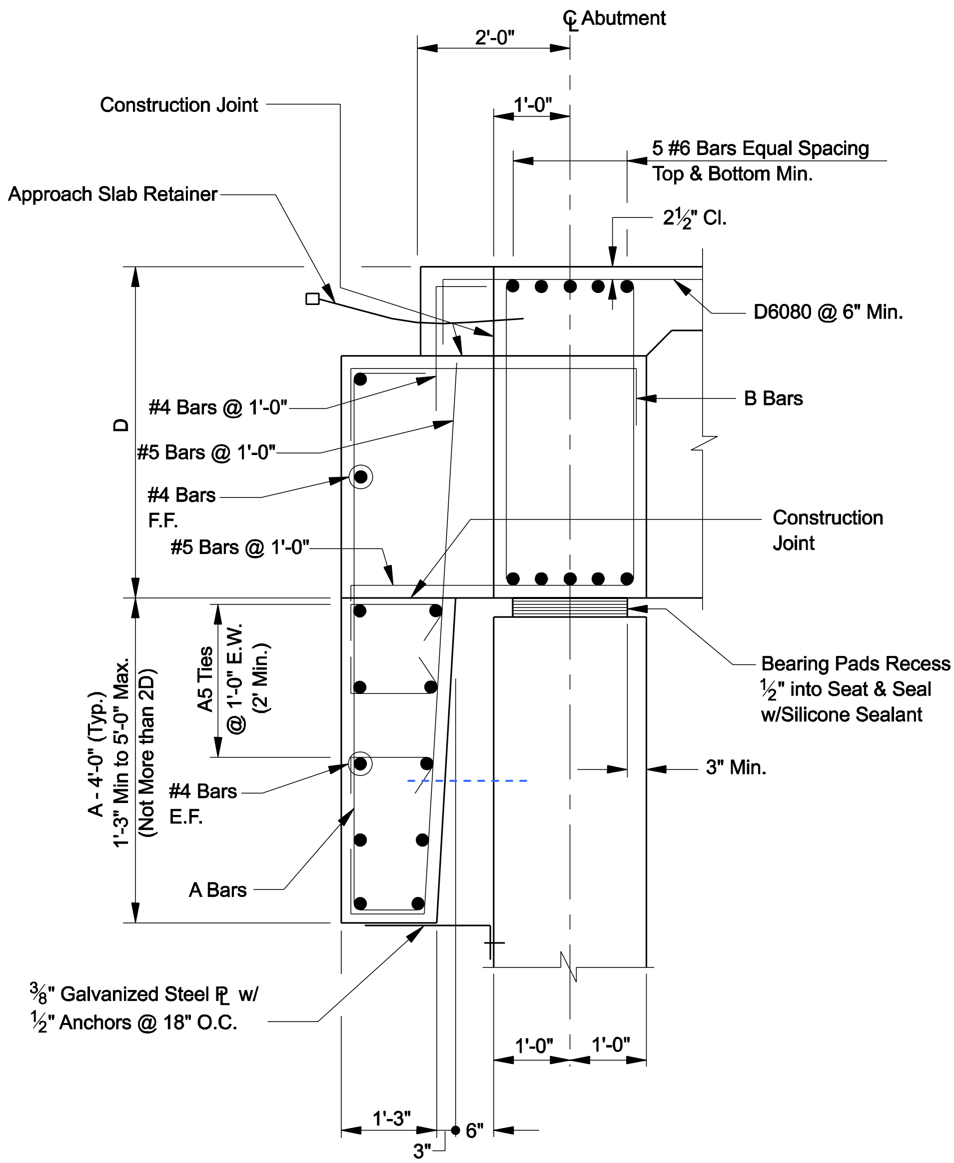
TYPICAL SHORT SEAT ABUTMENT

Figure 18.1-C



TYPICAL HIGH CANTILEVER ABUTMENT

Figure 18.1-D



TYPICAL DOZER ABUTMENT

Figure 18.1-E

18.1.8 Piles

Reference: LRFD Article 10.7

This discussion specifically addresses the use of driven piles with abutments. See Section 17.3 for additional information on piles.

18.1.8.1 General

The following criteria apply to piles for all abutments:

1. Number. The number of piles supporting an abutment shall not be less than three.

2. Batter. Vertical piles shall be used at integral abutments and are preferred at dozer and seat abutments. If battered piles are used, a refined analysis is required.

3. Construction. Consider the placement tolerances for all abutment types and ensure pile fit within the cap dimensions and relative to reinforcing steel.

18.1.8.2 Piles for Diaphragm-With-Driven-Pile Abutments

A single row of piles shall be used.

18.1.8.3 Piles for Seat Abutments

The following criteria apply to piles for seat abutments:

1. Pile Spacing. Consider using two rows of piles to achieve the necessary longitudinal stiffness. The minimum pile spacing is 30 in or 2½ times the pile width or as recommended by the Geotechnical Section.

2. Movement. The designer shall investigate the effects of movements due to overturning pressures or lateral pressures (e.g., ensure that the closing of joints does not occur).

18.1.9 Wingwalls

Reference: LRFD Article 11.6.1.4

Wingwalls shall be of sufficient length to retain the roadway embankment and to furnish protection against erosion. See Figure 11.6-B for the geometry of wingwalls.

With respect to abutments, the following applies to wingwalls:

1. Orientation. Typical NDOT practice is to use "parallel" wingwalls (i.e., wingwalls that are parallel to the centerline of the bridge). Occasionally, bridge or roadway geometric constraints will require the use of "perpendicular" wingwalls (i.e., wingwalls that are perpendicular to the centerline of bridge). "Flared" wingwalls are only used in combination with box culverts.

2. Thickness. The minimum thickness of any wingwall shall be 12 in.

3. Longer Parallel Wingwalls. In general, parallel wingwalls should not extend more than 20 ft behind the rear face of the abutment. If parallel wingwalls on seat abutments have a total length of more than 20 ft, investigate the use of an unattached wingwall.

4. Cantilevered Wingwalls. Use cantilevered wingwalls only on integral abutments.

5. Wingwall/Abutment Connection. The junction of the abutment and wingwall is a critical design element, requiring special considerations. Typical NDOT practice is to use a 2-ft triangular fillet at the junction of the back of the abutment and wingwall. Use fillet reinforcement with a minimum of #6 reinforcing bars at 12-in spacings properly anchored into the wingwall and abutment (see the *NDOT Bridge Drafting Guidelines*).

6. Design Forces. The design forces for wingwalls are due to earth pressure only. It is NDOT practice to extend the approach slabs over the wingwall, which eliminates the live load surcharge in the design of the wingwall. Seismic forces from the soil behind the wingwall must also be considered in the design of wingwalls.

7. Unattached Wingwalls. Unattached wingwalls shall be designed as retaining walls. Unattached wingwalls are generally cast-in-place concrete retaining walls. Provide an expansion joint between the unattached wingwall and abutment. See Section 23.1 for NDOT practices on retaining walls.

18.1.10 Abutment Construction Joints

To accommodate normal construction practices, the designer should detail the following horizontal construction joints in the contract documents:

1. Integral Abutments. A horizontal construction joint shall be detailed at the top of the end diaphragm at the joint with the soffit. See Figures 18.1-A and 18.1-B for this mandatory and other optional construction joints.

2. Seat Abutments. A horizontal construction joint shall be detailed between the top of the abutment seat and the bottom of the backwall. Some expansion joint types may require another construction joint at the approach slab seat.

3. Wingwalls. A permissible horizontal construction joint shall be detailed at an elevation that is the same as the top of the abutment seat.

Planned vertical construction joints are normally associated with staged construction issues. Make provisions for splicing or mechanical reinforcing couplers on horizontal reinforcing steel. Vertical reinforcing steel should be at least 3 in from the construction joint.

18.1.11 Drainage

The *NDOT Bridge Drafting Guidelines* provides a detail for the design of weepholes, which are intended to provide positive drainage as needed in the embankment behind the abutment and wingwalls. Provide weepholes spaced every 15 ft horizontally and located 3 in to 6 in above finished grade.

18.2 PIERS

Reference: LRFD Article 11.7

18.2.1 Design Preferences

Desirably, the column design will be controlled by seismic loads and not other load combinations. The columns shall be conservatively designed to take all of the longitudinal seismic force assuming that the abutment backwalls fail. A longitudinal open joint should be used where transverse temperature controls the column design.

18.2.2 Pier Caps

18.2.2.1 Usage

In general, NDOT uses integral caps and drop pier caps supported by a single column, multiple columns or a solid pier wall. See Section 11.6.3 for more information.

Use an integral pier cap for cast-in-place concrete bridges and drop caps for steel and precast concrete girder bridges. An integral pier cap may be used for steel and precast concrete girder bridges where vertical clearance restrictions exist under the cap.

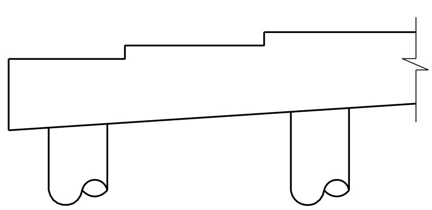
An outrigger cap is an integral cap that extends beyond the edge of the bridge superstructure. They are used where columns cannot be placed within the width of the bridge superstructure. The design shall consider the torsional effects resulting from longitudinal seismic displacements. A pin connection is preferred at the interface between the column top and bottom of the outrigger cap to minimize torsion in the cap.

18.2.2.2 Cap Width

The width of pier caps shall project beyond the sides of columns. The added width of the cap shall be a minimum of 3 in on each side of the column for a total of 6 in by which the cap is larger than the column. This width will reduce the reinforcement interference between the column and cap. The cap will also have short cantilevered ends, when practical, to balance positive and negative moments in the cap. These caps shall be designed to meet the deflection requirements of LRFD Article 2.5.2.6.2.

18.2.2.3 Drop Caps

The tops of drop caps shall be stepped as shown to account for elevation differences between girders:



The drop cap steps should be vertical, and the bearing surfaces should be level. For planar (superelevated) cross sections, the bottom of the cap shall be sloped at the same rate as the cross slope of the top of the bridge deck. For crowned sections, the bottom of the cap shall be level.

18.2.2.4 Integral Pier Cap

For integral pier caps, an additional layer of longitudinal reinforcement shall be placed approximately 3 in below the construction joint between the deck and cap, or lower if necessary to clear prestressing ducts. This reinforcement shall be designed for flexure using the Strength I load combination of LRFD Table 3.4.1-1, considering the dead load negative moment of that portion of the cap and superstructure located beneath the construction joint and within 10 ft of each side face of the cap. The Service limit states and shear design are not required for this condition. This reinforcement may be included in computing the flexural capacity of the cap only if a stress and strain compatibility analysis is made to determine the stress in the bars.

18.2.3 Column Cross Sections

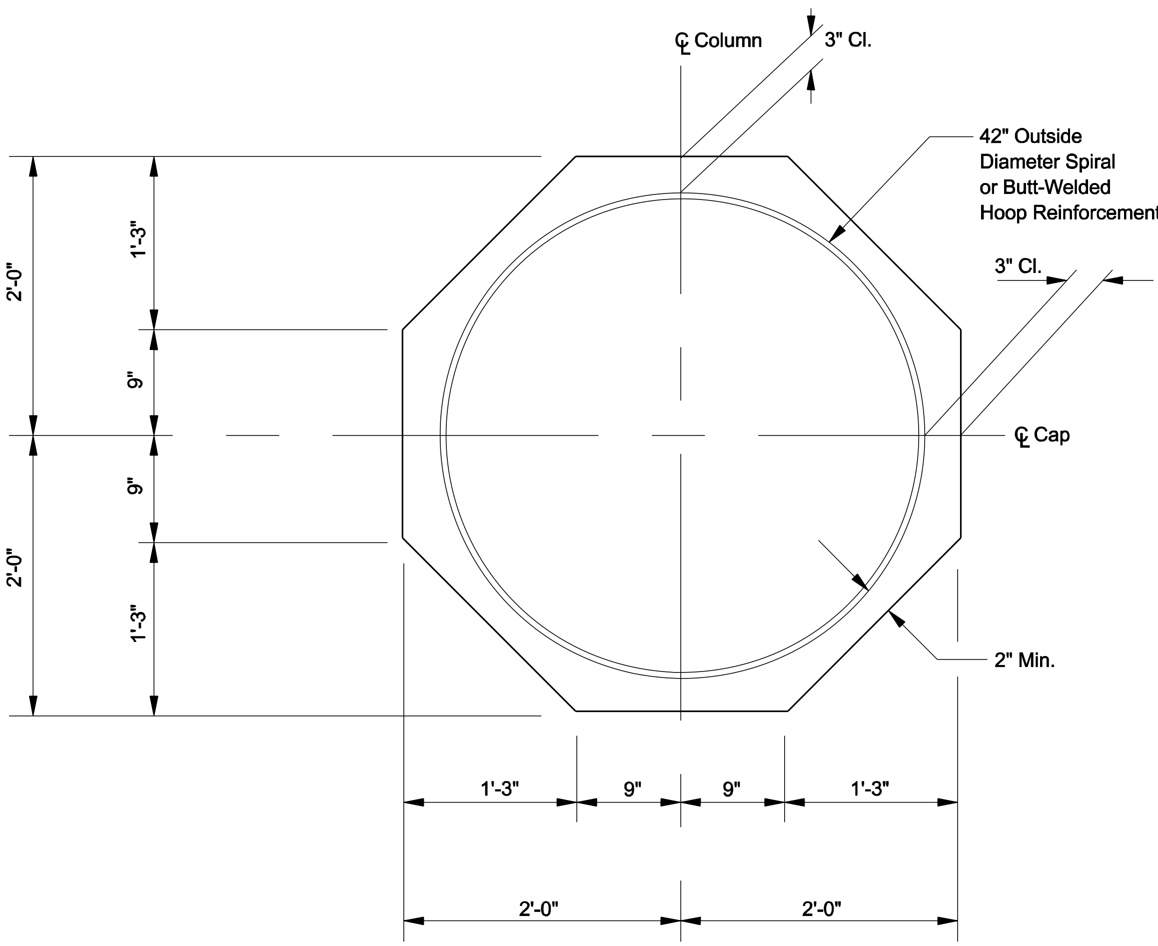
Reinforced concrete columns have traditionally been constructed with the octagonal shape shown in Figure 18.2-A. However, recently developed landscape aesthetic guidelines indicate a preference for round columns. Consult with the NDOT Landscape/Aesthetics Section for recommendations regarding column shape or other aesthetic features. Flared columns should not be used for new or replacement bridges. Flared columns may be considered for use on bridge widenings with the written approval of the Chief Structures Engineer.

For single-columns, the cross section is 4-ft thick with a variable width.

Where a pier wall is used, the wall shall be solid for its entire height. The minimum thickness is 2 ft (2′-6″ for railroad crash walls) and may be widened at the top to accommodate the bridge seat where required.

Caps shall be at least 6 in wider than the column’s greatest dimension. Seismic requirements for girder-seat widths may control cap width.

Where columns are supported on isolated drilled shafts, the shaft diameter is typically enlarged relative to the column to force plastic hinging in the column and protect the drilled shaft from inelastic action. The drilled shaft diameter is typically 18 in larger than the column diameter. The bridge designer must confirm that the diameters selected for the column and shaft will accommodate the overlapping reinforcing steel cages and clear cover requirements in both the column and drilled shaft. Refer to Figure 17.4-A in Chapter 17.



TYPICAL SECTION OF OCTAGONAL COLUMN

Figure 18.2-A

18.2.4 Column Reinforcement

18.2.4.1 General

Section 14.3 discusses NDOT practices for the reinforcement of structural concrete. This includes:

- concrete cover,
- bar spacing,
- lateral confinement reinforcement,
- corrosion protection,
- development of reinforcement, and
- splices.

The design of concrete pier columns shall meet all applicable requirements in Section 14.3.

18.2.4.2 Transverse Reinforcement

Reference: LRFD Article 5.10.11

18.2.4.2.1 *General*

Spirals or butt-welded spliced hoops shall be used as transverse reinforcing steel in octagonal or round columns. Ties are used in rectangular columns or shapes where spirals or hoops cannot be used. Reinforce columns with oblong cross sections and interlocking hoops with a center-to-center spacing not to exceed ¾ times the diameter of the cage. The overlaps shall be interlocked by a minimum of four bars. Figure 18.2-B illustrates the detailing of an oblong column.

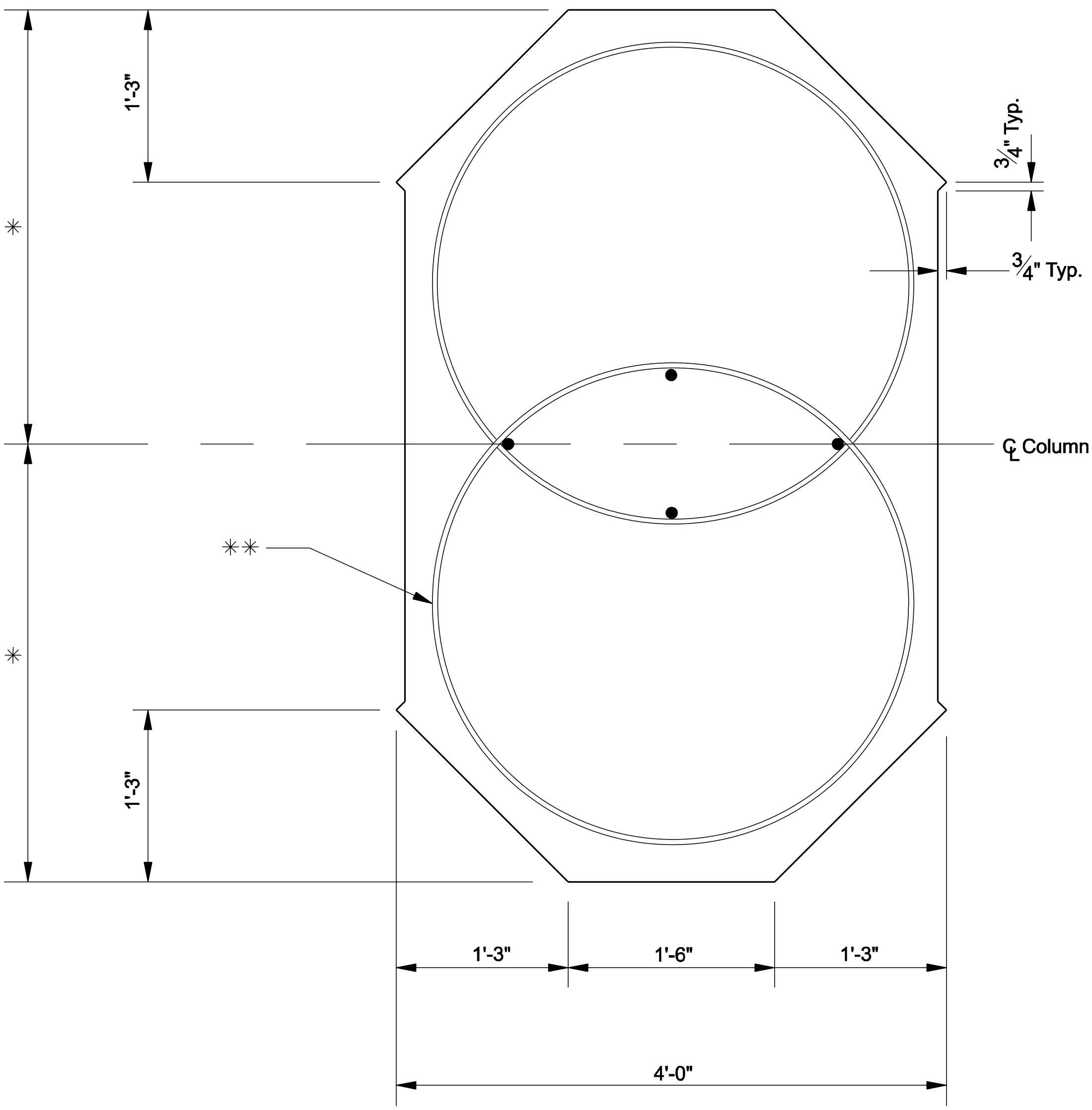
18.2.4.2.2 *Spiral Splices*

Almost all spiral reinforcement will require a splice. LRFD Article 5.10.11 provides requirements for splices in spiral reinforcement. The contract documents shall indicate plastic hinge regions where a spiral splice is not allowed. Refer to Section 14.3.1.8.2.

A lapped splice, where permitted, shall consist of an overlap distance of 60 bar diameters or 1½ column diameters whichever is more. The ends of both spirals shall terminate in a 135° hook, wrapped around a longitudinal bar, and having a tail length of at least 6 in. A detail or description of the lapped splice should be provided in the contract documents.

See Section 14.3.1.8 for a discussion on NDOT practices for welded and mechanical splices.

At locations where the spiral reinforcement extends into a footing or cap, the spiral reinforcement can be discontinuous. This allows easier placement of the top mat of footing or bottom mat of cap reinforcement. A detail or note shall be provided in the plans that shows an allowed discontinuity in the spiral with a splice.



✳ - Width of Column is Dependent Upon Design Requirements.

✳✳ - 42" Outside Diameter Spiral. Number of Spirals Depends on Width of Column. At Least 4 Vertical Bars Needed in Interlock Area.

TYPICAL SECTION OF MULTI-SPIRAL COLUMN

Figure 18.2-B

18.2.4.3 Longitudinal Reinforcement

Reference: LRFD Article 5.10.11

Longitudinal column reinforcing bars shall be #8 or larger, with #10 bars being the preferred minimum. Detail the longitudinal reinforcing steel continuous with a maximum spacing of 8 in center-to-center. The longitudinal column reinforcing bars must be fully developed where these bars enter into the pier cap and the spread footing, pile cap or drilled shaft.

The preferred detail for longitudinal reinforcement is continuous, unspliced reinforcement. A note on the plans should state that splices will not be allowed in the longitudinal reinforcement.

If column heights require splices, the provisions in LRFD Article 5.10.11 shall be used. Mechanical couplers or lap splices shall be used for splicing the longitudinal reinforcing steel. Do not locate splices within the plastic-hinge regions of the column. Refer to Section 14.3.1.8.2. A minimum stagger of 2′-0″ between adjacent splices shall be required and the locations shown in the plans. Splices in bundled bars shall also be staggered at a minimum of 2′-0″. If epoxy-coated bars are used, mechanical couplers shall be tested with reinforcing bars coated as required for the bridge, and the couplers shall be coated with a compatible coating.

Proposals by contractors to change the location or type of splice from those in the contract documents should not be allowed unless approved by the bridge designer. The resolution of conflicts or errors requires special consideration.

18.2.4.4 Compression Member Connection to Caps

Longitudinal bars should terminate at a point below the top cap reinforcement or prestressing ducts. If a hook is required, it should extend toward the compression member core. Minimum clearances must be maintained for the placement of cap concrete through tremies.

18.2.5 Column Construction Joints

Construction joints shall be used at the top and bottom of the column. Where columns exceed 25 ft in height, construction joints shall be shown such that concrete pours do not exceed 25 ft in height. Where applicable, locate all construction joints at least 12 in above the water elevation expected during construction.

18.2.6 Multi-Column Piers

Concrete frame piers may be used to support a variety of superstructures. The columns may be directly supported by individual footings, a combined footing or by drilled shafts. The following applies to the design and detailing of multi-column piers:

1. Column Spacing. In general, column spacing should not exceed approximately 25 ft center to center of columns.

2. Footings. Columns founded on spread footings have typically been designed with separate footings under each column. Existing analytical techniques provide tools for the analysis of a common footing for all columns, and this configuration may result in a more economical design.

3. Compressive Reinforcing Steel in Cap or Footing. Compressive reinforcing steel tends to buckle when the cover is gone or when the concrete around the steel is weakened by compression. If the initial design indicates the need for compressive steel, the pier shall be redesigned to eliminate this need.

18.2.7 Single-Column Piers

The following applies to the design of single-column piers:

1. Cantilevers. The design of the cantilever is affected by the cantilever depth-versus-length geometry. Where the distance between the centerline of the bearing and the column is less than approximately twice the depth of the cantilever, the strut-and-tie model in LRFD Article 5.6.3 should be considered for the design of the cantilever.

2. Cantilever Reinforcement. All of the calculated cantilever reinforcement shall be extended throughout the entire length of cap. Cap stirrups shall be placed in the cap within the limits of the shaft at a minimum spacing of 12 in.

18.2.8 Pier Walls

Pier walls should be solid full height. The dimensions of the wall in the transverse direction may be reduced by providing cantilevers to form a hammerhead pier. Figure 18.2-C illustrates the typical detailing for pier wall tie bars.

18.2.9 Dynamic Load Allowance (IM)

Reference: LRFD Article 3.6.2.1

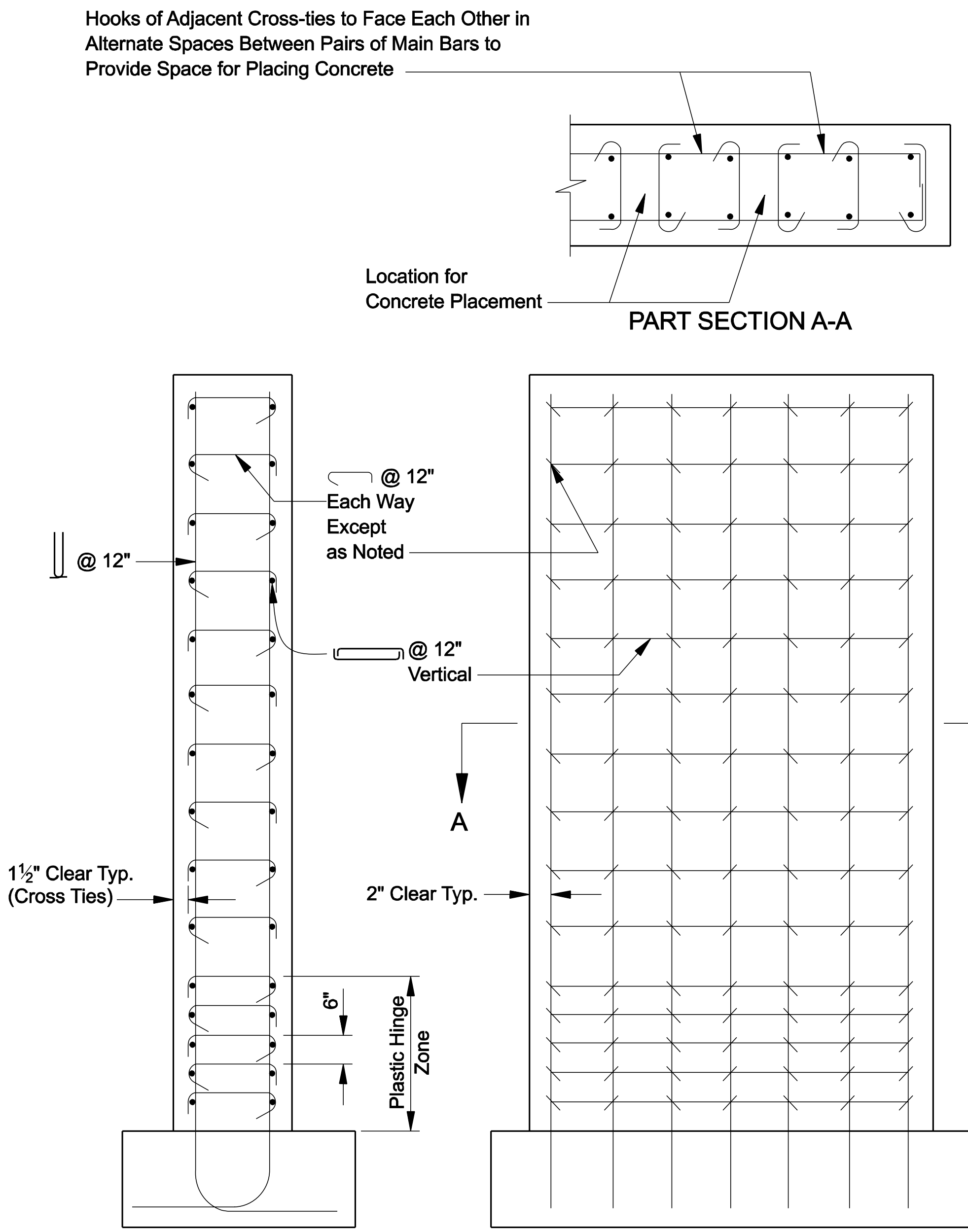
The *LRFD Specifications* allows the Dynamic Load Allowance (IM), traditionally called impact, to only be omitted on "foundation components that are entirely below ground level." NDOT requires that the dynamic load allowance be considered in the structural design of pier caps, pier columns and all piles, drilled shafts and footings, only if a significant portion of these elements is above ground.

18.2.10 Moment-Magnification

Reference: LRFD Article 5.7.4

Piers, pier columns and piles are referred to as compressive members, although their design is normally controlled by flexure. In most cases, the use of the moment-magnification approach in LRFD Article 5.7.4.3 is warranted. For exceptionally tall or slender columns/shafts where the slenderness ratio (Kl/r) is greater than 100, a refined analysis, as outlined in LRFD Article 5.7.4.1, should be performed. Where P-Delta design procedures are used, consideration shall be given in the design to the initial out-of-straightness of columns and the sustained dead load.

Moment magnification is not significant in seismic design and may be ignored.



PIER WALL TIE BARS

Figure 18.2-C

Chapter 19

EXPANSION JOINTS

NDOT STRUCTURES MANUAL

September 2008

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Chapter 19
EXPANSION JOINTS

LRFD Article 14.4 discusses bridge joint movements and loads, and LRFD Article 14.5 provides requirements for joints and considerations for specific joint types. This Chapter presents NDOT criteria for the design and selection of expansion joints in bridges.

19.1 GENERAL

19.1.1 Overview

Reference: LRFD Articles 14.4 and 14.5

The tributary expansion length equals the distance from the expansion joint to the point of assumed zero movement, which is the point along the bridge that is assumed to remain stationary when expansion or contraction of the bridge occurs. The location of the point of zero movement is a function of the longitudinal stiffness of the substructure elements.

Expansion joints in bridges are necessary to accommodate the expansion and contraction of bridges due to temperature variations. The following general criteria apply to all expansion joints in bridges:

1. General NDOT Practice. Bridges shall be designed to minimize the number of expansion joints because of their inherent operational and maintenance problems. Abutment seats tend to deteriorate due to leaky joints, to collect debris and to provide locations for animal and human habitation. The use of continuous structures minimizes the number of joints. As a consideration, when conditions permit, the designer may be able to eliminate the expansion joints and tie the approach slab into the superstructure, as suggested in Section 11.4.4. However, joints are always provided at the roadway end of approach slabs with an anchor or sleeper slab.

2. Consistency. Whenever possible, the bridge designer shall use the same type of joint and construction details used at the ends of bridges for locations of expansion at interior supports and in-span hinges.

3. Maintenance Problems. Many of the maintenance problems on bridges are the result of failed joints. Therefore, when joints are required, the selection, design and detailing of expansion joints are of critical importance. The potential for joint seal failure can be minimized by regular cleaning and timely maintenance.

4. Temperature Range. The bridge designer shall use Procedure A of LRFD Article 3.12.2.1 to determine the appropriate design thermal range. The minimum and maximum temperatures specified in Figure 19.1-A shall be taken as $T_{MinDesign}$ and $T_{MaxDesign}$, respectively, in LRFD Equation 3.12.2.3-1.

5. Recess Detail. Embedded steel elements, such as approach slab protection angles and strip seal expansion joint restrainers, shall be recessed ¼ in from finished grade. This recess provides protection from snow plow blades and accommodates milling of the concrete adjacent to the joints.

Region	Steel Bridges	Concrete Bridges
Outside of Clark County	-20°F to 105°F	0°F to 80°F
Clark County	20°F to 120°F	30°F to 100°F

LRFD PROCEDURE "A" TEMPERATURE CHANGES

Figure 19.1-A

6. Effects of Skew. The thermal movements of skewed bridges are such that asymmetrical movements ("racking") can occur along the length of the expansion joints. The movement is not solely in the longitudinal direction. The acute corners of a bridge with parallel skewed supports tend to expand and contract more than the obtuse corners, causing the joint to rack.

7. Other Geometric Considerations. Horizontally curved bridges and bridges with other special geometric elements, such as splayed girders, do not necessarily expand and contract in the longitudinal direction of the girders. Refined analysis of the entire bridge including superstructure and substructure elements may be necessary to characterize the thermal movement of complex bridges. The effect of thermal movements on the bearings of complex bridges could be more pronounced compared to bridges with simple geometrics. Refined analysis of horizontally curved, steel-girder bridges is recommended to estimate thermal effects because even slight curvature may develop large radial forces at bearings.

8. Blockouts. Provide blockouts in decks and approach slabs at expansion joints to allow for placement of the joint. The expansion joint assembly will be installed and the block-out concrete placed after profile grinding has been completed.

9. Cover Plates Over Expansion Joints. Cover plates shall be used over expansion joints at sidewalks. Where bicycles are anticipated in the roadway, the use of cover plates in the shoulder area shall be considered.

19.1.2 Estimation of General Design Thermal Movement, Δ_T

Reference: LRFD Article 3.12.2.3

The design thermal movement in inches shall be estimated by the following equation:

$$\Delta_T = \alpha L \left(T_{MaxDesign} - T_{MinDesign}\right) \quad \text{(LRFD Equation 3.12.2.3-1)}$$

where:

α = coefficient of thermal expansion, 6×10^{-6} for concrete bridges and 6.5×10^{-6} for steel girder bridges, in/in/°F

L = tributary expansion length, in

$T_{MaxDesign}$ = maximum design temperature from Figure 19.1-A.

$T_{MinDesign}$ = minimum design temperature from Figure 19.1-A.

19.1.3 Estimation of Design Movement

In addition to the thermal movement determined in Section 19.1.2, the effects of creep (CR) and shrinkage (SH) should be included in the total movement for prestressed concrete bridges.

For steel girder structures, creep and shrinkage effects are minimal and can be neglected in expansion joint design.

19.1.4 Setting Temperature

The designer shall determine gap widths at setting temperatures of 40°, 55°, 70°, 85° and 100°, consistent with the minimum and maximum temperatures at the bridge site. The gap widths should consider minimum gap widths and, for cast-in-place post-tensioned boxes, elastic shortening when appropriate. See the Design Examples in Section 19.3 for illustrations of typical calculations.

19.2 EXPANSION JOINT SELECTION AND DESIGN

19.2.1 General

Figure 19.2-A presents the typical application for several types of expansion joints used by NDOT. This Figure also provides the maximum joint movement and recommended usage.

The bridge designer determines the type of expansion joint and its required movement rating based on the expansion and racking demands, skew, gap widths and whether the joint is new or a retrofit. Gap width is the perpendicular distance between the faces of the joint at the road surface. Gap width does not directly apply to asphaltic plug joints. The minimum gap shall not be less than 1 in for steel bridges, as suggested in LRFD Article 14.5.3, but may be less for concrete bridges where creep and shrinkage must be considered. The maximum gap width should be 4½ in for strip seals and 3 in for individual components of modular joints.

Racking is the movement along the joint itself due to skew affects. It should be limited to 20% of the rated movement of the joint.

19.2.2 Strip Seal Joint

Reference: LRFD Article 14.5.6.7

A strip seal consists of a neoprene membrane (gland) rigidly attached to a steel restrainer on both sides of the joint. The material is premolded into a "V" shape that opens as the joint width increases and closes as the joint width decreases.

The strip seal expansion joint is NDOT's preferred deck expansion joint system for new bridges with estimated total design thermal movements ranging from 1 in to 5 in. The contractor will select a strip seal joint from the Qualified Products List (QPL) that provides the estimated total design thermal movement for each joint.

Strip seal joints are watertight when properly installed. Under the best conditions, the life of a strip seal tends to be longer than that of other joint seals. However, these seals are difficult to replace, and splices in the membrane should be avoided. They can be damaged by snowplows, especially if the skew is 20° or greater.

Joint Type	Total Joint Movement (in)	Typical Usage
Strip Seal	≤ 5	New and Retrofit
Modular Expansion	> 5	Where large movements are anticipated
Preformed Filler	≤ 2	Typically only used for Retrofits
Asphaltic Plug	≤ 1	Typically only Used for Retrofits
Pourable Seals	≤ 1	Typically used for Retrofits and Longitudinal Joints

EXPANSION JOINT SELECTION

Figure 19.2-A

Where practical and where additional protection for bearing assemblies and hinges is warranted, a secondary sealing system may be provided below the expansion joint assembly.

19.2.3 Modular Expansion Joint

Reference: LRFD Article 14.5.6.9

Modular joints are expensive and may require significant maintenance. Therefore, in the selection of modular joint systems, use only those that have been designed to facilitate the repair and replacement of components and that have been verified by long-term in-service performance. It is critical that the contract documents include a detailed description of the requirements for a modular joint system.

NDOT only uses modular expansion joints where large movements are anticipated. The following will apply to the design of modular-type expansion joints:

1. Expansion Movement. Modular joints should only be considered where the anticipated total expansion movement exceeds 5 in.

2. Joint Support. The blockouts and supports needed for modular joint systems are large and require special attention when detailing. For modular joints supported from the top of the girder, a detail of the supporting device shall be shown in the contract documents.

3. Splices. Where practical, modular joints should be full length with no field splices across the roadway width. If a field splice is required for staged construction on a slab-on-girder bridge, the support girders should be spaced at a maximum of 2 ft from the splice location, which should be outside of the wheel path. The splice will be designed according to the manufacturer's recommendations.

4. Neoprene Seal. The neoprene seal, which is a strip seal gland in a modular joint, will be one piece across the roadway width, regardless of construction staging considerations.

19.2.4 Preformed Joint Filler

Reference: LRFD Article 14.5.6.6

NDOT practice is to use preformed joint fillers where anticipated movements are small. The movement capacity of this type of joint is dictated by the joint width at the time of installation. Preformed joint fillers are relatively easy to maintain because local joint failures can be repaired. This system can be bonded to concrete or steel surfaces.

Preformed joint fillers are available in a variety of materials including elastomeric compression seals and expansion foam. The contractor will select a preformed joint filler from the QPL that provides the estimated total design thermal movement for the joint. If a specific type of joint filler is required for a design, then it should be clearly defined in the contract documents. Movements up to 2 in can be accommodated with this type of joint. Some preformed joint fillers do not perform well due to racking; therefore, preformed joint fillers should not be used where racking exceeds 15% of the specified movement rating of the joint.

19.2.5 Asphaltic Plug Joint

Reference: LRFD Article 14.5.6.5

NDOT typically only uses an asphaltic plug for retrofit applications for total movements of up to 1 in. This joint system is a smooth, durable, load-bearing surface that uses a combination of polymer-modified asphaltic binder and selected aggregate. Its advantages include the elimination of any mechanical anchorage system, ease of placement, low maintenance and rideability. Its disadvantages include its non-flexibility in cold temperatures and its tendency to rut under heavy traffic and turning movements in hot weather.

19.2.6 Pourable Seals

Reference: LRFD Article 14.5.6.5

Traditionally, pourable seals are used on shorter spans and longitudinal joints where the movement is ½ in or less.

Currently available systems typically include pourable silicone sealer and polyethylene foam backer rod as joint filler. The silicone is a self-leveling, rapid-curing, two-component polymer material. The backer rod is squeezed into the joint opening to prevent the sealant from spilling through the joint and to form the shape of the sealer. The silicone sealant is poured into the opening on top of the backer rod. It is important that the joint edges be clean and sound so that the silicone bonds tightly. The thickness of the silicone at the center should be no more than half the width of the joint. The bottom of the silicone must not bond to the material below. Pourable seals perform best if the seal is poured when the ambient temperature (which must be above 40°F) is at the middle of the historical range or the joint opening is at the midpoint.

There are certain advantages to this type of seal. Unlike many premolded seals, the performance of pourable seals is generally unaffected by joint walls that are not perfectly parallel or perfectly vertical. It is also relatively easy to repair. If a short portion of the seal fails, it is easy to remove the seal, clean the walls and quickly refill the joint. This activity minimizes traffic disruption and work zone hazards.

19.3 EXAMPLE PROBLEMS

The following presents two example problems for the design of expansion joints.

* * * * * * * * * *

<u>Example 19.3-1 — Cast-in-Place, Post-Tensioned Concrete Box-Girder Bridge</u>

Given: Cast-in-place, post-tensioned concrete box-girder bridge (not in Clark County)
L = expansion length = 240 ft
θ = skew angle = 0°

CR + SH = 1.00 in

Portion of total creep and shrinkage occurring after joint setting.

Problem: Determine expansion joint movement requirements.

Solution: Estimated design thermal movement:

$$\Delta_T = \alpha L \left(T_{MaxDesign} - T_{MinDesign}\right) \quad \text{(LRFD Equation 3.12.2.3-1)}$$

For a concrete superstructure:

$\alpha = 6.0 \times 10^{-6}$ in/in/°F
$T_{MaxDesign}$ = 80°F based upon the bridge location and Figure 19.1-A
$T_{MinDesign}$ = 0°F based upon the bridge location and Figure 19.1-A
Δ_T = $(6.0 \times 10^{-6}$ in/in/°F) (240 ft) (12 in/ft) (80° – 0°)
Δ_T = 1.4 in
Δ_{total} = Δ_T + CR + SH = 1.4 + 1.00 = 2.40 in (See Section 19.1.3)

A strip seal joint is acceptable because the estimated total design movement is within the range for strip seals.

Movements from setting temperature of 70°F:

1.4 in/(80°F – 0°F) = 0.0175 in/°F
Contraction (from 70°F to 0°F) = 1.23 in
Expansion (from 70°F to 80°F) = 0.18 in

Check the joint performance for both initial (without CR and SH) and final (with CR and SH) conditions:

<u>Initial Condition:</u>

Minimum joint opening @ 80°F = 1.50 in (least gap opening)
Joint opening at time of installation @ 70°F = 1.50 + 0.18 = 1.68 in

Final Condition:

Joint Opening @ 80°F = 1.50 + (CR + SH) = 1.50 + 1.00 = 2.50 in
Joint Opening @ 70°F = 2.50 + 0.18 = 2.68 in
Joint Opening @ 0°F = 2.68 + 1.23 = 3.91 in

Conclusion:

The gap width will vary from 1.5 in at 80° to 3.91 in at 0°. The strip seal must have a 4-in movement rating. The gap width at installation is 1.68 in, assuming 70°.

Using this Example, a table of installation gap widths can be developed to account for varying field temperatures during installation:

Setting Temperature	Gap Width
40°	2-3/16 in
55°	1-15/16 in
70°	1-11/16 in
80°	1½ in

Example 19.3-2 — Steel Girder Bridge with Concrete Deck

Given: Steel plate girders supporting a reinforced concrete bridge deck in Clark County
L = expansion length = 250 ft
θ = skew angle = 30°

Problem: Determine expansion joint movement requirements.

Solution: Estimated design thermal movement:

$$\Delta_T = \alpha L \left(T_{MaxDesign} - T_{MinDesign}\right) \quad \text{(LRFD Equation 3.12.2.3-1)}$$

For a steel superstructure:

$\alpha = 6.5 \times 10^{-6}$ in/in/°F
$T_{MaxDesign}$ = 120°F based upon the bridge location and Figure 19.1-A
$T_{MinDesign}$ = 20°F based upon the bridge location and Figure 19.1-A
Δ_T = $(6.5 \times 10^{-6}$ in/in/°F) (250 ft) (12 in/ft) (120° – 20°)
Δ_T = 2.0 in

A strip seal joint is acceptable because the estimated design thermal movement times the cosine of the skew angle (2.0 in (cos 30°) = 1.7 in) is within the range for strip seals.

Movements for setting temperature of 70°F:

2.0 in/(120°F – 20°F) = 0.02 in/°F
Contraction (from 70°F to 20°F) = 1.0 in
Expansion (from 70°F to 120°F) = 1.0 in

Joint openings (normal to the joint) @120°F = 1.5 in (assumed minimum gap):

@ 70°F = 1.5 in + (0.02 in/°F)(120°F – 70°F)(cos 30°) = 2.37 in

@ 20°F = 1.5 in (assumed minimum gap) + 2.0 in (estimated design thermal movement) x (cos 30°) = 3.23 in

Minimum nominal seal width to accommodate racking:

Using a 4-in strip seal, the amount of racking that can be accommodated:

Maximum allowed racking normal to seal = 4.0 x 0.20 = 0.80 in
Corresponding movement parallel to joint = 0.80/sin 30° = 1.6 in
Corresponding temperature range = 100° x (1.6/2.0) = 80°
Minimum installation temperature = 120° – 80° = 40°
Maximum installation temperature = 20° + 80° = 100°

Conclusion:

The gap width of the seal varies from 1.5 in to 3.2 in. A 4-in strip seal would be required for longitudinal movement. Racking requires limiting the installation temperature to between 40° and 100°. The gap width at 70° installation is 2.37 (2⅜) in.

Using this Example, a table of installation gap widths can be developed to account for varying field temperatures during installation:

Setting Temperature	Gap Width
40°	2⅞ in
55°	2⅝ in
70°	2⅜ in
85°	2⅛ in
100°	1⅞ in

Do not install joints outside of this temperature range.

Chapter 20

BEARINGS

NDOT STRUCTURES MANUAL

September 2008

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Chapter 20
BEARINGS

20.1 GENERAL

Reference: LRFD Articles 14.4 and 14.6

Bridge bearings accommodate the movements of the superstructure and transmit the loads to the substructure. The type of bearing selected depends upon the magnitude and type of movement and the magnitude of the load.

20.1.1 Movements

The consideration of movement is important for bearing design. Movements include both translations and rotations. The sources of movement include initial camber or curvature, construction loads, misalignment, construction tolerances, settlement of supports, thermal effects, elastic shortening due to post-tensioning, creep, shrinkage, and seismic and traffic loading.

20.1.2 Effect of Camber and Construction Procedures

The initial camber of bridge girders induces bearing rotation. Initial camber may cause a larger initial rotation on the bearing, but this rotation may decrease as the construction of the bridge progresses. Rotation due to camber and the initial construction tolerances are sometimes the largest component of the total bearing rotation. Evaluate both the initial rotation and its short duration. At intermediate stages of construction, the designer must add deflections and rotations due to the progressive weight of the bridge elements and construction equipment to the effects of live load and temperature. The direction of loads, movements and rotations must also be considered, because it is inappropriate to simply add the absolute maximum magnitudes of these design requirements. The designer should anticipate the worst possible (but yet realistic) condition. Do not consider combinations of absolute maximums that can not realistically occur. In special cases, it may be economical to install the bearing with an initial offset, or to adjust the position of the bearing after construction has started, to minimize the adverse effect of these temporary initial conditions.

20.1.3 Design Thermal Movements

Reference: LRFD Article 3.12.2

A change in temperature causes an elongation or shortening of a bridge component, leading to translation at its supports. The design thermal movement shall be estimated in accordance with Section 19.1.2.

Setting temperatures of 40°, 55°, 70°, 85° and 100°F, consistent with the minimum and maximum temperatures at the bridge site, shall be assumed for the installation of the bearings. At the time of construction, the appropriate setting conditions can be chosen based upon the ambient temperature.

Note that a given temperature change causes thermal movement in all directions. Because the thermal movement is a function of the expansion length as shown in LRFD Equation 3.12.2.3-1, a short, wide bridge may experience greater transverse movement than longitudinal movement.

20.1.4 Estimation of Total Design Movement

In addition to the thermal movement determined in Section 20.1.3, the effects of creep (CR) and shrinkage (SH) should be considered in the total movement for bridges in accordance with Section 19.1.3.

20.1.5 Serviceability, Maintenance and Protection Requirements

Reference: LRFD Article 2.5.2.3

Bearings under deck joints may be exposed to dirt, debris and moisture that promote corrosion and deterioration. As a result, these bearings should be designed and installed to minimize environmental damage and to allow easy access for inspection.

The service demands on bridge bearings are very severe and result in a service life that is typically shorter than that of other bridge elements. Therefore, allowances for bearing replacement must be part of the design process. Refer to Section 15.5.5 for specific requirements on jacking.

20.1.6 Seismic Requirements

Reference: LRFD Article 14.6.5

Bearing selection and design shall be consistent with the intended seismic response of the entire bridge system and related to the characteristics of both the superstructure and the substructure.

Bearings (other than seismic isolation bearings or structural fuse bearings) may be classified as rigid or deformable. Rigid bearings transmit seismic loads without any movement or deformations. Deformable bearings transmit seismic loads limited by plastic deformations or a restricted slippage of bearing components.

Where rigid bearings are used, the seismic forces from the superstructure shall be assumed to be transmitted through diaphragms or cross frames and their connections to the bearings and then to the substructure without reduction due to local inelastic action along this load path.

Steel-reinforced elastomeric bearing assemblies are typically designed to accommodate imposed seismic loads and displacements. Alternatively, if survival of the elastomeric bearing itself is not required, other means such as restrainers, shock transmission units or dampeners shall be provided to prevent unseating of the superstructure.

These provisions shall not apply to seismic isolation bearings or structural fuse bearings.

20.1.7 Anchor Bolts

Reference: LRFD Article 14.8.3

Anchor bolts shall be used to transfer horizontal forces through bearing assemblies when external devices such as shear keys are not present. In addition, anchor bolts are used as hold downs for bearings.

Holes for anchor bolts in steel elements of bearing assemblies shall be ¼ in larger in diameter than the diameter of the anchor bolt. The centerlines of anchor bolts shall be a minimum of 2 in from the edge of the girder. A larger offset may be necessary to facilitate installation. The designer must consider the space necessary for nuts, washers, base plate welds and construction tolerances and establish anchor bolt locations accordingly. Maintain ½-in clearance from the edge of the elastomeric bearing to the edge of the anchor bolt.

Anchor bolts shall be designed to behave with ductility. The anchor bolts shall be designed for the combined effect of bending and shear for seismic loads as specified in LRFD Article 14.6.5.3.

Sufficient reinforcement shall be provided around the anchor bolts to develop the horizontal forces and anchor them into the mass of the substructure unit. Potential concrete crack surfaces next to the bearing anchorage shall be identified and their shear friction capacity evaluated. Conflicts between anchor bolt assemblies and substructure reinforcement is common, especially for skewed bridges. Therefore, the bridge designer must ensure that all reinforcing steel can fit around the bearing assemblies.

20.1.8 Bearing Plate Details

The bearing plate shall be at least 1 in wider than the elastomeric bearing on which the plate rests. Use a minimum bearing plate thickness of 1½ in. When the instantaneous slope of the grade plus the final in-place camber exceeds 1%, bevel the bearing plate to match the grade plus final camber. For beveled bearing plates, maintain a minimum of 1½ in thickness at the edge of the bearing plate.

At expansion bearings, the designer shall provide slotted bearing plates. Determine the minimum slot size according to the amount of movement and end rotation calculated. The slot length, L, should be:

$$L = \text{(diameter of anchor bolt)} + 1.2\,\text{(total movement)} + 1.0\text{ in}$$

The multiplier of 1.2 represents the load factor from LRFD Table 3.4.1-1 for TU, CR and SH. The total movement should include an effect of girder end rotation at the level of the bearing plate. The slot length should be rounded to the next higher ¼ in. To account for the possibility of different setting temperatures at each stage, provide offset dimensions in the contract documents for stage-constructed projects. For all other projects, the designer must also consider the need to provide offset dimensions.

20.1.9 Leveling Pad at Integral Abutments

A plain elastomeric pad shall be detailed under the bearing plate of girders at integral abutments to provide a level and uniform bearing surface. Structural grout is not an acceptable substitute.

20.2 BEARING SELECTION

20.2.1 General

Where possible, steel-reinforced elastomeric bearings should be used for all girder bridges. Bridges with large bearing loads and/or multi-directional movement may require other bearing devices such as pot, spherical or disc bearings.

Bearing selection is influenced by many factors including loads, geometry, maintenance, available clearance, displacement, rotation, deflection, availability, policy, designer preference, construction tolerances and cost. In general, vertical displacements are restrained, rotations are allowed to occur as freely as possible, and horizontal displacements may be either accommodated or restrained. Distribute the loads among the bearings in accordance with the superstructure analysis.

See Figure 20.2-A for a general summary of bearing capabilities. The values shown in the table are for preliminary guidance only. The final step in the selection process consists of completing a design of the bearing in accordance with the *LRFD Specifications*. The resulting design will provide the geometry and other pertinent specifications for the bearing. If the load falls outside of the optimal ranges, the bridge designer should contact the bearing manufacturer.

Typically, concrete shear keys are used with elastomeric bearings to transfer horizontal forces from a concrete box girder superstructure to the substructure. Bearing plates and anchor bolts are used for steel and precast concrete girder superstructures.

The following Sections summarize typical NDOT practices for the selection of a bearing type.

Type		Load (kips)	Translation (in)		Rotation Limit (Rad)	Cost	
		Optimal Design Range[1]	Min	Max		Initial	Maintenance
Steel-Reinforced Elastomeric Bearing		50 to 650	0	4	0.04	low	Low
High-Load, Multi-Rotational (HLMR) Bearings	Pot Bearing	270 to 2250	0[2]	0[2]	0.04 - 0.05	high	High
	Disc Bearing	270 to 2250	0[2]	0[2]	0.03	high	High
	Spherical Bearing	270 to 2250	0[2]	0[2]	> 0.05	high	high
Plain Elastomeric Pad		0 to 150	0	¾	0.0175	low	low

[1] *Higher and lower values may be applicable if necessary.*

[2] *High-Load, Multi-Rotational (HLMR) bearings, such as pot bearings, disc bearings and spherical bearings have no inherent translational capability. Expansion bearings are achieved by using them in conjunction with flat PTFE sliding surfaces.*

SUMMARY OF BEARING CAPABILITIES

Figure 20.2-A

20.2.2 Steel-Reinforced Elastomeric Bearings

Reference: LRFD Article 14.7.6

Where possible, steel-reinforced elastomeric bearings should be used for all girder bridges. They are usually a low-cost option and require minimal maintenance. Figure 20.2-B illustrates a steel-reinforced elastomeric bearing assembly used with a steel girder. Section 20.3 discusses these bearings in more detail. Section 20.4 provides a design example for a steel-reinforced elastomeric bearing for a steel girder bridge.

Elastomeric expansion bearings shall be provided with adequate seismic-resistant anchorages to resist the horizontal forces in excess of those accommodated by shear in the pad. The sole plate and the base plate shall be made wider to accommodate the anchor bolts.

Elastomeric fixed bearings shall be provided with a horizontal restraint adequate for the full horizontal load.

20.2.3 Plain Elastomeric Bearing Pads

Reference: LRFD Article 14.7.6

Plain elastomeric bearing pads are usually a low-cost option, and they require minimal maintenance. However, their use is restricted to lighter bearing loads for practical reasons. They are used as leveling pads at integral abutments for girder bridges.

20.2.4 High-Load, Multi-Rotational (HLMR) Bearings

20.2.4.1 General

These bearing types are generally avoided due to their cost. They may be appropriate at bridges with large vertical loads; i.e., in excess of 650 kips.

High-load, multi-rotational (HLMR) bearings are used in applications where loads exceed the capabilities of steel-reinforced elastomeric bearings. The choice among HLMR bearings is based upon the rotational capabilities presented in Figure 20.2-A.

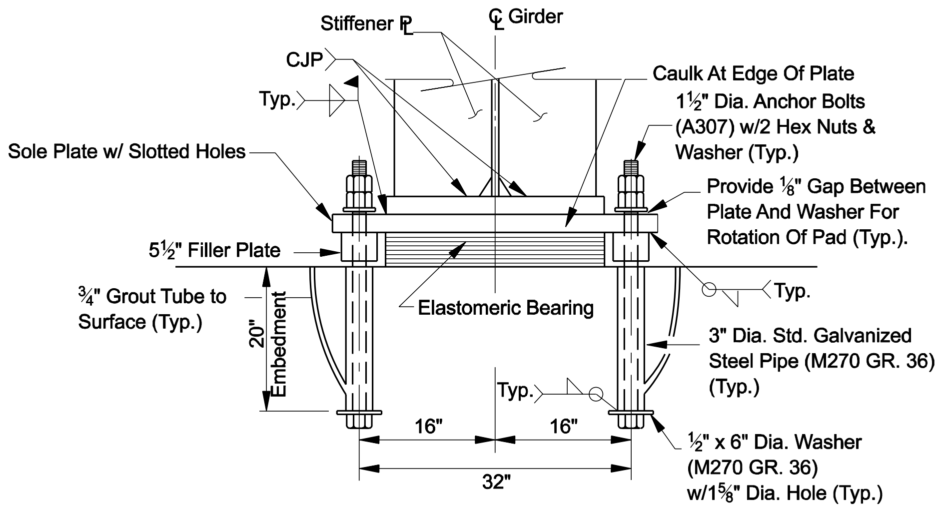
The contract documents for bridges with HLMR bearings should not include specific details for the bearings. Only schematic bearing details, combined with specified loads, movements and rotations, need to be shown. The bearing is designed by the manufacturer, which advantageously uses the cost-effective fabrication procedures that are available in the shop.

Figure 20.2-C illustrates the schematics for HLMR bearings.

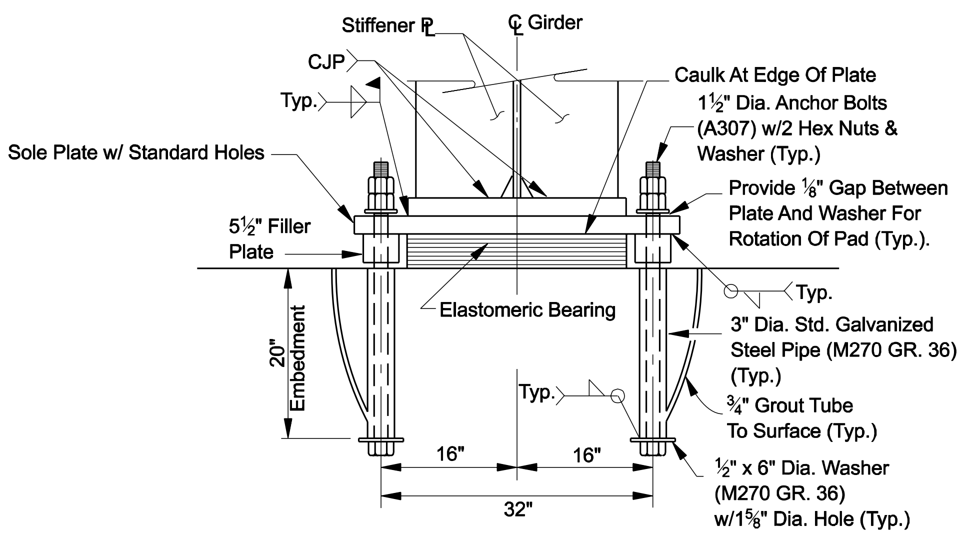
20.2.4.2 Pot Bearings

Reference: LRFD 14.7.4

Pot bearings consist of a pot/piston assembly within which an elastomeric disc is encapsulated and fitted with an anti-extrusion sealing device. Under load, this encapsulated elastomeric disc



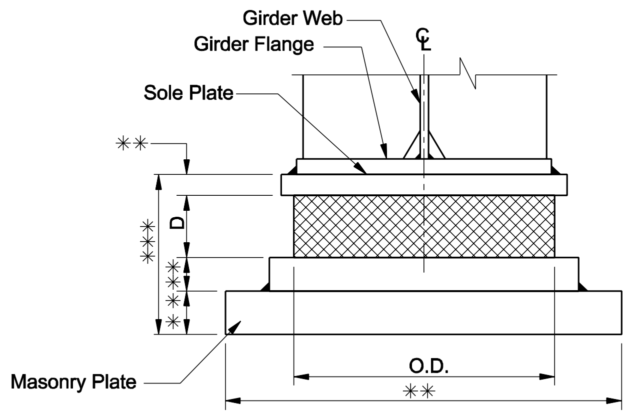
TYPICAL EXPANSION BEARING ASSEMBLY



TYPICAL FIXED BEARING ASSEMBLY

ELASTOMERIC BEARING ASSEMBLY

Figure 20.2-B



Not to Scale

✳✳ To be Determined by Designer
Note: Anchor Bolts are not Shown.

✳✳✳ The Overall Height Should be Based on the Largest "D" Values From All Potential Bearing Manufacturers.

Indicates Fixed, Guided or Free Portion of the Bearing Designed by The Manufacturer

HIGH-LOAD, MULTI-ROTATIONAL BEARING

Figure 20.2-C

acts in a similar manner to an uncompressible confined fluid, enabling the pot and piston to rotate relative to each other. Pot bearings enable rotation in any direction. The pot and piston feature fittings for securing the bearing to the bridge structure.

Fixed pot bearings are constrained horizontally. Identical in construction to fixed bearings, free-sliding pot bearings are fitted with a PTFE sliding surface in contact with a steel plate, enabling the bearing to slide in all directions. Guided sliding pot bearings are also identical in construction to free-sliding bearings but are also fitted with one or more guides to limit the bearing movement to only one direction.

Pot bearings are able to support large compressive loads, but their elastomer can leak and their sealing rings can suffer wear or damage.

20.2.4.3 Spherical Bearings

Reference: LRFD Article 14.7.3

Spherical bearings, termed “Bearings with Curved Sliding Surfaces,” include bearings with both spherical and cylindrical sliding surfaces. Spherical bearings are able to sustain large rotations but require proper clearances and very smooth and accurate machining.

A spherical bearing relies upon the low-friction characteristics of a curved PTFE-stainless steel interface to provide a high level of rotational flexibility in multiple directions. An additional flat PTFE-stainless steel surface can be incorporated into the bearing to provide either guided or non-guided translational movement capability. Woven PTFE is generally used on the curved surfaces of spherical bearings. Woven PTFE exhibits enhanced creep (cold flow) resistance and durability characteristics relative to unwoven PTFE. When spherical bearings are detailed to accommodate translational movement, woven PTFE is generally also specified on the flat sliding surface.

Most spherical bearings are fabricated with the concave surface oriented downward to minimize dirt infiltration between PTFE and the stainless steel surface. Refined modeling of the overall structure must recognize that the center of rotation of the bearing is not coincident with the neutral axis of the girder above.

20.2.4.4 Disc Bearings

Reference: LRFD Article 14.7.8

A disc bearing is composed of an annular shaped urethane disc designed to provide moderate levels of rotational flexibility. A steel shear-resisting pin in the center provides resistance against lateral force. A flat PTFE-stainless steel surface can be incorporated into the bearing to also provide translational movement capability, either guided or non-guided.

Disc bearings are susceptible to uplift during rotation, which may limit their use in bearings with polytetrafluoroethyl (PTFE) sliding surfaces.

20.2.4.5 Polytetrafluoroethyl (PTFE) Sliding Surfaces

Reference: LRFD Article 14.7.2

For expansion high-load, multi-rotational bearings and where the maximum movements of elastomeric bearings are exceeded, the designer may consider using PTFE sliding surfaces with the bearing to obtain translational capability. PTFE sliding surfaces can also be used in conjunction with an elastomeric bearing to obtain translational capability.

The following design information applies to PTFE sliding surfaces:

- Optimal design range for loads: 0 kips to 2250 kips
- Translation: 1 in to > 4 in

20.2.5 Seismic Isolation Bearings

There are various types of seismic isolation bearings, most of which are proprietary. The use of seismic isolation bearings is discussed in the AASHTO *Guide Specifications for Seismic Isolation Design* and the FHWA *Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges*.

These bearings can assist in achieving seismic objectives in retrofit situations; see Section 22.9.4.6. The Chief Structures Engineer must approve the use of seismic isolation bearings.

20.3 PLAIN ELASTOMERIC BEARING PADS AND STEEL-REINFORCED ELASTOMERIC BEARINGS

Reference: LRFD Articles 14.7.5 and 14.7.6

20.3.1 General

Plain elastomeric bearing pads and steel-reinforced elastomeric bearings have fundamentally different behaviors and, therefore, they are discussed separately. It is usually desirable to orient elastomeric pads and bearings so that the long side is parallel to the principal axis of rotation, because this orientation better accommodates rotation.

20.3.2 Shape Factor

Elastomers are used in both plain elastomeric bearing pads and steel-reinforced elastomeric bearings. The behavior of both pads and bearings is influenced by the shape factor (S) where:

$$S = \frac{\text{Plan Area}}{\text{Area of Perimeter Free to Bulge}}$$

20.3.3 Holes in Elastomer

NDOT prohibits the use of holes in steel-reinforced elastomeric bearings or plain elastomeric bearing pads.

20.3.4 Edge Distance

For elastomeric pads and bearings resting directly on a concrete bridge seat, the minimum edge distance shall be 3 in.

20.3.5 Elastomer

Reference: LRFD Articles 14.7.5.2 and 14.7.6.2

NDOT only uses neoprene for its elastomeric bearing pads and steel-reinforced elastomeric bearings.

All elastomers are visco-elastic, nonlinear materials and, therefore, their properties vary with strain level, rate of loading and temperature. Bearing manufacturers evaluate the materials on the basis of Shore A Durometer hardness, but this parameter is not a good indicator of the shear modulus, G. Use a Shore A Durometer hardness of 50 or 55. This leads to shear modulus values in the range of 0.095 to 0.200 (use the least favorable value for design) ksi @73°F. The shear stiffness of the bearing is its most important property because it affects the forces transmitted between the superstructure and substructure.

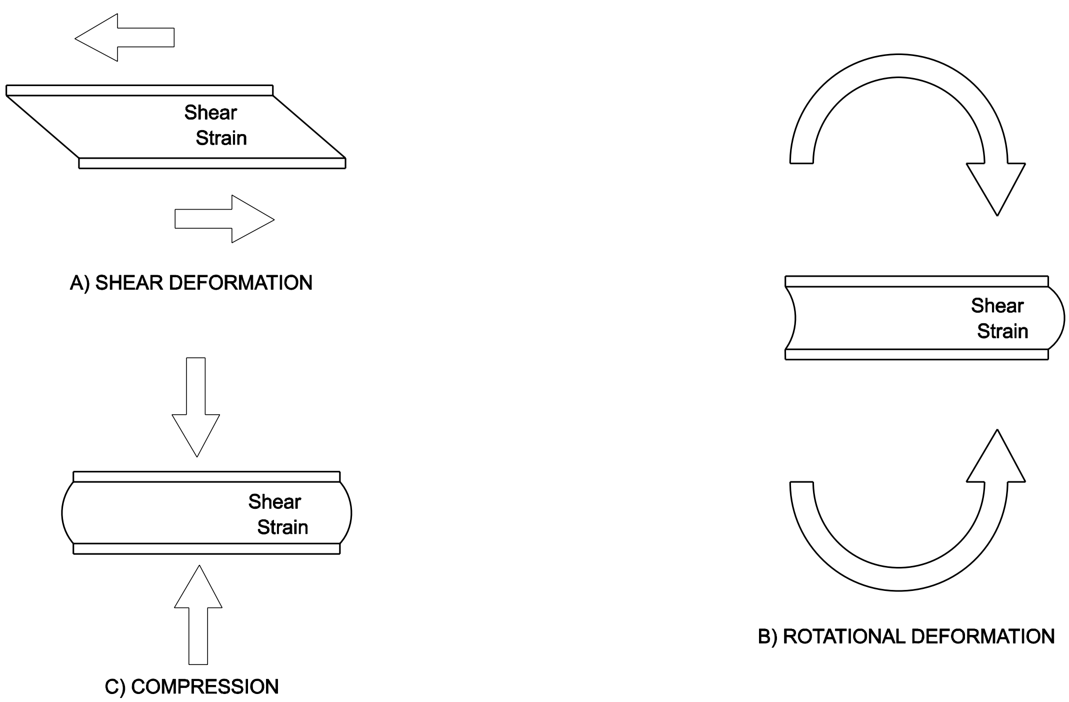
Elastomers are flexible under shear and uniaxial deformation, but they are very stiff against volume changes. This feature makes possible the design of a bearing that is flexible in shear but stiff in compression.

Elastomers stiffen at low temperatures. The low-temperature stiffening effect is very sensitive to the elastomer compound, and the increase in shear resistance can be controlled by the selection of an elastomer compound that is appropriate for the climatic conditions. The minimum low-temperature elastomer shall be Grade 3. The designer shall indicate the elastomer grade in the contract documents.

20.3.6 Steel-Reinforced Elastomeric Bearings

The use of steel-reinforced elastomeric bearings in combination with steel bearing plates is preferred for slab-on-girder bridges. Use a 1-in minimum clearance between the edge of the elastomeric bearing and the edge of the bearing plate in the direction parallel to the beam or girder. Use a ½-in minimum clearance between the edge of the elastomeric bearing pad and the anchor bolt in the direction perpendicular to the girder.

Steel-reinforced elastomeric bearings behave differently than plain elastomeric bearing pads. Steel-reinforced elastomeric bearings have uniformly spaced layers of steel and elastomer. The bearing accommodates translation and rotation by deformation of the elastomer. The elastomer is flexible under shear stress but stiff against volumetric changes. Under uniaxial compression without steel reinforcement, the flexible elastomer would shorten significantly and sustain large increases in its plan dimension but, with the stiff steel layers, lateral expansion is restrained. This restraint induces a bulging pattern as shown in Figure 20.3-A and provides a large increase in stiffness under compressive load. This permits a steel-reinforced elastomeric bearing to support relatively large compressive loads while accommodating large translations and rotations.



STRAINS IN A STEEL-REINFORCED ELASTOMERIC BEARING

Figure 20.3-A

The design of steel-reinforced elastomeric bearings requires an appropriate balance of compressive, shear and rotational stiffnesses. The shape factor affects the bearing's ability to compress and rotate, but it has no impact on the bearing's ability to translate horizontally.

A bearing must be designed to control the stress in the steel reinforcement and the strain in the elastomer. This is accomplished by controlling the elastomer layer thickness and the shape factor of the bearing. The design must satisfy the fatigue, stability, delamination, yield and rupture of the steel reinforcement; the stiffness of the elastomer; and the geometric constraints.

Large rotations and translations require thicker bearings. Translations and rotations may occur about the longitudinal or transverse axis of a steel-reinforced elastomeric bearing.

Steel-reinforced elastomeric bearings become large if they are designed for loads greater than approximately 650 kips. The maximum practical load capacity of a steel-reinforced elastomeric bearing pad is approximately 750 kips. Uniform heating and curing during vulcanization of such a large mass of elastomer becomes difficult, because elastomers are poor heat conductors. Manufacturing constraints thus impose a practical upper limit on the size of most steel-reinforced elastomeric bearings. If the design loads exceed 650 kips, the designer should check with the manufacturer for availability.

20.3.7 <u>Plain Elastomeric Bearing Pads</u>

Plain elastomeric bearing pads can support modest gravity loads, but they can only accommodate limited rotation or translation. Hence, they are best suited for bridges with small expansion lengths or specialty situations.

Plain elastomeric bearing pads rely on friction at their top and bottom surfaces to restrain bulging due to the Poisson effect. Friction is unreliable, and local slip results in a larger elastomer strain than that which occurs in steel-reinforced elastomeric pads and bearings. The increased elastomer strain limits its load capacity, and the pad must be relatively thin if it will carry the maximum allowable compressive load. A maximum friction coefficient of 0.20 should be used for the design of elastomeric pads that are in contact with clean concrete or steel surfaces. If the shear force is greater than 0.20 of the simultaneously occurring compressive force, then the bearing should be secured against horizontal movement. If the designer is checking the maximum seismic forces that can be transferred to the substructure through the pad, then a friction coefficient of 0.40 should be used.

Plain elastomeric bearing pads shall be designed and detailed in accordance with Method A of LRFD Article 14.7.6.

20.3.8 <u>Design of Steel-Reinforced Elastomeric Bearings</u>

Reference: LRFD Articles 14.7.5 and 14.7.6

The Method A procedure in LFRD Article 14.7.6 shall be used for steel-reinforced elastomeric bearings. The Method B procedure in LRFD Article 14.7.5 may be used for high-capacity bearings, but only with the approval of the Chief Structures Engineer. High-capacity elastomeric bearings should be used only where very tight geometric constraints, extremely high loads, or special conditions or circumstances require the use of higher grade material.

The Method B design procedure allows significantly higher average compressive stresses. These higher allowable stress levels are justified by an additional acceptance test, specifically a long-duration compression test. Designers must prepare a unique Special Provision for inclusion in the contract documents if a high-capacity elastomeric bearing is used.

Design criteria for both methods are based upon satisfying fatigue, stability, delamination, steel reinforcement yield/rupture and elastomer stiffness requirements.

The minimum elastomeric bearing length or width shall be 6 in. Generally, all pads shall be 50 or 55 durometer hardness. A minimum of ⅛ in of cover shall be provided at the edges of the steel shims. The top and bottom cover layers shall be no more than 70% of the thickness of the interior layers.

In determining bearing pad thickness, it should be assumed that slippage will not occur. The total elastomer thickness shall be no less than twice the maximum longitudinal or transverse deflection. The designer shall check the bearing against horizontal walking in accordance with LRFD Article 14.7.6.4.

A setting temperature of 70°F shall be used for the installation of the bearings unless the time of construction is known. In this case, the setting temperature may be modified accordingly. NDOT practice is to use 80% of the total movement range for design. This value assumes that the bearing is installed within 30% of the average of the maximum and minimum design temperatures. LRFD Article C14.7.5.3.4 recommends using 65% of the total movement range for design but, due to the wide variation in temperatures across the State and variations within a single day, the design value is increased. The formulas for determining the total elastomer thickness are as follows:

1. For precast, prestressed concrete girder spans, an allowance must be made for half of the shrinkage. Creep is not typically considered when determining the total elastomer thickness for precast concrete girders. The design thermal movement (Δ_T) shall be based upon $T_{MaxDesign}$ and $T_{MinDesign}$ from Figure 19.1-A. Therefore, the minimum total elastomer thickness for precast girders = 2 (Δ_T + ½ ΔSH).

2. For steel girder spans, the design thermal movement (Δ_T) shall be based upon $T_{MaxDesign}$ and $T_{MinDesign}$ from Figure 19.1-A. No allowance is needed for shrinkage. Therefore, the minimum total elastomer thickness for steel girders = 2 (Δ_T).

3. For cast-in-place and post-tensioned concrete spans, the full shrinkage, elastic shortening and creep shall be considered in addition to the thermal movement. Therefore, the minimum total elastomer thickness = 2 (Δ_T + ΔSH + ΔCR + ΔEL).

The bearing details must be consistent with the design assumptions used in the seismic analysis of the bridge.

20.4 DESIGN EXAMPLE

The following presents a design example for a steel-reinforced elastomeric expansion bearing for a single-span, five-girder steel bridge in Clark County. The example proportions a steel-reinforced elastomeric bearing by selecting the number and thickness of alternating elastomeric and steel layers. Further, the plan dimensions of the bearing are checked.

20.4.1 Given Data

The steel-reinforced elastomeric bearing design example is for an expansion bearing at the abutment of a single-span steel girder:

- Service I Limit State:

  DL = 78.4 k
  LL = 92 k
  P_{sd} = 68 k (factored permanent load at the Strength Limit State considering minimum load factors)
  WS = 31 k
  WL = 6 k
  θ_{sx} = 0.0121 rad (total rotation about transverse axis including 0.005 rad for uncertainty)

- Bridge located in Clark County, $T_{MaxDesign}$ = 120°F and T_{min} = 20°F
- Type II soil profile, S = 1.2
- Acceleration coefficient, A = 0.15
- Shore A durometer hardness = 50
- f_y = 36 ksi (for steel reinforcement)
- Length of Bridge = 120 ft

20.4.2 Trial Bearing Geometry

Exterior or cover thickness ≤ 70% of internal layer thickness (LRFD Article 14.7.6.1)

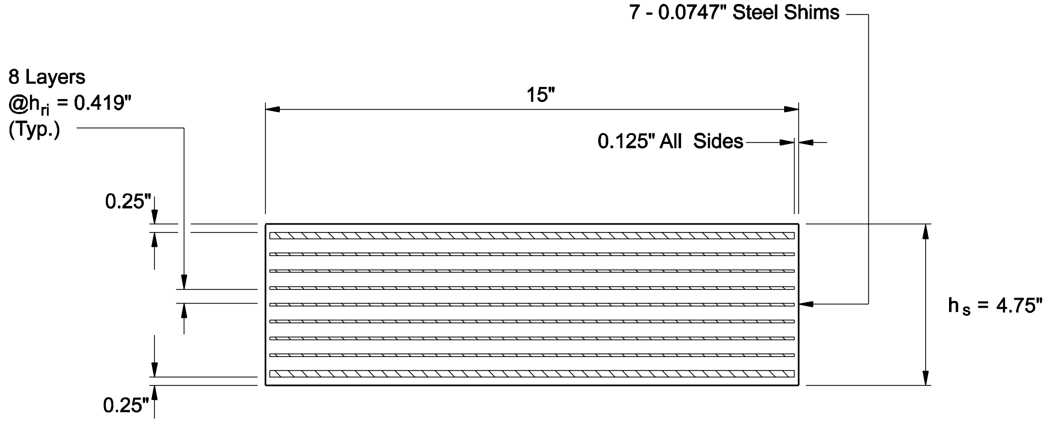
- Total bearing height: h_s = 4.75 in
- Length (longitudinal direction): L = 15 in
- Width (transverse direction): W = 15 in
- Elastomer exterior thickness: h_{re} = 0.25 in
- Thickness of interior steel reinforcement: t_s = 0.0747 in (14 gage)
- Number of shims: n_{shims} = 7
- Thickness of outermost steel reinforcement: t_{p1} = 0.1875 in

- Thickness of elastomer internal layer = h_{ri} =

$$\frac{\left[4.75\text{ in} - 2\left(0.1875\text{ in}\right) - 7\left(0.0747\text{ in}\right) - 2\left(0.25\right)\right]}{8} = 0.419\text{ in}$$

- Total thickness of elastomer = $h_{rt} = 8(0.419) + 2(0.25) = 3.85$ in

See Figure 20.4-A.



ELASTOMER LAYERS

Figure 20.4-A

20.4.3 <u>Solution</u>

Use Method A for Bearing Design.

Reference: LRFD Article 14.7.6

- Compute Shape Factor

  For rectangular bearing without holes:

$$S = \frac{\text{Plan Area}}{\text{Area of Perimeter Free to Bulge}}$$

$$= \frac{(15\text{ in})(15\text{ in})}{2(h_{ri})(15+15)} = \frac{(15)(15)}{2(0.419)(30)}$$

$$= 8.95$$

- Compressive Stress

  Allowable stress:

$\sigma_s \leq 1.0$ ksi (LRFD Equation 14.7.6.3.2-7)
and
$\sigma_s \leq 1.0$ GS: (LRFD Equation 14.7.6.3.2-6)

$G_{min} = 0.095$ ksi (LRFD Table 14.7.6.2-1)

$1.0\ G_{min}S = 1.0\ (0.095)\ (8.95) = 0.85$ ksi

Therefore, $\sigma_s \leq 0.85$ ksi

Average compressive stress:

$$\sigma_s = \frac{DL + LL}{(15\text{ in})(15\text{ in})}$$

$$= \frac{78.4 + 92\text{ k}}{225\text{ in}^2}$$

$$= 0.76\text{ ksi} \leq 0.85\text{ ksi}$$ **OK**

- <u>Compressive Deflection</u> (LRFD Articles 14.7.5.3.3 and 14.7.6.3.3)

Instantaneous deflection:

$$\delta_i = \Sigma\varepsilon_i h_{ri}$$

For $\sigma_s = 0.76$ and S = 8.95

From LRFD Figure C14.7.6.3.3-1: $\varepsilon_i = 0.035$

<u>Dead Load</u>:

For $\sigma d = 0.35$ and S = 8.95

From LRFD Figure C14.7.6.3.3-1: $\varepsilon_i = 0.015$

δd = 0.015 [2(0.25) + 8(0.419)]
= 0.06 in

<u>Live Load</u>:

For $\sigma l = 0.41$ and S = 8.95

From LRFD Figure C14.7.6.3.3-1: $\varepsilon_i = 0.021$

δl = 0.021[2(0.25) + 8(0.419)]
= 0.08 in < 0.125 in **OK** (LRFD Article C14.7.5.3.3)

<u>Creep</u>:

δ_{creep} = acrδd

acr = 0.25 (LRFD Table 14.7.6.2-1)

δ_{creep} = (0.25)(0.06)
= 0.015 in

Total long-term dead load deflection:

$\delta lt \;=\; \delta d + acr\delta d$
$\;=\; 0.06 + 0.015$
$\;=\; 0.075$ in

Initial compressive deflection in any layer at Service Limit State without Dynamic Load Allowance $\leq$ 0.07 h_{ri}:

$$\delta_{i_{one\ layer}} = \varepsilon_i h_{ri}$$
$$= 0.035\,(0.419)$$
$$= 0.0147 \text{ in}$$

$$0.07\left(h_{ri}\right) = 0.07\left(0.419\right) = 0.029$$

Therefore, $\delta_{i_{one\ layer}} = 0.0147 \text{ in} \leq 0.029 \text{ in}$ **OK**

- Shear Deformation

$h_{rt} \geq 2\Delta_s$ (LRFD Equation 14.7.6.3.4-1)

$h_{rt} = 3.85$ in

$\Delta_s = \Delta_O = 0.80\,(\Delta_T) + \Delta SH + \Delta CR + \Delta EL$

$\Delta SH + \Delta CR + \Delta EL = 0$ (Steel bridge)

Δ_S is taken as Δ_O, modified to account for the substructure stiffness and construction procedures. Assuming that the abutment does not accommodate any bridge movement and that good construction procedures are followed:

$\Delta_S = \Delta_O$

Procedure A for design movements of elastomeric bearings: (LRFD Article 3.12.2.1)

$\Delta_T = \alpha L\,(T_{Max\ Design} - T_{Min\ Design})$: (LRFD Equation 3.12.2.3-1)

$\alpha = 6.5 \times 10^{-6}$ in/in/°F (LRFD Article 6.4.1)

$L = 120 \text{ ft} = 1440 \text{ in}$

$T_{Max\ Design} = 120°F$ (Figure 19.1-A)
$T_{Min\ Design} = 20°F$

$\Delta_T = (6.5 \times 10^{-6})(1440)(120 - 20)$
$= 0.93$ in

Check for $\Delta_S = 0.80\,\Delta_T$

$h_{rt} \geq 2\Delta_S = 2(0.80)(0.93) = 1.49$ in

h_{rt} = 3.85 in > 1.49 = $2\Delta_S$ **OK**

- <u>Rotation</u>

$$\sigma_s \geq 0.5\,GS\left(\frac{L}{h_{ri}}\right)^2 \frac{\theta_{sx}}{n}$$ (LRFD Equation 14.7.6.3.5d-1)

θ_{sx} = 0.0121 rad

$$n = 8 + \frac{1}{2} + \frac{1}{2} = 9$$

$$0.5\,GS\left(\frac{L}{h_{ri}}\right)^2 \frac{\theta_{sx}}{n} = (0.5)(0.095)(8.95)\left(\frac{15}{0.419}\right)^2 \left(\frac{0.0121}{9}\right) = 0.73\text{ ksi}$$

σ_s = 0.76 > 0.73 ksi **OK**

- <u>Stability</u>

$$h_s \leq \text{the lesser of } \frac{L}{3} \text{ or } \frac{W}{3}$$ (LRFD Article 14.7.6.3.6)

$$\frac{L}{3} = \frac{15}{3} = 5\text{ in}$$

$$\frac{W}{3} = \frac{15}{3} = 5\text{ in}$$

h_s = 4.75 in < 5 in = $\frac{L}{3}$ = $\frac{W}{3}$ **OK**

- <u>Reinforcement</u> (LRFD Articles 14.7.5.3.7 and 14.7.6.3.7)

At the Service Limit State:

$$h_s \geq \frac{3\,h_{max}\,\sigma_s}{f_y}$$ (LRFD Equation 14.7.5.3.7-1)

$$\geq \frac{3(0.419)(0.76\text{ ksi})}{36\text{ ksi}}$$

h_s ≥ 0.0265 in

h_s = 0.0747 in ≥ 0.0265 in **OK**

At the Fatigue Limit State:

$$h_s \ \geq \ \frac{2\ h_{max}\ \sigma_L}{\Delta F_{TH}} \qquad \text{(LRFD Equation 14.7.5.3.7-2)}$$

$$\geq \ \frac{2(0.419)(0.41\,\text{ksi})}{24.0\ \text{ksi}}$$

Note: ΔF_{TH} = 24.0 ksi (Fatigue Category A)

$$h_s \ \geq \ 0.0143\ \text{in}$$

$$h_s \ = \ 0.0747\ \text{in} \ \geq \ 0.0143\ \text{in}$$ **OK**

- <u>Anchorage for Wind</u>

Reference: LRFD Articles 3.8 and 14.8.3.1

Transverse horizontal movement check:

P_{sd} = minimum vertical force due to permanent loads
= 68 kips

WS = 31 k
WL = 6 k

Strength III (per bearing):

$$V_{wind} \ = \ \frac{1.4\ WS}{5} \ = \ 8.7\ k$$

Strength V (per bearing):

$$V_{wind} \ = \ \frac{0.4\ WS \ + \ 1.0\ WL}{5} \ = \ 3.7\,k$$

Assuming a coefficient of friction of 0.20 (LRFD Article C14.8.3.1):

$$\left(V_{wind}\right)_{max} \ = \ 8.7 \ < \ 13.6 \ = \ 0.20\ P_{sd}$$

Therefore, no anchorage is necessary for wind considerations.

- <u>Anchorage for Seismic</u>

Reference: LRFD Article 3.10.9.1

Minimum design connection force = S x A x DL = 1.2 (0.15) (78.4) = 14.1 kips

Because 14.1 kips > 0.2 P_{sd} = 13.6 kips, anchorage for seismic forces is required.

Provide anchorage through anchor bolts (assume ¾-in diameter ASTM F1554 anchor rods):

$$R_n = 0.38 A_b F_{ub} N_s = 0.38 \times 0.44 \times 75 \times 2 = 25\ \text{kips} \qquad \text{(LRFD Equation 6.13.2.7-2)}$$

Assume φ = 0.80 (*LRFD Specifications* does not specify a φ for ASTM F1554)

φR_n = 0.8 x 25 = 20 kips

Because 20 kips > 14.1 kips, only one bolt is required; therefore, use four (one at each corner of the bearing)

- Summary

Therefore, the trial bearing geometry shown in Figure 20.4-A is acceptable for all design requirements.

Chapter 21

RAILROADS

NDOT STRUCTURES MANUAL

September 2008

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Chapter 21
RAILROADS

21.1 HIGHWAY BRIDGES OVER RAILROADS

21.1.1 Design Policies and Practices

Highway bridges constructed over railroads must be designed to be consistent with the requirements from a variety of sources. The following summarizes these sources:

1. FHWA. The Code of Federal Regulations (23 CFR 646 Subpart B “Railroad-Highway Projects”) prescribes the FHWA policies and procedures for advancing Federal-aid projects involving railroad facilities.

2. *Nevada Administrative Code* (NAC). Chapter 705 “Railroads” presents the State of Nevada requirements with respect to railroads operating in the State. NDOT must receive approval from the Nevada Public Utilities Commission (PUC), which enforces the NAC, on all NDOT projects that impact railroads.

3. AREMA. The American Railway Engineering and Maintenance-of-Way Association (AREMA) provides a forum for the development and study of recommended engineering practices for railroad design and construction throughout the United States. To document these practices, the organization has published the *AREMA Manual for Railway Engineering*. This *Manual* has approximately the same status to railroad engineers as the *LRFD Specifications* has to highway bridge engineers.

4. Railroad Companies. The following Railroads operate in the State of Nevada:

   - Union Pacific (UPRR),
   - Nevada State Railroad Museum Boulder City Branch Line,
   - Northern Nevada, and
   - Virginia and Truckee.

   In addition, Burlington Northern Santa Fe (BNSF) has trackage rights over the UPRR lines, and AMTRAK (i.e., the *California Zephyr*) operates on the freight railroad lines across northern Nevada between Chicago and Oakland.

   UPRR has promulgated its specific criteria for highway bridges over railroads in a BNSF/UPRR publication “Guidelines for Railroad Grade Separation Projects.” NDOT policy is that this publication will be used in the development of all projects for highway bridges over railroads.

5. *LRFD Specifications*. LRFD Article 3.6.5.2 presents criteria for the design of highway abutments and piers within 50 ft of the centerline of a railway track.

For each highway-bridge-over-railroad project, the bridge designer’s responsibility is to evaluate each of the above during project development. Section 21.1 has been organized by project design element and, as applicable, references one or more of the above sources for the information.

21.1.2 Structure Type and Configuration

Chapter 11 of the *NDOT Structures Manual* presents NDOT criteria on the selection and configuration of a structure type for highway bridges. Specifically for highway bridges over railroads, the following applies:

1. Span Length/Configuration. Railroads usually require that their tracks and maintenance roads be clear spanned. Therefore, the typical span configuration over a railroad is a single-span or three-span bridge.

2. Structure Type. Railroads may prefer bridges that do not use falsework over their tracks. This usually limits the superstructure selection to structural steel girders or precast concrete girders. Continuous steel girder bridges can span up to 400 ft; precast, prestressed concrete I-girder bridges can span up to 150 ft.

See Chapter 11 for more information.

21.1.3 Geometrics

21.1.3.1 Basic Configuration

The basic geometric configuration of the railroad cross section passing beneath a highway bridge is based on the following:

- number and type of tracks,
- drainage treatments,
- access/maintenance roadway (if present),
- lateral clearances, and
- vertical clearances.

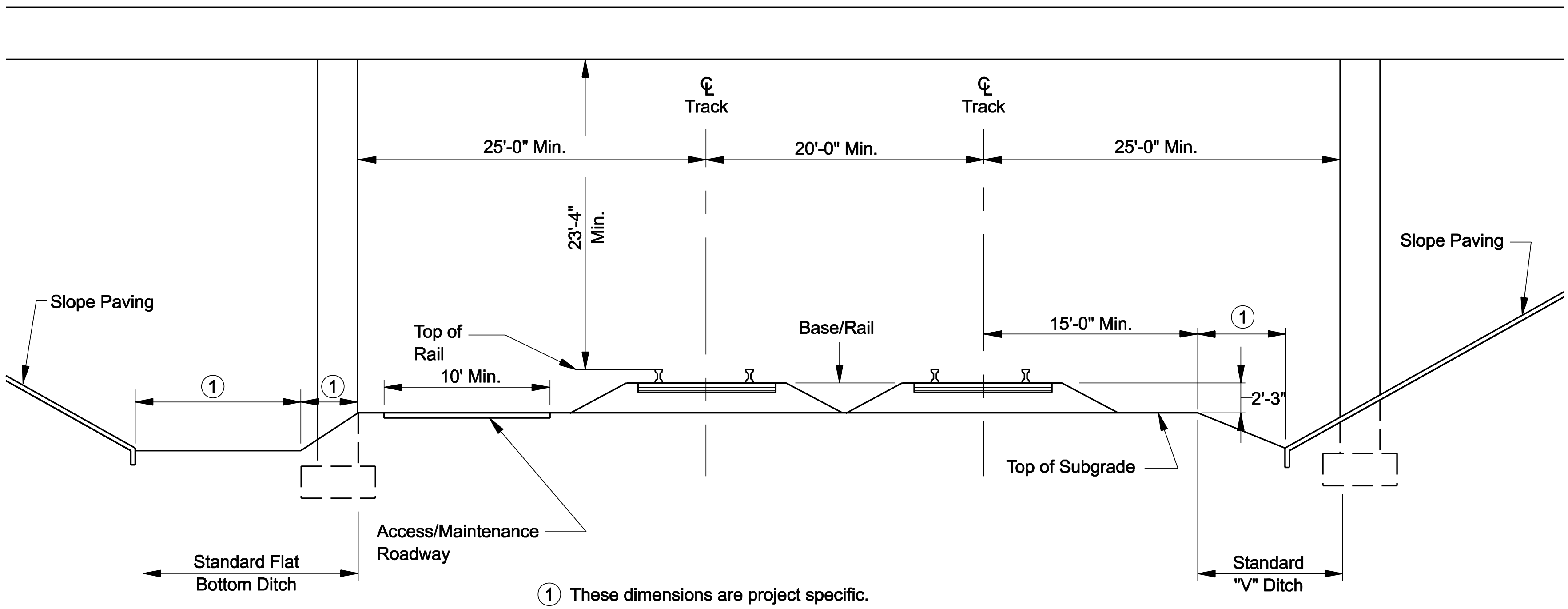
Figure 21.1-A presents the basic railroad cross section based on these variables. This Figure can be used for preliminary design and the preparation of the Bridge Front Sheet. The following Sections present additional information that must be considered.

21.1.3.2 Lateral Clearances

21.1.3.2.1 *FHWA*

The Appendix to Subpart B of 23 CFR 646 presents FHWA Federal-aid participation limits for lateral clearances. The following summarizes these criteria:

1. Basic Clearance. FHWA will fully participate in the costs of a 20′-0″ horizontal distance measured at right angles from the centerline of track at the top of rails to the face of the embankment slope at a height equal to the elevation of the top of the outside rail.



Notes:

1. *Horizontal dimensions shown are perpendicular to centerline of track.*
2. *Horizontal dimensions shown are the minimum distances to construct a standard railroad roadbed section. Actual required horizontal clearances may need to be increased due to the existing roadbed section and alignment, location of parallel ditches, and/or hydrologic conditions.*

RAILROAD CLEARANCES

Figure 21.1-A

2. Additional Clearance. FHWA will participate in lateral clearances greater than 20′-0″:

- to provide for drainage, if justified by a hydraulic analysis; or
- to allow adequate room to accommodate special conditions, if the railroad demonstrates that this is its normal practice.

3. Maintenance Equipment. FHWA will participate in an additional 8′-3″ of lateral clearance (an increase in the clearance from either No. 1 or No. 2) for off-track maintenance equipment, provided that adequate horizontal clearance is not available in adjacent spans of an existing access road or by evidence of the future need for such equipment.

4. Piers. All piers should be placed at least 9′-3″ horizontally from the centerline of the track and preferably beyond the drainage ditch. However, based on UPRR requirements, NDOT policy is to place piers beyond the drainage ditch or at a 25′-0″ horizontal clearance, whichever is more and if practical.

5. Multiple Tracks. For multiple track facilities, all dimensions apply to the centerline of the outside track.

21.1.3.2.2 *UPRR*

See Sections 4 and 5 of the “UPRR Guidelines for Railroad Grade Separation Projects” for the Railroad Company’s criteria for permanent lateral clearances. Section 4.1.2 of the Guidelines requires a minimum spacing of 20 ft between two freight tracks and 25 ft between freight and commuter tracks.

For minimum temporary horizontal construction clearances, the railroad underpass shall provide 12 ft, as measured perpendicular from the centerline of the nearest track to all physical obstructions including but not limited to formwork, stockpiled materials, parked equipment, bracing or other construction supports. The temporary horizontal construction clearance shall provide sufficient space for drainage ditches parallel to the standard roadbed section or provide an alternative system that maintains positive drainage.

21.1.3.2.3 *Nevada Administrative Code*

§705 of the *Nevada Administrative Code* presents the lateral clearance requirements for railroads in Nevada. These requirements are typically less than those required for Federal-aid projects. The minimum centerline of main track to centerline of main track is 14′-0″. The minimum centerline of main track to non-main track is 15′-0″.

21.1.3.2.4 *AREMA*

AREMA stipulates the following:

1. Horizontal Clearances (Tangent Track). Abutments and/or piers for overhead bridge structures shall be located to clear the ditches of a typical track roadbed section and, where possible, be set with a minimum of 25 ft from the face of pier to the centerline of the track.

2. Horizontal Clearances (Curved Tracks). On curved track, the lateral clearances on each side of the track centerline shall be 1½ in per degree of curve on the railroad alignment. When the fixed obstruction is adjacent to the tangent track but the track is curved within 80 ft of the obstruction, the lateral clearances on each side of the track centerline shall be increased as shown in Figure 21.1-B.

   On superelevated track, the track centerline remains perpendicular to a plane across the top of rails. Where the track is superelevated, clearances on the inside of the curve shall be increased by 3½ in for each inch of elevation differential between the inside and outside edges of the superelevated section.

Distance from Obstruction to Curved Track (ft)	**Increase Per Degree of Curvature (in)**
20	1½
40	1⅛
60	¾
80	⅜

Note: To convert radius of curve (R, in ft) to degree of curvature (D, based on the chord definition), $D = 2(\sin^{-1}(50/R))$.

LATERAL CLEARANCE INCREASE
(For Tracks on Horizontal Curves)

Figure 21.1-B

21.1.3.3 Vertical Clearances

21.1.3.3.1 *FHWA*

The Appendix to Subpart B of 23 CFR 646 presents FHWA Federal-aid participation limits for vertical clearances. The following summarizes these criteria:

1. Basic Clearance. FHWA will fully participate in the costs of a vertical clearance of 23′-4″ above the top of rails, which includes an allowance for future ballasting of the railroad tracks.

2. Additional Clearance. Vertical clearances greater than 23′-4″ may be approved when the Public Utilities Commission requires a vertical clearance in excess of 23′-4″ or on a site-by-site basis where justified by the Railroad to the satisfaction of NDOT and FHWA. A Railroad's justification for increased vertical clearance should be based on an analysis of engineering, operational and/or economic conditions at a specific structure location.

3. Electrification. Federal-aid highway funds are eligible to participate in the cost of providing vertical clearances greater than 23′-4″ where a Railroad establishes to the satisfaction of NDOT and FHWA that it has a definite, formal plan for electrification of its rail systems where the proposed grade separation project is located. The plan must cover a logical, independent segment of the rail system and be approved by the Railroad's corporate headquarters. For a 25-kv line, a vertical clearance of 24′-3″ may be approved. For a 50-kv line, a vertical clearance of 26′-3″ may be approved.

A Railroad's justification to support its plans for electrification shall include:

- maps and plans or drawings showing those lines to be electrified;
- actions taken by its corporate headquarters committing it to electrification including a proposed schedule; and
- actions initiated or completed to date implementing its electrification plans such as documenting the funding amounts and the identification of structures, if any, where the Railroad has expended its own funds to provide added clearance for the proposed electrification.

If available, the Railroad's justification should also include information on its contemplated treatment of existing grade separations along the section of its rail system proposed for electrification.

The cost of reconstructing or modifying any existing railroad-highway grade separation structures solely to accommodate electrification will not be eligible for Federal-aid highway fund participation.

4. Temporary Clearances. For temporary applications, the minimum vertical clearance for a highway over railroad may be reduced to 21′-0″ upon approval of the Railroad.

5. Summary. See Figure 11.9-A for a summary of vertical clearance information for highway bridges over railroads.

21.1.3.3.2 *UPRR*

See Sections 4 and 5 of the "UPRR Guidelines for Railroad Grade Separation Projects" for the Railroad Company's criteria for permanent vertical clearances. In general, UPRR stipulates the FHWA maximum vertical clearance of 23′-4″ (for Federal-aid participation) as its minimum vertical clearance. In addition, UPRR requires additional vertical clearance for items such as:

- correction of sag in the track,
- construction requirements, and
- future track raises.

UPRR will consider the potential need for track re-profiling when evaluating plans for new or widened overhead structures. Preliminary plan submittals from NDOT must include track survey information at 100-ft maximum centers for a minimum of 1000 ft on both sides of the structure centerline.

The railroad underpass shall provide a minimum temporary vertical construction clearance of 21 ft as measured above the top of high rail for all tracks. The 21-ft temporary vertical clearance shall not be violated due to deflection of formwork. Greater temporary vertical clearances may be required. The temporary vertical clearances are subject to Railroad local operating unit requirements.

21.1.3.3.3 *Nevada Administrative Code*

§705 of the *Nevada Administrative Code* requires a minimum vertical clearance of 23′-0″. Use this clearance for all non Federal-aid projects.

21.1.3.4 Pier Protection

To limit damage by the redirection and deflection of railroad equipment, piers supporting highway bridges over railways and with a clear distance of less than 25 ft from the centerline of a railroad track shall be of heavy construction (defined below) or shall be protected by a reinforced concrete crash wall. The following will apply:

1. Single-Column Piers. Crashwalls for single-column piers shall be a minimum of 2′-6″ thick and shall extend a minimum of 10 ft above the top of high rail. The wall shall extend a minimum of 6 ft beyond the column on each side in the direction parallel to the track.

2. Multiple-Column Piers. The columns shall be connected with a wall of the same thickness as the columns or 2′-6″, whichever is greater. The wall shall extend a minimum of 2′-6″ beyond the end of outside columns in a direction parallel to the track and shall extend at least 4 ft below the lowest surrounding grade.

3. Reinforcing Steel. Reinforcing steel to adequately anchor the crashwalls to the column and footing shall be provided.

4. Heavy Construction. For piers of heavy construction, crashwalls may be omitted. Heavy construction is considered as solid piers with a minimum thickness of 2′-6″ and a length of 20 ft; single-column piers of a minimum of 4 ft by 12′-6″ dimensions; or any other solid bent sections with equivalent cross sections and a minimum of 2′-6″ thickness. In addition, LRFD Article 3.6.5.2 applies to piers not protected by crashwalls.

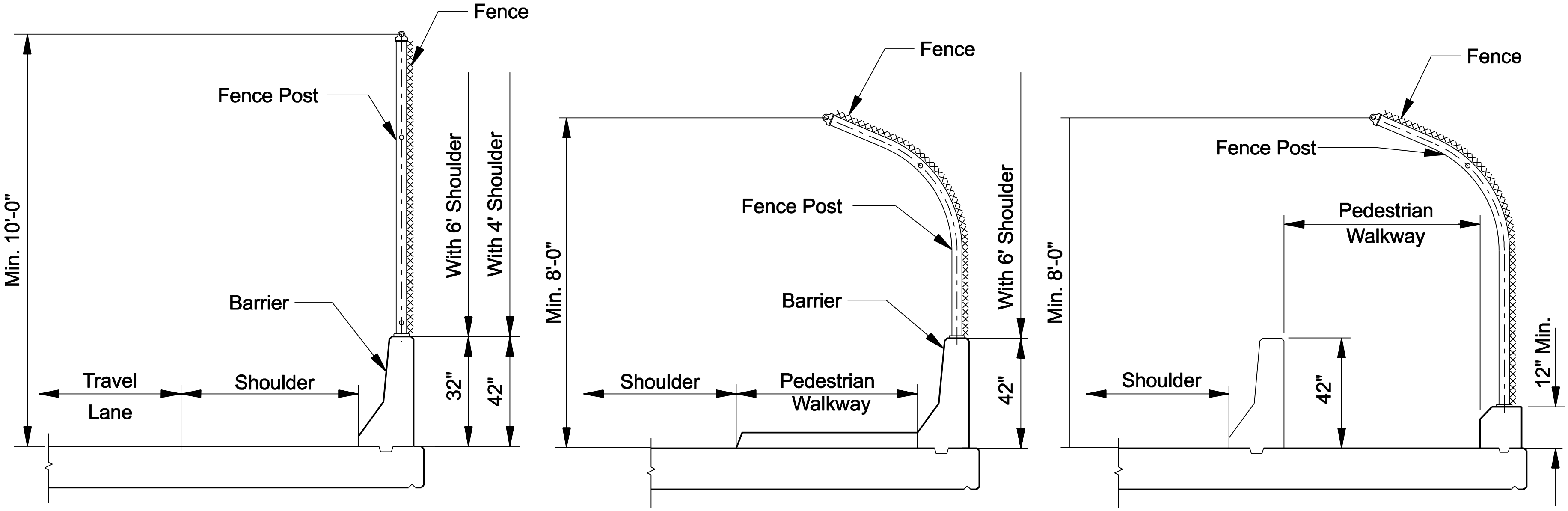
See Section 5.5.2 of the “UPRR Guidelines” for additional information.

21.1.3.5 Side Slopes

To prevent embankment material from sloughing and drainage waters from undermining the track subgrade, embankment slopes adjacent to tracks should be paved with concrete around the curved face to a line opposite the abutment or other appropriate slope protection. Provide self-cleaning paved ditches to carry water through the highway overpass area and disperse the water away from the track. Side slopes shall be no steeper than 2H:1V. See Section 5.5.3 of the “UPRR Guidelines” for additional information.

21.1.4 Fencing

A protective fence across the highway bridge shall be provided on both sides of highway bridges over railroads. The limits of the fence with barrier rail shall extend to the limits of the Railroad right-of-way or a minimum of 25 ft beyond the centerline of the outermost existing track, future track or access road, whichever is greater. All parallel overhead structures that have a gap of 2 ft or more shall be protected with fencing. Structures with a gap of 2 ft or less shall either have the gap covered or be fenced on both sides. Figure 21.1-C illustrates acceptable fencing applications for bridges over UPRR track. See Sections 4.6 and 5.4.2 of the “UPRR Guidelines” for more information.



FENCE ON BARRIER

WALKWAY FENCE
ON BARRIER

WALKWAY FENCE ON
CURB With BARRIER

Note: Barrier heights shown differ from UPRR guidelines in order to satisfy AASHTO height requirements.

FENCING APPLICATIONS
(UPRR)

Figure 21.1-C

21.1.5 Control of Drainage from Highway Bridge Deck

Deck drains shall not be allowed to discharge onto railroad right-of-way. Section 16.4 of the *NDOT Structures Manual* discusses bridge deck drainage. Where drains are required within the Railroad right-of-way, a closed drainage system shall be used, and the drainage shall be directed away from the Railroad right-of-way. See Section 5.7 of the "UPRR Guidelines" for more information.

21.1.6 Construction Requirements

For information on shoring for construction excavations, see the "UPRR Guidelines for Temporary Shoring" and the *AREMA Manual for Railway Engineering*. In addition, see Plan No. 710000 "General Shoring Requirements" in the "UPRR Guidelines for Railroad Grade Separation Projects." Figure 21.1-D duplicates a portion of UPRR Plan No. 710000.

Section 4.4 of the "UPRR Guidelines for Railroad Grade Separation Projects" discusses many other elements of construction that apply. In addition, UPRR has published a separate document "Guidelines for Preparation of a Bridge Demolition and Removal Plan for Structures over Railroad" that should be consulted as needed.

Temporary steel casings must be used in the construction of drilled shafts that are in load influence zones of railroad tracks. Casings shall be used for the entire length of drilled shafts. The required thickness of casings shall be decided on a case-by-case basis. The Geotechnical Section is responsible for ensuring that the following are shown in the contract documents:

- temporary casing for the entire length of the drilled shaft, and
- the minimum thickness of the temporary casing.

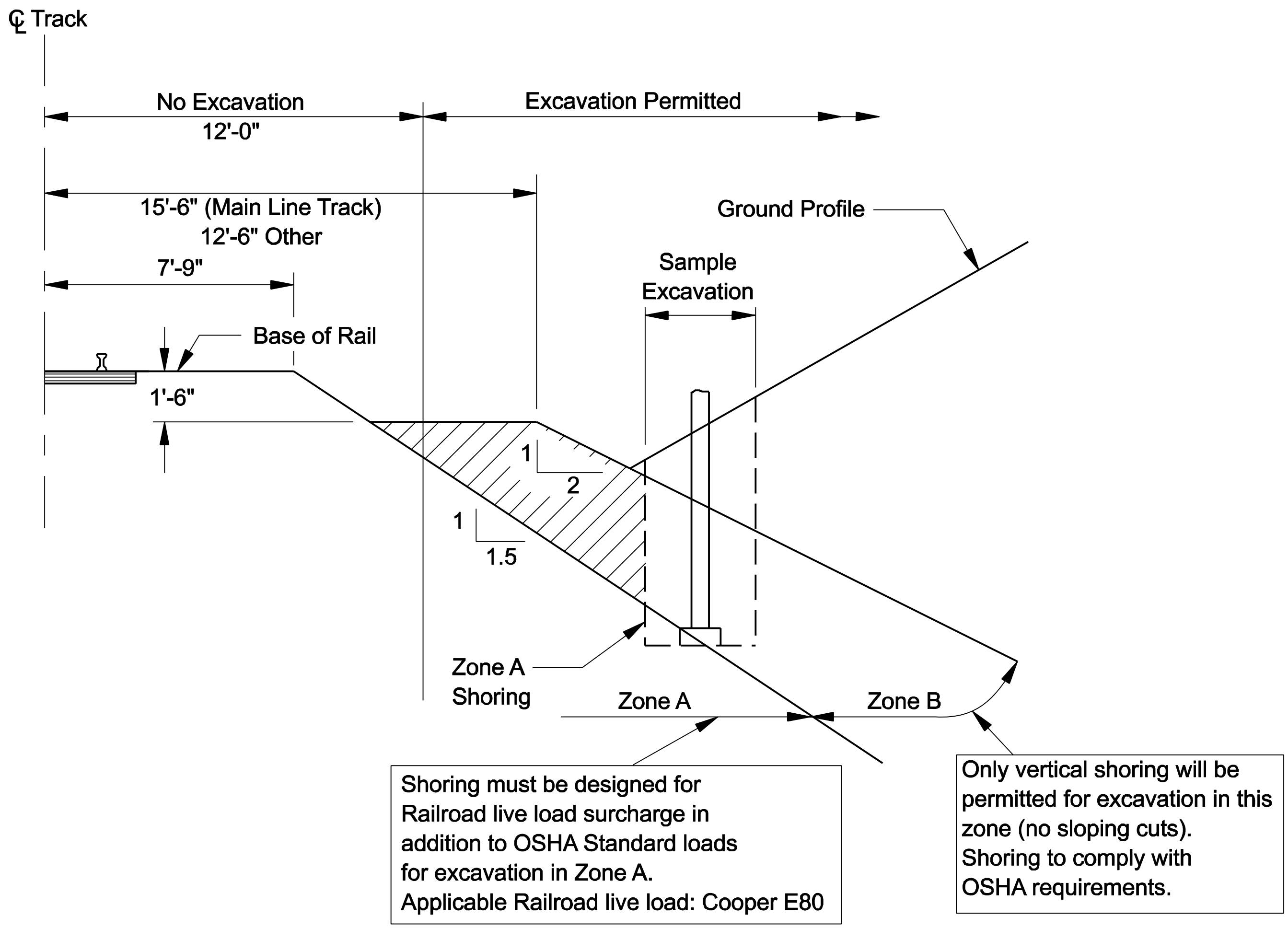
21.1.7 Utilities

See Section 4.9 of the "UPRR Guidelines" for the Railroad requirements for utilities. See Section 16.5.4 of the *NDOT Structures Manual* for NDOT requirements.

21.1.8 NDOT Procedures

21.1.8.1 Right-of-Way Division, Utilities Section

The Right-of-Way Division, Utilities Section is responsible for coordinating with the Railroad Companies where NDOT projects impact railroads. The Utilities Section's responsibilities include obtaining cost estimates for securing agreements with Railroads for the relocation and adjustment of their facilities, as required for highway construction, and conducting direct negotiations with Railroads, when necessary.



TEMPORARY SHORING
(UPRR)

Figure 21.1-D

21.1.8.2 Project Development

Because of the unique nature of highway-railroad grade separations, special coordination is necessary where a railroad alignment and a highway alignment intersect or where these alignments are in close proximity to each other. The bridge designer must prepare a preliminary design and Bridge Front Sheet considering the minimum required horizontal and vertical clearances, which is submitted to the Utilities Section. The Utilities Section will coordinate with representatives from the impacted Railroad.

The Utilities Section will advise the bridge designer if the preliminary design is acceptable or if revisions are needed. Final bridge and roadway plans will be developed and then forwarded to the Utilities Section. The Utilities Section will forward the final plans to the Railroad for review and final approval for construction. The final plan submittal to the Railroad must be stamped by a Nevada registered professional civil/structural engineer.

Temporary structures, falsework, shoring, erection plans, demolition, etc., produced by the contractor will also require a Registered Professional Engineer stamp. The contractor will be responsible for producing the drawings for these items and ensuring that the drawings are stamped by a Nevada registered professional civil or structural engineer. The Special Provisions for a project must include these requirements plus appropriate review times. NDOT must review and approve the submittal drawings prior to submitting to the Railroad for its approval.

21.2 RAILROAD BRIDGES OVER HIGHWAYS

21.2.1 Preliminary Design

In the past, concrete railroad bridges were typically discouraged, most likely because of the tradition of wooden and steel railroad bridges. However, currently, the US railroad industry spends approximately 50% of its bridge capital improvements on concrete bridge construction. Concrete bridges represent approximately 20% of the total railroad bridge inventory based upon bridge length.

Many Railroads prefer simple-span bridges to continuous-span bridges, believing that they are easier to maintain and construct with less interruption to traffic. Specifically, Railroads find that:

- Simple-span railroad bridges have a long history of good performance.
- Repair or replacement of simple-span superstructure elements can be accomplished with less interruption of railroad traffic than for continuous-span superstructures.
- Construction of simple-span bridges can be completed more quickly than the construction of continuous-span bridges.
- Substructure settlement can be accommodated more easily with simple spans thereby reducing potential traffic interruption.

21.2.2 AREMA Requirements

The American Railway Engineering and Maintenance-of-Way Association (AREMA) *Manual for Railway Engineering* provides recommended practices for railroad bridge design. A major difference between the AASHTO *LRFD Specifications* and the AREMA *Manual* is the live-load model. AREMA specifies the Cooper E 80 load, which is 18 concentrated axle loads followed by a trailing uniform load, or a fraction thereof for each track. Further, the *Manual for Railway Engineering* is based upon the allowable stress design (ASD) methodology.

Chapter 22

BRIDGE REHABILITATION

NDOT STRUCTURES MANUAL

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Chapter 22
BRIDGE REHABILITATION

Chapter 22 presents NDOT's practices and policies for bridge rehabilitation and bridge widening.

22.1 INTRODUCTION

22.1.1 Importance

Properly timed bridge maintenance and rehabilitation can maximize the service life of a bridge and delay the need for its replacement. This will minimize the probability that these bridges will deteriorate to an unsafe or unserviceable condition. This protects the large capital investment in Nevada's inventory of bridges and minimizes the potential adverse consequences to the public.

22.1.2 Scope of Work Definitions

Section 10.2.4 presents scope of work definitions to distinguish between the various levels of bridge work. Specifically for the types of work addressed in Chapter 22, Section 10.2.4 provides definitions for:

- major bridge rehabilitation,
- minor bridge rehabilitation,
- safety work, and
- bridge widening.

22.1.3 Highway Bridge Program

The Federal Highway Bridge Program (HBP), formerly known as the Highway Bridge Replacement and Rehabilitation Program, provides funds for eligible bridges located on any public road. The HBP is the cornerstone of FHWA efforts to correct, on a priority basis, deficient bridges throughout the nation. The number of structurally deficient and/or functionally obsolete bridges in Nevada compared to the number nationwide is a factor in determining Nevada's share of HBP funds.

The Code of Federal Regulations (CFR) in 23 CFR, Part 650, Subpart D presents the Federal regulations that govern the funding, eligibility and application for HBP projects. The following summarizes the basic process:

- The National Bridge Inspection Standards (NBIS) requires that each State DOT develop and maintain a bridge inspection and inventory program for all public bridges within that State not owned by a Federal agency. See Chapter 28 for a discussion on the Nevada Bridge Inspection Program.

- As part of its Bridge Inspection Program, NDOT submits to FHWA the Structure Inventory and Appraisal (SI&A) data based on NDOT's bridge inspections.

- Based on the SI&A data, a Sufficiency Rating is calculated for each bridge, which is used as the basis for establishing eligibility and priority for the replacement or rehabilitation of bridges. FHWA then provides each State with a list of bridges within that State that are eligible for HBP funding. FHWA also requires that no less than 15% of the funds must be used on public roads that are not on the Federal-aid system.

HBP funds can be used for total replacement or for rehabilitation. HBP funds can also be used for repainting structural steel bridges, non-corrosive deicers, deck replacements, preventive maintenance, seismic retrofit and program administrative costs. Due to the limited funding available under the HBP, NDOT policy is to provide priority to program administration, replacement projects and rehabilitation projects. HBP funds can also be used for a nominal amount of roadway approach work to connect the new bridge to the existing alignment or to tie in with a new profile. HBP funds cannot be used for long approach fills, connecting roadways, interchanges, ramps and other extensive earth structures.

The Sufficiency Rating (SR) (0-100) is based on a numerical equation that considers many aspects of a bridge (e.g., structural adequacy, safety, serviceability, functionality, detour length). The following applies:

1. Replacement. Bridges qualify for replacement with a SR less than 50 and must be classified as structurally deficient or functionally obsolete.

2. Rehabilitation. Bridges qualify for rehabilitation with a SR less than 80 and must be classified as structurally deficient or functionally obsolete. Rehabilitation must correct all deficiencies that render the bridge eligible for HBP funding. In addition, consideration should be given to upgrading other features (e.g., bridge rails, approach guardrail, seismic retrofit) to current standards and including all needed repairs. Seismic retrofit is not considered a deficiency under the HBP.

3. Exception. If the cost of rehabilitation approaches the cost of replacement, then consider replacing the bridge. Coordination with FHWA is required to determine if a bridge eligible only for rehabilitation can be replaced.

4. 10-Year Rule. If a bridge has received HBP funds in the past for replacement or rehabilitation, it is not eligible for additional HBP funds for 10 years.

5. SR ≥ 80. If a bridge has an SR greater than or equal to 80, it is not eligible for HBP funds.

22.1.4 Nevada Bridge Management System

The Nevada Bridge Management System, using the AASHTOWare® software, PONTIS®, is currently used for data collection. Ultimately, PONTIS will be used for bridge inventory and asset management. When fully operational, PONTIS will assist NDOT in developing a Statewide bridge preservation program. See Chapter 29 for more discussion.

22.1.5 Rehabilitation Strategy

The development of a bridge rehabilitation project involves the following basic steps:

- Perform a field investigation of the existing bridge.
- Collect the available data on the existing bridge (e.g., as-built plans, bridge inspection reports, traffic volumes).
- Identify the necessary condition surveys and tests (e.g., coring, chain drag, chloride analysis, identifying fracture-critical members).
- Evaluate the data from the condition surveys and tests.
- Select the appropriate bridge rehabilitation technique(s) to upgrade the bridge to meet the necessary structural and functional objectives.

The remainder of Chapter 22 presents NDOT practices on implementing this bridge rehabilitation strategy.

22.2 BRIDGE REHABILITATION REPORT

22.2.1 NDOT Project Development Process

Chapter 3 discusses, with an emphasis on the bridge work portion, the overall project development process used by NDOT to advance a project from programming to completion of the contract document package. Early in project development, the Roadway Design Division prepares the Preliminary Design Field Survey (PDFS) Report. The PDFS Report presents the project location, termini, anticipated environmental/right-of-way impacts, project Scope of Work, etc. For those NDOT projects that will include bridge rehabilitation, the PDFS Report will document the anticipated work. The Structures Division prepares a Bridge Rehabilitation Report that becomes a part of the PDFS Report. Section 22.2.2 discusses the field work for and content/format of the Bridge Rehabilitation Report.

22.2.2 Field Inspection

After assimilation of the relevant background material (e.g., as-built plans, shop drawings, Bridge Inspection Reports, SI&A data, traffic data), the bridge designer will attend the PDFS and/or perform a separate site visit. One objective is to identify the various condition tests and surveys that may be needed. See Section 22.4. The following guidelines apply to the field inspection:

1. Attendees. Depending on the nature of the bridge rehabilitation, attendees may include the following representatives:

    - the Bridge Design Squad responsible for the project;
    - District maintenance and bridge maintenance, construction, utilities and right-of-way;
    - other NDOT units as deemed appropriate (e.g., Geotechnical Section, Hydraulics Section, Environment Services Division);
    - FHWA (if bridge is subject to oversight); and/or
    - local agency (if bridge is not on the State highway system).

2. As-Built Plans. The bridge designer should review the as-built plans from the various contracts that built or modified the bridge before the field inspection. The as-built plans are located in NDOT's Central Records. In addition, the bridge designer should review the change order file for each contract to identify changes not shown on the as-built plans.

3. Field Work. During the inspection, the bridge designer should:

    - note any areas of special concern (e.g., fatigue-critical details, bridge rail, width of structure, alignment, utilities);
    - take the necessary photographs showing approaches, side view, all four quadrants of the bridge, the feature being crossed, and any deficient features to be highlighted in the Report;

- ensure that all information is gathered as necessary to complete the Bridge Rehabilitation Report; and

- use the appropriate personal protective equipment.

In addition, the bridge designer should verify that the condition and configuration of the bridge matches the as-built plans. Determine if details match those shown in the plans and shop drawings. Check for evidence of repair work or revisions not indicated in the plans and shop drawings.

22.2.3 Bridge Rehabilitation Report

The Bridge Rehabilitation Report is intended to:

- document the findings from the field inspection, including photographs;
- identify deficient items and provide recommendations for upgrade or repair;
- document the seismic prioritization rating and provide recommendation for further seismic retrofit study;
- make recommendations on the proposed bridge rehabilitation improvements;
- note scour susceptibility and provide a recommendation for upgrade or repair, if appropriate;
- provide a preliminary project cost estimate; and
- identify a proposed strategy for traffic control during construction.

Figure 22.2-A presents the format and content of the Bridge Rehabilitation Report.

I. COVER SHEET/TITLE PAGE

Provide a cover sheet or title page as illustrated below.

BRIDGE REHABILITATION REPORT

STRUCTURE NUMBER: ____________________

ROUTE IDENTIFICATION AND FEATURE CROSSED:
____________ over _____________

PROJECT I.D. NUMBER: ____________________

PROJECT DESCRIPTION: ____________________________________

PREPARED BY: ___________________________ (NDOT/ Consultant designer)

DATE: _________________

II. TABLE OF CONTENTS

If the magnitude of the Report warrants, provide a Table of Contents segregated by major Report sections (e.g., “Existing Structure Data,” “Recommendations”).

III. FIELD INSPECTION DATA

Date of Inspection: ___________________
Time of Inspection: ___________________
Attendees (Name, Organization, Unit within Organization): ____________

IV. EXISTING STRUCTURE DATA

Include a copy of the Front Sheet, Geometric Sheet and appropriate detail sheets, and complete the data in Item B for information not covered or addressed in the plans.

A. Construction History

Year Built: ______________
Construction Contract(s): __________________________
Previous Repairs and Other Actions: (Provide details and year)

BRIDGE REHABILITATION REPORT

Figure 22.2-A

B. Structure/Dimensions

Deck Surface: (Original concrete deck, asphalt overlay, etc.)
Out to Out of Bridge Rail: (Width)
Skew: (Angle and direction; i.e., left or right)
Type of Superstructure: (Prestressed concrete, structural steel, etc.)
Spans: (No. and length of each span)
Type of Substructure/Foundation: (Pier type & shape, abutment type, piles or spread footings, etc.)

C. Geometrics

Functional Classification: ______________________________
Vertical Clearance: ___________________________________
Longitudinal Gradient: ________________________________
Cross Slope/Superelevation: ____________________________
Horizontal Degree of Curve: ____________________________
Vertical Curve (K-Value): ______________________________
Sidewalks: ____

D. Deck Protection

Epoxy-coated rebar, top or both mats: ______________________________________
Overlay (membrane, low-slump concrete, polymer): __________________________
Concrete type (conventional, EA, HPC): ____________________________________

E. Appurtenances

Bridge Rail: (Type, height)
Curbs: (Presence, height)
Pedestrian Fencing: (Type, height)

F. Approaches

Roadway Width: ______________________________
Surface Type: (Asphalt or concrete)
Guardrail: (Type)
Guardrail Transition: (Type)

V. ENVIRONMENTAL COMPLIANCE

Document the environmental factors that are likely to be involved, including the following:

BRIDGE REHABILITATION REPORT

Figure 22.2-A
(Continued)

- impact on wetlands (a color photograph of each quadrant should be labeled and included);
- possible permitting issues;
- historical significance of the bridge, if applicable (i.e., Section 106); and
- potential construction staging areas.

See the *NDOT Environmental Services Manual* for more information on environmental considerations and permits.

VI. EXISTING CONDITIONS

Review the most recent Bridge Inspection Reports, compare current condition, and provide brief statements as needed for the recommended action based on the condition of the various structural elements. Make reference to NBI ratings and PONTIS Condition States where applicable. The following provides guidance on the content of this section.

A. Bridge Deck

1. General. Note the overall condition of the bridge deck (excellent, fair, poor).
2. Overlay. If applicable, indicate the type, depth, condition and year installed.
3. Surface Condition. Describe the extent and location of spalling, presence of existing patches, extent and location of cracking, relative indication of available skid resistance, etc.
4. Underside Condition. Describe the overall condition of the deck underside (if visible), extent and location of cracking, signs of leakage, etc.
5. Joints. Indicate the type, number, location and condition. If joint rehabilitation will be considered, measure gap widths and record ambient temperature.
6. Drainage. Indicate the condition of bridge deck inlets. Describe the adequacy and condition of the drainage conveyance system beneath the bridge deck. If known, state any deck drainage problems (e.g., excessive ponding).
7. Bridge Rail. Indicate the type, condition and height of the bridge rail, and provide a statement on whether or not the rail meets NDOT's current performance criteria.

BRIDGE REHABILITATION REPORT

Figure 22.2-A
(Continued)

8. <u>Curbs/Sidewalks</u>. If present, provide a statement on the overall condition.

B. <u>Superstructure</u>

1. <u>General</u>. Note the overall condition of the superstructure (excellent, fair, poor).

2. <u>Repair/Maintenance Work</u>. If known or if visible, identify any prior repair and maintenance work performed.

3. <u>Specific Deficiencies</u>. Where applicable, identify the extent and location of any specific structural deficiencies (e.g., cracking, spalling of concrete, rust on metal components, deformation, loss in concrete or metal components).

4. <u>Fracture Critical Members and Low Fatigue Life Details</u>. Identify any fracture critical or fatigue-prone members.

5. <u>Damage</u>. Identify any damage due to collisions by vessels, vehicles, etc.

6. <u>Bearings, Pedestals</u>. State the functionality of these elements and indicate any deficiencies, including seismic compatibility.

C. <u>Substructures/Foundations</u>

1. <u>General</u>. Note the overall condition of the substructures and foundations and slope protection (excellent, fair, poor). Also indicate the substructure and foundation types and materials.

2. <u>Repair/Maintenance</u>. If known or if visible, identify any prior repair or maintenance work performed (e.g., patching of concrete).

3. <u>Specific Deficiencies</u>. Where applicable, identify the extent and location of any specific structural deficiencies (e.g., cracking, leaching, deterioration, settlement, rotation, exposed reinforcement).

4. <u>Drainage</u>. Indicate overall adequacy of drainage with respect to the substructure and foundation and note any problems (e.g., erosion).

D. <u>Seismic Assessment</u>. Research seismic prioritization rating and Seismic Zone. Indicate the structure's apparent ability to meet current NDOT criteria for seismic load-carrying capacity based on the Seismic Zone (e.g., adequate or inadequate support length). Provide a preliminary assessment of potentially vulnerable elements and provide recommendation for further seismic retrofit study.

BRIDGE REHABILITATION REPORT

Figure 22.2-A
(Continued)

E. Scour Assessment. Research scour assessment and provide recommendations for mitigation.

F. Approaches

1. General. Note the overall condition of the approaches (excellent, fair, poor).

2. Approach Slab/Pavement. Indicate the condition of the approach slabs, pavement relief joints and the approach pavement immediately adjacent to the bridge or approach slab.

3. Guardrail. For each quadrant, indicate the type, length(s) and condition of the guardrail, guardrail transition (or the absence of one), and guardrail end treatment and provide a statement on whether or not the system meets current performance criteria.

4. Roadway Drainage. Indicate the location and condition of drainage structures adjacent to the bridge or approach slabs.

G. Slope Pavement. Note the overall condition and material of existing slope pavement (excellent, fair, poor).

H. Utilities. Identify all apparent existing utilities, attached to various structural elements, and their locations (e.g., conduits, electrical boxes, gas lines, water lines).

VII. RECOMMENDATIONS

A. Condition Surveys and Tests

Section 22.4 identifies an array of condition surveys and tests. Indicate which of these, if any, should be undertaken before definitive rehabilitation recommendations are made.

B. Bridge Deck

Identify the proposed work to the bridge deck. Where applicable, document the following:

- patching (indicate approximate depth) or replacement of a portion or all of the existing bridge deck;
- the proposed bridge deck overlay in conjunction with deck patching;

BRIDGE REHABILITATION REPORT

Figure 22.2-A
(Continued)

- removal, replacement and/or addition of curbs, pedestrian fencing, sidewalks and/or medians;
- bridge expansion joint repair and/or replacement;
- drainage improvements; and
- upgrading or replacing bridge rails and/or guardrail-to-bridge-rail transitions.

Section 22.5 identifies bridge deck rehabilitation techniques.

C. Superstructure

Identify the proposed work, if any, to the existing superstructure. Where applicable, document the following:

- removing, replacing, adding or strengthening structural members;
- patching concrete structural members;
- replacing or repairing bearing assemblies;
- cleaning and painting structural steel beams; and
- fatigue repair or upgrade.

Sections 22.6 and 22.7 identify rehabilitation techniques for concrete and steel superstructures.

D. Substructures/Foundations

Identify the proposed work, if any, to the existing substructure and foundation. Where applicable, document the following:

- repairing, adding or strengthening structural members;
- providing seismic retrofit measures (e.g., seat extensions, restrainers);
- repairing deteriorated concrete;
- implementing remedial actions for hydraulic scour; and
- constructing or repairing slope protection.

Section 22.8 identifies rehabilitation techniques for the substructure and foundation. See Section 22.9 for information on seismic retrofit rehabilitation techniques.

BRIDGE REHABILITATION REPORT

Figure 22.2-A

(Continued)

E. Approaches

Identify the proposed work to the bridge approaches. Where applicable, document the following:

- repairs to or replacement of approach slabs and bridge rail;
- repairs to or replacement of pavement relief joints; and
- repairs to or replacement of bridge rail/guardrail connections.

F. Utilities

Identify any known utility adjustments necessitated by the bridge rehabilitation work. Contact the Utilities Section for more information on the utility.

G. Traffic Control During Construction

Identify the proposed strategy for maintaining traffic during construction and how it coincides with the proposed rehabilitation. This could include alternating one-way traffic with signals, using stage construction or diverting the traffic to a detour route.

VIII. PRELIMINARY COST ESTIMATE

Provide a preliminary cost estimate for the proposed bridge rehabilitation work. See Chapter 6.

IX. ECONOMIC COST COMPARISON

A major bridge rehabilitation should include a cost estimate for rehabilitation versus replacement.

X. SCHEMATICS

Provide schematics for the proposed bridge improvements. The schematics should indicate the following:

- width for:
    + travel lanes,
    + shoulders,

BRIDGE REHABILITATION REPORT

Figure 22.2-A
(Continued)

  + clear roadway,
  + out-to-out of bridge rail, and
  + overhangs;

- roadway cross slope;
- height of curb;
- sidewalk width;
- bridge rail type and basic dimensions; and
- girder type, depth and spacing.

XI. PHOTOGRAPHS

Provide color photographs depicting in sufficient detail the overall condition of the structure and its elements. The pictures can then be used in reviewing and evaluating the existing condition and rehabilitation recommendations.

BRIDGE REHABILITATION REPORT

Figure 22.2-A
(Continued)

22.3 BRIDGE REHABILITATION LITERATURE

The design of new bridges is based primarily on the AASHTO *LRFD Bridge Design Specifications*. No national publication exists that, in a single document, presents accepted practices, policies, criteria, etc., for the rehabilitation of existing bridges as the *LRFD Specifications* provides for original design. However, the highway research community has devoted significant resources to identifying practical, cost-effective methods to rehabilitate existing highway bridges.

Publications are available that may be of special interest to the bridge designer when rehabilitating an existing bridge. The designer is encouraged to evaluate the research literature to identify publications that may be useful on a project-by-project basis. Visit the websites for FHWA, AASHTO, Transportation Research Board, etc., for more information. The bridge designer should also review the publications available in NDOT's research library.

22.4 BRIDGE CONDITION SURVEYS AND TESTS

Section 22.4 discusses NDOT policies and practices for condition surveys and tests for a bridge rehabilitation project. The discussion does not pertain to any condition surveys and tests performed for the Nevada Bridge Inspection Program (see Chapter 28) nor the NDOT Bridge Management System (see Chapter 29).

22.4.1 General

The bridge designer is responsible for:

- arranging and conducting field reviews;
- requesting specific tests to be performed by others (e.g., chloride-content analysis);
- evaluating data collected during the field survey and provided by others;
- determining the appropriate scope of rehabilitation or if replacement is appropriate; and
- preparing the contract documents.

The decision on the type and extent of bridge rehabilitation is based on information acquired from condition surveys and tests. The selection of these condition surveys and tests for a proposed project is based on a case-by-case assessment of the specific bridge site. The bridge designer should request assistance from the Non-Destructive Testing Squad (see Chapter 26) and from the Materials Division. The Materials Division can offer support in the following areas:

- geotechnical evaluation/foundation recommendations,
- concrete corings for cracking and/or strength assessment,
- chloride sampling and testing,
- corings to determine depth of surfacing materials,
- slope stability analysis and recommendations,
- evaluation of bond strength of overlay materials, and
- skid testing.

22.4.2 Concrete Bridge Decks

22.4.2.1 General

For this Chapter, concrete bridge decks include the structural continuum directly supporting the riding surface, expansion joints, curbs, barriers, approach slabs and utility hardware (if suspended from the deck). Concrete bridge decks include decks supported on girders and the top slabs of cast-in-place box girders. The bridge deck and its appurtenances provide the following functions:

- support and distribution of wheel loads to the primary structural components;
- protection of the structural components beneath the deck;
- provide a smooth riding surface; and
- a safe passageway for vehicular and bicycle/pedestrian traffic (e.g., skid-resistant surface, bridge rails, guardrail-to-bridge-rail transitions).

Any deterioration in these functions warrants investigation and possible remedial action. A bridge deck has a finite service life, which is a function of both adverse and beneficial environmental factors. The most common cause of concrete bridge deck deterioration is the intrusion of chloride ions from roadway deicing agents into the concrete. The chloride causes formation of corrosive cells on the steel reinforcement, and the corrosion product (rust) induces stresses in the concrete resulting in cracking, delamination and spalling. Chloride ion (salt) penetration is a time-dependent phenomenon. There is no known way to prevent penetration, but it can be decelerated such that the service life of the deck is not less than that of the remaining structure. Chloride penetration is, however, not the only cause of bridge deck deterioration. Other significant problems include:

1. Freeze-Thaw. Results from inadequate air content of the concrete. Freezing of the free water in the concrete causes random, alligator cracking of the concrete and then complete disintegration. There is no known remedy other than replacement.

2. Impact Loading. Results from vehicular kinetic energy released by vertical discontinuities in the riding surface, such as surface roughness, delamination and inadequately set or damaged expansion joints. Remedial actions are surface grinding, overlay or replacement of deck concrete and rebuilding expansion joints.

3. Abrasion. Normally results from metallic objects, such as chains or studs attached to tires. Remedial actions are surface grinding or overlay.

Certain factors are symptomatic indicators that a bridge deck may have a shorter than expected service life and that it is actually in the latter phases of its service life. Some examples are:

- extensive cracking (shrinkage, stress, etc);
- extensive delamination;
- exposed reinforcing steel; and
- spalls.

The deck can be placed into one of the following categories (based on NBI ratings):

1. Very good decks that need little attention. These are the (8) and (9) rated decks.

2. Decks that are in reasonably good shape and need no substantial repair, but their lives can be extended with a nominal maintenance expenditure. These are the (7) rated decks. Decks in this condition range would most likely need some minor crack sealing and minor patching.

3. Decks that need considerable repair but are still quite sound and capable of serving adequately for five to ten more years. These are candidates for repair and overlay with some type of non-permeable concrete. These are the (5) and (6) rated decks. The designer would most likely consider an overlay for bridge decks in this condition range, depending on the extent of chloride contamination.

4. Decks that are no longer serviceable and will soon need replacement regardless of any remedial action. Significant expenditures of funds are not justified until replacement. However, minor maintenance expenditures could extend the remaining life several years. These are the (3) and (4) rated decks. Decks in these conditions fall into the "replace deck" category.

When considering a bridge for rehabilitation, the Structures Division requests a number of tests to collect data on the deck's condition. The data allows the designer to determine whether deck rehabilitation or deck replacement is appropriate and, if the choice is rehabilitation, the information allows the determination of the appropriate level of treatment.

The following information may be collected during a deck evaluation:

- a plot locating existing delaminations, spalls and cracks;
- representative measurements of crack width;
- measurements of the depth of cover on the top mat of reinforcing steel;
- sampling and laboratory analysis to determine the existing levels of chloride contamination;
- measurements of electrical potential on a grid pattern to locate areas of active corrosion; and
- deck concrete compressive strength assessed through destructive testing of deck core samples.

Expect to obtain at least some degree of confirmation and conflicting test results because these field tests each have a degree of uncertainty. Thus, sampling multiple locations within a traffic lane is important to estimate the true condition of the deck and the extent of active corrosion. Engineering judgment must be applied when analyzing multiple test results. The following provides more information on each type of data collected and their use in determining an appropriate deck treatment.

22.4.2.2 Visual Inspection

Description: A visual inspection of the bridge deck should establish:

- the approximate extent of cracking, representative crack widths and spalling;
- evidence of any corrosion;
- evidence of pattern cracking, efflorescence or dampness on the deck underside;
- rutting of the riding surface and/or ponding of water;
- operation of expansion joints;
- functionality of deck drainage system; and
- bridge rails and guardrail-to-bridge-rail transitions meeting current NDOT standards.

Purpose: The visual inspection of the bridge deck will achieve the following:

- By establishing the approximate extent of cracking and crack width, corrosion, delamination and spalling (and by having evidence of other deterioration), the designer can determine if a more extensive inspection is warranted.
- The inspection will identify substandard roadside safety appurtenances.

When to Use: All potential deck rehabilitation projects.

Analysis of Data: Pattern cracking, heavy efflorescence or dampness on the deck underside suggests that this portion of the deck is likely to be highly contaminated and active corrosion is taking place. In addition, the designer should consider:

- traffic control;
- timing of repair;
- age of structure;
- average annual daily traffic (AADT);
- slab depth;
- structure type;
- depth of cover to reinforcement; and
- crash history (e.g., wet weather).

22.4.2.3 Delamination Sounding

Description: Establishes the presence of delamination, based on audible observation, by chain drag or hammer. Based on the observation that delaminated concrete responds with a "hollow sound" when struck by a metal object. See ASTM D4580 *Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding*.

Purpose: To determine the location and area of delamination.

When to Use: On all concrete deck rehabilitation projects, except where asphalt overlays prevent performance of the test.

Analysis of Data: Based on the extent of the bridge deck spalling, the following will apply:

- 10% delamination of surface area is a rough guide for considering remedial action.
- 40% delamination is a rough guide for considering bridge deck replacement.

22.4.2.4 Chloride Analysis

Description: A chemical analysis of pulverized samples of concrete extracted from the bridge deck. Concentrations of water-soluble chlorides are determined using the *Gravimetric Method — Silver Chloride Method* as described in *Scott's Standard Methods of Chemical Analysis*, 6th Edition, March 1962, (D. Van Nostrand). As an option, chloride testing by others for NDOT may be conducted using potentiometric titration with silver nitrate per AASHTO T 260 *Sampling and Testing for Chloride Ion in Concrete and Concrete Raw Materials*.

Purpose: To determine the chloride content profile from the deck surface to a depth of approximately 3 in or more.

When to Use: Use on bridge decks where the need for major rehabilitation or replacement is anticipated. Take chloride samples at three to five locations along the travel lane per span from each span 100 ft or less in length. Increase the number of samples for longer spans.

Analysis of Data: The "threshold" or minimum level of water-soluble chloride contamination in concrete necessary to corrode reinforcing steel is approximately 1.3 to 2.0 lbs/yd^3. Chloride concentrations of less than this threshold indicate a sound deck that will in most cases not require rehabilitation. Consideration may be given to adding a deck protection system. Chloride concentrations within or greater than this range above the top reinforcing mat require the removal of at least enough concrete so that the remaining concrete contamination is below the threshold.

Threshold or greater chloride concentrations at the level of the top reinforcing mat require either 1) demolition to remove enough concrete to ensure that the remaining concrete is below the threshold values, or 2) possibly deck replacement. Threshold contamination or worse at or near the level of the bottom mat of reinforcing steel may require deck replacement

22.4.2.5 Pachometer Readings

Description: The pachometer produces a magnetic field in the bridge deck. A disruption in the magnetic field, such as induced by a steel reinforcing bar, is displayed.

Purpose: To determine the location and depth of steel reinforcing bars. These properties can be established to a depth of approximately 4 in.

When to Use: Pachometer readings are used on all concrete rehabilitation projects to verify reinforcement location as needed. They are often used to locate steel to avoid damage when drilling or coring concrete.

22.4.2.6 Ground-Penetrating Radar (GPR)

Description: Ground-coupled or air-coupled radar antennas emit very short, precisely timed pulses of radio-frequency electromagnetic energy into the bridge deck. When the pulses transition from either one material to another, or across areas of the same material having different dielectric properties (such as from an area of sound concrete into a deteriorated area), part of the energy is reflected back to a receiver positioned at the surface, and varying amounts of energy are absorbed or diffracted within the material. Deteriorated materials absorb/refract more energy than sound materials. Computer software analyzes variations in the return strength versus absorption of this pulse-echo and the length of time required for the echo to return to the antenna. The program will generate condition reports.

Purpose: When the GPR system is used to survey a concrete bridge deck, the following information can be obtained:

- apparent location and depth of unsound concrete (subject to ground-truth verification),
- depth of the reinforcing steel, and
- thickness of the bridge deck and overlay materials.

This information is used to supplement other inspection methods to locate sections of a bridge deck in need of repair.

When to Use: Asphalt-overlayed bridge decks are excellent candidates for GPR investigation, as are decks constructed using stay-in-place formwork. GPR should be considered where traffic must be maintained during testing. Because vehicle-mounted antennas can be effective at low to moderate speeds, the need for lane closures may possibly be avoided. The test is nondestructive; therefore, there is no follow-up repair work.

22.4.2.7 Half-Cell Method

Description: Copper/copper sulphate half-cell method for the measurement of electrical potential as an indicator of corrosive chemical activity in the concrete. See ASTM C876 “Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete.”

Purpose: To determine the level of active corrosion in the bridge deck.

When to Use: This test method is not often used by NDOT. Even if a concrete deck has a wearing surface, half-cell readings can be made after areas of the deck are exposed.

Analysis of Data: A voltage potential difference of -0.35 volts or less indicates active corrosion; more recent work suggests that -0.23 volts is the threshold of corrosion. Less negative readings indicate more active corrosion, while higher negative (smaller in absolute value) readings indicate lower corrosion.

22.4.2.8 Coring

Description: 2-in or 4-in diameter cylindrical cores are taken. In decks with large amounts of reinforcement, it is difficult to avoid cutting steel if 4-in diameter cores are used.

Purpose: To establish strength, composition of concrete, crack depth, position of reinforcing steel.

When to Use: On all concrete deck rehabilitation projects when doubt exists on the compressive strength or soundness of the concrete or if the visual condition of the reinforcement is desired.

Analysis of Data: Less than 2 in of concrete cover is considered inadequate for corrosion protection. If compressive strengths are less than 3 ksi, the designer must determine whether to proceed with the deck rehabilitation or to proceed with a deck replacement.

22.4.2.9 Testing for Alkali-Silica Reactivity (ASR)

Alkali-silica reactivity is the process in which an expanding gel is produced by the breakdown of certain minerals (mostly glass-type silica) in the presence of moisture within the highly alkaline concrete environment. The expanding gel induces tensile forces in the concrete matrix causing cracking of the concrete. This cracking allows free water to infiltrate into the concrete creating more gel and, subsequently, more expansion. Ultimately, the concrete fails or disintegrates.

Test procedures for ASR are tabulated below.

Test	Purpose	Description
ASTM C856, petrographic examination of hardened concrete	Outlines petrographic examination procedures for hardened concrete; useful in determining condition or performance	Short-term visual (unmagnified) and microscopic examination of prepared samples
ASTM C856 (AASHTO T299), annex uranyl-acetate treatment procedure	Identifies products of ASR in hardened concrete	Staining of a freshly exposed concrete surface and immediate viewing under UV light
Los Alamos staining method	Identifies products of ASR in hardened concrete	Staining of a freshly-exposed concrete surface with two different reagents

The ASTM C856 annex uranyl-acetate treatment procedure and the Los Alamos staining method identify small amounts of ASR gel whether they cause expansion or not. These tests should be supplemented by the ASTM C856 petrographic examination, or physical tests, for determining concrete expansion.

22.4.3 Superstructure

As defined in this *Manual*, the superstructure consists of the bearings and all of the components and elements resting upon them. See Section 22.4.2 for condition assessments and surveys on bridge decks. The following briefly describes those condition surveys and tests that may be performed on the superstructure elements to determine the appropriate level of rehabilitation.

22.4.3.1 Visual Inspection

Description: A visual inspection of the superstructure should include an investigation of the following to supplement the information contained in the NBI Bridge Inspection Report:

- surface deterioration, cracking and spalling of concrete;
- major loss in concrete components;
- evidence of efflorescence;
- corrosion of reinforcing steel or prestressing tendons;
- loss in exposed reinforcing steel or prestressing tendons;
- pealing and delaminating coating system;
- corrosion of structural metal components;
- loss in metal components due to corrosion;
- cracking in metal components;
- excessive deformation in components;
- loosening and loss of rivets or bolts;
- deterioration and loss in wood components;
- damage due to collision by vehicles, vessels or debris;
- leakage through expansion joints;
- ponding of water on abutment seats;
- state and functionality of bearings; and
- distress in pedestals and bearing seats.

Purpose: To record all deterioration and signs of potential distress for comparison with earlier records and for initiating rehabilitation procedures if warranted.

When to Use: On all bridge rehabilitation projects.

Analysis of Data: As required, if the deterioration is deemed significant enough to result in loss of load-carrying capacity.

22.4.3.2 Ground-Penetrating Radar (Concrete)

See Section 22.4.2.6.

22.4.3.3 Testing for Alkali-Silica Reactivity (ASR) (Concrete)

See Section 22.4.2.9.

22.4.3.4 Fracture-Critical Members (Steel)

A fracture-critical member is a metal structural component, typically a superstructure tension or bending member that would cause collapse of the structure or span if it fails. Fracture-critical structures in Nevada have been identified and catalogued; contact the Assistant Chief Structures Engineer – Inventory/Inspection. The designer must recognize typical fracture-critical details when conducting the field review because it may affect the scope of bridge rehabilitation. Typical bridges in Nevada containing fracture-critical members are:

- steel trusses (pins, eye-bars, bottom chords and other tension members);
- two-girder steel bridges;
- transverse girders (supporting longitudinal beams and girders); and
- pin-and-hanger connections (located on suspended spans or at transverse girders).

22.4.3.5 Load-Induced Fatigue Analysis (Steel)

<u>Description</u>: Fatigue is defined as steady-state crack growth. Failure of the component can result from growth of existing flaws in steel members to a critical size at which fracture is no longer effectively resisted by the toughness of the steel. The crack growth is a function of:

- crack size;
- location of crack (i.e., stress concentration at the structural detail);
- toughness (energy-absorbing characteristics of metal);
- temperature; and
- frequency and level of nominal stress range (transient stresses).

<u>Purpose</u>: To establish type and urgency of remedial action.

<u>When to Use</u>: Where cracks, found by visual inspection, are believed to be caused by fatigue or at fatigue-prone details.

<u>Analysis of Data</u>: Analysis should be performed by a structural engineer who is experienced in fatigue-life assessment. For the analysis, fatigue characteristics of the metal should be established. For the stress range, the *LRFD Specifications* provides an upper-bound criterion of 75% weight of one design truck plus impact per bridge. The actual stress range of a given bridge component may be far lower than that specified by the *LRFD Specifications*, and it may be warranted to establish it by physical means. See Section 15.4 for further discussion. The following discussion illustrates how to calculate the stress cycles for existing bridges not satisfying the infinite-life check.

* * * * * * * * * *

For existing bridges not satisfying the infinite fatigue life check, LRFD Article 6.6.1.2.5 shall be used to define the total number of stress cycles (N) as:

$$N = (365)(75)n(ADTT)_{SL} \qquad \text{(LRFD Eq. 6.6.1.2.5-2)}$$

Where:

n = number of stress range cycles per truck passage. As defined in LRFD Article 6.6.1.2.5, for simple and continuous spans not exceeding 40 ft, n = 2.0. For spans greater than 40 ft, n = 1.0, except at locations within 0.1 of the span length from a continuous support, where n = 1.5.

ADTT = the number of trucks per day in one direction averaged over the design life of the structure.

$ADTT_{SL}$ = Average Daily Truck Traffic in a single lane = (p)(ADTT), which is LRFD Equation 3.6.1.4.2-1.

p = the fraction of truck traffic in a single lane. As defined in LRFD Article 3.6.1.4.2, when one direction of traffic is restricted to:

- 1 lane: p = 1.00
- 2 lanes: p = 0.85
- 3 or more lanes: p = 0.80

The portion of LRFD Equation 6.6.1.2.5-2 that is $(365)(75)(ADTT)_{SL}$ represents the total accumulated number of truck passages in a single lane during the 75-year design life of the structure. If site-specific values for the fraction of truck traffic data are unavailable from the NDOT Traffic Information Services, the values provided in LRFD Table C3.6.1.4.2-1 may be used.

Example 22.4-1

Given: Total number of truck passages in a single lane during the 75-year design life (from NDOT Traffic Information Services) = 9.75×10^6

Two spans, 160 ft each

Longitudinal connection plate located 30 ft from the interior support

Unfactored DL stress at the toe of the connection plate-to-web weld = 4 ksi compression

Unfactored fatigue stresses at the toe of the connection plate-to-web weld using unmodified single-lane distribution factor = 3.9 ksi tension and 4.5 ksi compression

Find: Determine the fatigue adequacy at the toe of a longitudinal connection plate-to-web weld with a transition radius of 4 in with the end welds ground smooth.

Solution:

Step 1: The LRFD Specifications classifies this connection as Detail Category D. Therefore:

- A = Detail Category Constant = 22.0×10^8 ksi^3 (LRFD Table 6.6.1.2.5-1)
- $(\Delta F)_{TH}$ = Constant Amplitude Fatigue Threshold = 7.0 ksi (LRFD Table 6.6.1.2.5-3)

Step 2: Compute the factored live-load fatigue stresses by applying dynamic load allowance and fatigue load factor and removing the multiple presence factor:

Tension: 3.9(1.15)(0.75)/1.2 = 2.8 ksi
Compression: 4.5(1.15)(0.75)/1.2 = <u>3.2 ksi</u>
Fatigue Stress Range: = 6.0 ksi

Step 3: Determine if fatigue must be evaluated at this location:

- Net tension = (DL stress) – (Fatigue stress)
- Net tension = 4 ksi (Compressive) – 3.9 ksi (Tensile) = 0.1 ksi (Compressive)

Although there is no net tension in the web at the location of the longitudinal connection plate, the unfactored compressive DL stress (4 ksi) does not exceed twice the tensile fatigue stress (5.6 ksi). Therefore, fatigue must be considered.

Step 4: Check for infinite life:

First, check the infinite life term. This will frequently control the fatigue resistance when traffic volumes are large. $(\Delta F)_n = ½(\Delta F)_{TH} = 0.5(7.0) = 3.5$ ksi. Because the fatigue stress range (6.0 ksi) exceeds the infinite life resistance (3.5 ksi), the detail does not have infinite fatigue life.

Step 5: Determine "n" for LRFD Equation 6.6.1.2.5-2:

The span exceeds 40 ft and the point being considered is located more than 0.1 of the span length away from the interior support. Therefore, n = 1.0.

Step 6: Using LRFD Equation 6.6.1.2.5-2, compute the number of stress cycles:

$N = (9.75 \times 10^6)(n)$
$N = (9.75 \times 10^6)(1.0)$
$N = 9.75 \times 10^6$

Step 7: Using LRFD Equation 6.6.1.2.5-1, compute the nominal fatigue resistance:

Nominal Fatigue <u>Resistance</u>		75-Year Life <u>Resistance</u>		Infinite Life <u>Resistance</u>
$(\Delta F)_n$	=	$(A/N)^{1/3}$	≥	$½(\Delta F)_{TH}$

Step 8: Check to see if the detail will have at least a 75-year fatigue life:

$$(\Delta F)_n = (A/N)^{1/3} = [(22.0 \times 10^8)/(9.75 \times 10^6)]^{1/3} = 6.1 \text{ ksi}$$

The 75-year factored fatigue resistance (6.1 ksi) exceeds the fatigue stress range (6.0 ksi); therefore, the detail is satisfactory.

* * * * * * * * *

22.4.4 <u>Substructures</u>

As discussed in Chapter 18, substructure elements include piers and abutments. For the purpose of Chapter 22, substructures also include foundations, which are discussed in Chapter

17. The following briefly describes those condition surveys and tests that may be performed on these elements to determine the appropriate level of rehabilitation.

22.4.4.1 Visual Inspection

Description: A visual inspection of the substructure components should address the following to supplement the NBI Bridge Inspection Report:

- surface deterioration, cracking and spalling of concrete;
- major loss in concrete components;
- evidence of corrosion in reinforcing steel;
- loss in exposed reinforcing steel;
- deterioration or loss of integrity in wood components;
- leakage through joints and cracks;
- dysfunctional drainage facilities;
- collision damage;
- changes in geometry such as settlement, rotation of wingwalls, tilt of retaining walls, etc;
- seismic vulnerabilities;
- accumulation of debris;
- erosion of protective covers;
- changes in embankment and water channel; and
- evidence of significant scour.

Purpose: To record all deterioration and signs of potential distress for comparison with earlier records and for initiating rehabilitation procedures if warranted.

When to Use: On all potential bridge rehabilitation projects.

Analysis of Data: As required, if the deterioration is deemed significant enough to result in loss of load-carrying capacity.

22.4.4.2 Ground-Penetrating Radar

See Section 22.4.2.6.

22.4.4.3 Testing for Alkali-Silica Reactivity (ASR)

See Section 22.4.2.9.

22.4.5 Summary

The bridge condition surveys, test, analyses and reports will indicate the extent of the problems and the objectives of rehabilitation. Sections 22.5 through 22.9 present specific bridge rehabilitation techniques that the designer may employ to address the identified deficiencies. These Sections are segregated by structural element (i.e., bridge decks, steel superstructures, concrete superstructures, substructures and seismic retrofit).

22.5 BRIDGE DECK REHABILITATION

22.5.1 General

Chapter 16 provides an in-depth discussion on the design of bridge decks that are constructed compositely in conjunction with concrete and steel girders and as part of cast-in-place, post-tensioned box girders for new bridges. Many of the design and detailing practices provided in the Chapter may also apply to deck rehabilitation. Therefore, the designer should review Chapter 16 to determine its potential application to a bridge deck rehabilitation project.

22.5.2 Typical NDOT Practices

The following discusses typical NDOT practices for bridge deck rehabilitation.

22.5.2.1 Bridge Deck Overlay

The following identifies typical NDOT practices on bridge deck overlays:

1. Patching. Patching the bridge deck with a fast-setting concrete should be considered a temporary measure to provide a reasonably acceptable riding surface until a more permanent solution can be applied. The longevity of patches is highly dependent upon the deck preparation, patching materials and location of the patch. Avoid patching with asphalt.

2. Polymer Concrete Overlays. Polymer concrete overlays have been in use in Nevada since the early 1990s. They have a good performance history. Contrary to cement-based overlays, the construction of a polymer concrete overlay is enhanced in a dry climate. In general, polymer concrete is preferred over other overlay materials.

3. Resin Overlays. Thin resin overlays have been occasionally used in Nevada since the early 2000s. They have a fair to good performance history. The thin resin overlay is used for bridge deck protection and to restore skid resistance.

4. Asphalt Overlay with Sheet Membrane. This method was used in the 1960s and early 1970s with limited success. The difficult construction tolerances for surface preparation, membrane discontinuities and application temperature have resulted in poor results. However, it is still used occasionally on certain bridges such as side-by-side boxes where reflective cracking through a concrete or polymer overlay is a concern.

   A damaged waterproofing system is counterproductive in that it retains salt-laden water and continues supplying it to the deck which, thus, never dries out. Also, rain water or washing efforts cannot remove the salt.

5. Replacement Overlay. It is acceptable to remove an existing overlay and replace it with a new one. NDOT policy is to not allow a new overlay to be placed over an existing bridge deck overlay, because it is counterproductive and adds to the dead load of the structure.

22.5.2.2 Expansion Joints

The service life of bridge deck expansion joints is much shorter than that of the bridge, and leaking and faulty joints represent a hazard for the deck and the main structural components. Where applicable, the bridge deck rehabilitation should be consistent with the criteria described in Chapter 19 relative to the design of bridge deck expansion joints. Chapter 19 identifies the following types of expansion joints that are typically used to retrofit an existing bridge:

- strip seal,
- preformed joint filler,
- asphaltic plug, or
- pourable seals.

22.5.3 <u>Rehabilitation Techniques</u>

The remainder of Section 22.5 presents a brief discussion on bridge deck rehabilitation techniques that may be considered:

- Patching
- Polymer Concrete Overlay
- Resin Overlay
- Waterproof Membrane/Asphalt Overlay
- Epoxy-Resin Injection
- Crack Sealant
- Silane Seal
- Joint Rehabilitation
- Joint Replacement
- Upgrade/Retrofit Bridge Rails
- Approach Slabs

22.5.3.1 Patching

A permanent repair can be assured only if all concrete in areas having a chloride content sufficient to sustain corrosion are removed. For partial depth repairs, concrete should be removed to a depth of ¼ in plus the maximum size of the aggregate below the bottom of the top mat of reinforcing steel. The actual corrosion threshold can be as low as 1.3 lb of Cl per cubic yard of a typical deck concrete, but a value of 2 lb of Cl per cubic yard is commonly accepted as the level beyond which removal of the concrete is warranted. Unless the contaminated concrete is removed, differences in the surface conditions on the reinforcing bar may cause the formation of anodic and cathodic areas and a resumption of the corrosion process. However, removal of concrete below the reinforcing steel may be extremely costly, and complete removal and replacement of the deck may be more economical. Patching of the deck followed by the installation of a protective overlay is a less costly and often used alternative.

An evaluation of the corrosion process indicates that patches cannot be considered permanent repairs, and field experience tends to verify this conclusion. Newly delaminated areas are often found adjacent to areas patched months before. Nevertheless, patching can be an appropriate temporary action until more extensive restoration is performed, and it can provide substantial service with the subsequent installation of a protective overlay.

The area to be patched can be defined in the deck by sounding and GPR. The concrete is then removed using pneumatic hammers with a maximum mass of 35 pounds. Surface preparation is critical. Roughen the exposed surface to ¼ in amplitude and avoid feathered edges. Any exposed reinforcing steel is cleaned. A bonding agent is applied to the existing concrete surface, when required, and the repair material is placed and cured.

A wide variety of materials has been used for patching bridge decks. Although conventional Portland cement concrete is often used, many other materials have been developed to provide rapid strength development and to allow early opening of the deck to traffic. It is essential that the manufacturer's requirements for mixing, placing and curing be rigidly followed. If a polymer concrete overlay is proposed, it can also be used as the deck patching material.

Bonding components vary with the repair materials. Usually a bonding epoxy is brushed into the clean, sound surface of the underlying concrete prior to placement of a cement-based patch. Some prepackaged polymer-modified concretes develop sufficient adherence so that a bonding agent is not required. Consult the manufacturers of all prepackaged fast-setting patching materials for the proper bonding agents. A methacrylate primer is used for polymer overlay patches.

22.5.3.2 Polymer Concrete Overlay

Polymer concrete is a combination of a polymer resin (polyester/styrene) and well-graded durable aggregates. Unlike concrete overlays, polymer overlays provide a waterproof barrier. Its normal thickness is ¾ in but can be placed as thin as ½ in and has been placed as thick as 4 in. A methacrylate primer is needed to keep the polyester/styrene resin from being in contact with the alkaline concrete deck. The methacrylate primer also has the benefit of sealing any cracks in the deck.

The polymer concrete overlay has a set time of less than 2 hours. Traffic can be placed on the overlay usually on the same day of construction. Surface preparation includes shotblast removal of the top paste of concrete to ensure a bond between the deck and overlay.

22.5.3.3 Resin Overlay

Resin overlays consist of 1 to 3 layers of resin and fine aggregate. A special resin is spread on the deck with fine aggregate broadcast on top. Once the resin sets, this operation is repeated until the system is complete. Resin overlays provide a waterproof barrier.

Resin overlays set and cure quickly, and traffic can be placed on the overlay usually on the same day as application. Resin overlays are thin (i.e., approximately ⅜ in). Tapering of the approach roadway is not usually required with resin overlays.

22.5.3.4 Waterproof Membrane/Asphalt Overlay

NDOT does not normally use waterproof membranes with an asphalt overlay. It is impossible to inspect a bridge deck covered with asphalt. The overlay adds dead load to the bridge, which can reduce live-load capacity and the overlay traps moisture in the concrete further aggravating corrosion of the slab reinforcing. However, on certain bridges such as side-by-side box beams, a waterproof membrane with asphalt overlay has demonstrated better performance than concrete or polymer overlays. The concrete and polymer overlays have developed cracking at the joints between the box beams due to the differential movement of the boxes.

A waterproof membrane with asphalt overlay has comparable construction time frames as the other overlay systems. The surface preparation for the membrane is minimal. Only high points or exposed rocks must be removed so that they will not puncture the membrane. Traffic should never be allowed on the exposed membrane.

22.5.3.5 Epoxy-Resin Injection

Epoxy-resin injection is commonly used to fill cracks in decks. Because the resin is injected under pressure, it is usually possible to fill the entire depth of crack. Reinforcing bars are located with a Pachometer, and holes are drilled to an appropriate depth into the cracks between reinforcing bars. The crack between the injection ports is sealed with a putty-like epoxy applied to the concrete surface by hand. Injection ports are placed at the holes, and a suitable epoxy system capable of bonding to wet surfaces is injected into the entry hole under pressure until it appears in the exit hole(s). A pumping system, in which the two components of the epoxy are mixed at the injection nozzle, is usually employed.

22.5.3.6 Crack Sealant

A low-viscosity organic liquid compound is flooded over the deck, and fills the cracks by gravity and capillary action. Accordingly, the success of this operation depends on the crack size, selection of the appropriate compound, temperature, contamination on the crack walls and the skill of the operator. The deck surface must be cleaned prior to application of the sealant. This includes power sweeping the entire surface and blowing all loose material from the cracks using high pressure air. All traces of asphalt and petroleum products must be removed by sand blasting. Care is needed to not damage the existing concrete surface.

22.5.3.7 Silane Seal

One method of slowing the entry of chloride ions into the concrete is by sealing its surface with a penetrating silane sealer. Penetrating silane sealers have a service life of from 3 to 5 years but are a low-cost preventive maintenance alternative for sound decks. The entire surface is treated and is applied as recommended by the manufacturer. The method of surface preparation is the same as for the Crack Sealant.

22.5.3.8 Joint Rehabilitation and Replacement

Joint rehabilitation refers to the repair of a portion of an existing joint and not its complete replacement. Joint rehabilitation includes repairing or replacing loose or broken restrainers on strip seal expansion joints, failed header materials adjacent to joints or torn seals. In most cases, the failure is due to vehicle impact. Failure may also be due to incompressibles in the joint.

Broken concrete adjacent to the joint should be removed with hand-operated equipment limited in size to approximately 15 lbs. If the size of the broken concrete is large, dowelled reinforcement should be added to hold the repair together. Quick-set concrete or polymer concrete can be considered for patching material; a material compatible with the existing header material should be used. The concrete should be saw cut outside the limits of the broken concrete to an approximate depth of 1 in. Adjust the depth of saw cut to avoid damaging the reinforcing steel. All corners of the patch must be square.

Bridges with asphalt overlays require a concrete header adjacent to the expansion joint unless an asphaltic plug joint is used. Concrete headers should be at least 8 in wide but preferably 12 in. Deck concrete should be removed down to a distance below the top mat of reinforcing steel to provide development length for the new header reinforcement.

Use a minimum number of joint splices with a full-length seal preferred. Torn strip seals can be repaired by vulcanizing or gluing. However, vulcanizing is preferred.

Where joint rehabilitation is not feasible, a replacement of an existing damaged or malfunctioning joint may be necessary.

Chapter 19 provides guidance on joint selection.

22.5.3.9 Upgrade/Retrofit Bridge Rails

Section 16.5.1 presents NDOT practices for new bridge rails, which is based on NCHRP 350. Desirably, existing bridge rails on a bridge rehabilitation project will meet the criteria in Section 16.5.1 or will be replaced with a new, NCHRP-350 compliant bridge rail. However, this is not always practical for a variety of reasons (e.g., dead load considerations, incompatibility with an existing bridge deck). Therefore, the following presents NDOT policy on upgrading existing bridge rails:

1. Crash History. Review the crash history and the maintenance and repair history of the bridge rail.

2. Critical Design Details. Inspect the existing bridge rail to verify the integrity of critical design details, such as:

    - base plate connections,
    - anchor bolts,
    - welding details,
    - concrete cracking, and
    - reinforcement development.

3. Safety Deficiencies. Even in the absence of an adverse crash history, an inspection of the existing bridge rail may reveal inherent safety deficiencies in the rail design, such as:

    - potential for snagging (i.e., no blockouts);
    - presence of curb in front of bridge rail;
    - inadequate height; and/or
    - inadequate guardrail-to-bridge-rail transition.

Ultimately, any retrofit to an existing bridge rail, intended to improve the rail to an acceptable performance level, will be made on a case-by-case basis. The following describes three basic conceptual approaches for a retrofit:

1. Guardrail Retrofit. A relatively inexpensive retrofit is to install an approaching roadside barrier that meets the NCHRP 350 criteria and to continue the longitudinal member of the guardrail across the existing bridge rail to provide rail continuity. This retrofit can significantly improve the impact performance of a substandard bridge rail. Extending guardrail across a bridge is limited to short-span bridges only.

2. Concrete Retrofit of Steel Rails. A concrete barrier, either an F-shape or vertical wall, can sometimes be added to an existing substandard bridge rail. However, this retrofit is only feasible if the existing bridge can accommodate the additional dead load and if the existing curb and railing configuration can meet the anchorage requirements of the retrofitted barrier.

3. Concrete Retrofit of Concrete Rails. An F-shape or vertical wall can be used to replace an existing concrete bridge rail not meeting the height or strength requirements. A partial or complete removal of the existing rail is required depending upon the amount of existing reinforcing steel, its development and the condition of the existing concrete. The challenge of most retrofits is the additional strength requirements needed to meet the requirements of Section 16.5.1. In most cases, additional reinforcement is required, but there is a limited amount of deck thickness to develop the reinforcement. The width of the rail can be increased, but an evaluation of the existing deck/superstructure may be required to accommodate the additional dead load.

22.5.3.10 Approach Slabs

An approach slab should preferably be added during rehabilitation to any existing bridge without one.

22.6 CONCRETE SUPERSTRUCTURES

22.6.1 General

Chapter 14 provides a detailed discussion on the design of concrete superstructures. Many of the design and detailing practices provided in this Chapter also apply to the rehabilitation of an existing concrete bridge. Therefore, the designer should review Chapter 14 to determine its potential application to the bridge rehabilitation project.

22.6.2 Rehabilitation Techniques

The remainder of Section 22.6 presents a brief discussion on concrete superstructure rehabilitation techniques that may be considered:

- Remove/Replace Deteriorated Concrete
- Crack Repair
- Bearings
- Post-Tensioning Tendons
- FRP Strengthening

22.6.2.1 Remove/Replace Deteriorated Concrete

A clean, sound surface is required for any repair operation; therefore, all physically unsound concrete, including all delaminations, should be removed.

To prevent damaging sound concrete, pneumatic hammers should be restricted to 15 lbs. Saw-cut the edges of removal areas to an approximate depth of 1 in, adjust the depth of saw cut to avoid damaging reinforcing steel. Concrete should be removed to a depth of ¼ in plus the maximum size of the aggregate below the exposed reinforcing steel. Replace missing or severely corroded reinforcing steel; tie all reinforcing at each intersection point. Ensure that all corners are square. Surface preparation is critical. Roughen the exposed surface to ¼ in amplitude and avoid feathered edges. Finally, the existing concrete surface and the exposed bars should be blast cleaned.

Verify that the remaining concrete is capable of resisting its weight, any superimposed dead load, live load (if the bridge will be repaired under traffic), formwork, equipment and the plastic concrete without the need for supplemental temporary support. The formwork should resist the plastic concrete without slipping or bulging. Prior to placing concrete, the forms should be cleaned and treated with a bond breaker.

If the concrete surface is cleaned by high-pressure water blasting, it should be allowed to dry before any epoxy bonding agent is applied. The new concrete should be applied before the bonding agent sets. For large repair areas, the use of pneumatically placed concrete (shotcrete) may be considered.

22.6.2.2 Crack Repair

Epoxy resin injection is commonly used to fill cracks in superstructure units. Because the resin is injected under pressure, it is usually possible to fill the entire depth of crack. Reinforcing bars are located with a Pachometer and holes are drilled to an appropriate depth into the cracks between reinforcing bars. The crack between the injection ports is sealed with a putty-like epoxy applied to the concrete surface by hand. Injection ports are placed at the holes, and a suitable epoxy system capable of bonding to wet surfaces is injected into the entry hole under pressure until it appears in the exit hole(s). A pumping system, in which the two components of the epoxy are mixed at the injection nozzle, is usually employed.

22.6.2.3 Bearings

Often, the existing bearings may only need cleaning or repositioning. Extensive deterioration, or frozen bearings, may indicate that the design should be modified. A variety of elastomeric devices may be substituted for sliding and roller bearing assemblies. If the reason for deterioration is a leak in the expansion joint, the joint should be repaired.

If the bearing is seriously dislocated, its anchor bolts badly bent or broken, or the concrete seat or pedestal is structurally cracked, the bridge may have a system-wide problem usually caused by temperature or settlement, and should be so investigated.

The bearing design may require alteration if warranted by seismic effects. See Section 22.9.

See Chapter 20 for more information on bearings.

22.6.2.4 Post-Tensioning Tendons

The addition of post-tensioned tendons can be used to restore the strength of the prestressed concrete girders where original strands or tendons have been damaged. Strengthening by post-tensioning may also be applied to non-prestressed concrete girders.

Collision of overheight vehicles or equipment with a bridge constructed with prestressed concrete girders may result in damage to or severing of the girder tendons. Exposure to water and salt may also cause damage, particularly where the concrete cover is damaged or cracked. Because the steel tendons determine the load-carrying capacity of the girder, any damage impairs resistance and must be repaired. External longitudinal post-tensioning along the sides of pier caps can be used to close transverse cracks and improve seismic performance.

At a minimum, the following steps apply:

1. Conduct an investigation on the extent of damage.

2. Perform a structural evaluation to determine the extent of repair.

3. Evaluate the existing diaphragms to ensure their adequacy to support the end anchorage of the tendons.

4. Determine the placement of the temporary load to be applied to the bridge prior to removal and placement of concrete in prestressed concrete girders, if any, to ensure the proper distribution of loads in the final condition.

The post-tensioning system should be designed and constructed in accordance with the manufacturer's recommendations. Wedge-type anchorages are susceptible to high seating losses for short-length tendons. High-strength prestressing bars are preferred in this application.

22.6.2.5 FRP Strengthening

Webs of girders with inadequate internal shear reinforcement or damaged reinforcement can be strengthened with externally applied, fiber-reinforced polymer (FRP) laminate reinforcement bonded to the surfaces of the webs. Bending capacity can also be increased with the application of FRP reinforcement. NCHRP Project 12-75 “Development of FRP Systems for Strengthening Concrete Girders in Shear” is developing a design method for this process.

22.7 STEEL SUPERSTRUCTURES

22.7.1 General

Chapter 15 provides a detailed discussion on the structural design of steel superstructures for new bridges. Many of the design and detailing practices provided in that Chapter also apply to the rehabilitation of an existing steel superstructure. Therefore, the designer should review Chapter 15 to determine its potential application to bridge rehabilitation projects.

22.7.2 Rehabilitation Techniques

The remainder of Section 22.7 presents a brief discussion on steel superstructure rehabilitation techniques that may be considered:

- Fatigue Damage Countermeasures
- Section Losses
- Strengthening
- Bearings
- Painting
- Heat Straightening
- Beam Saddles

22.7.2.1 Fatigue Damage Countermeasures

Fatigue damage entails the formation of cracks in base metal or welds. If not repaired in a timely manner, fatigue cracks can lead to brittle fractures. The type of repair and its timing are dependent upon many factors including:

- reason for the cracking (e.g., poor detailing, heavier than anticipated truck traffic, poor notch-toughness, load induced or distortion induced, constraint);
- location of the crack (e.g., cross frame, stiffener, weld, heat-affected zone, main member);
- depth, length and geometry of the crack; and
- redundancy.

The following options are available to correct fatigue damage:

1. Grinding. If the penetration of surface cracks is small, the cracked material can be removed by selective grinding without substantial loss in structural material. Grinding should preferably be performed parallel to the principal tensile stresses, and surface striations should carefully be removed because they may initiate future cracking.

   The most common application of grinding is to the toe of the fillet weld at the end of cover plates to meet fatigue requirements. Grinding can also be used when girders are nicked while removing old decks.

2. Drilled Holes. At the sharp tip of a crack, the tensile stress exceeds the ultimate strength of the metal, causing rapid progression if the crack size attains a critical level. The purpose of drilled holes is to blunt the sharp crack tip. The location of the tip should

therefore be established by one of the crack detection methods provided in Section 26.3.2. Missing the tip renders this process useless. Drilling holes at crack tips may be a final solution for distortion-induced fatigue cracks, but it is not a final solution for load-induced fatigue cracks.

Sections must be checked to ensure that the reduced member capacity due to the crack and the drilled hole is still adequate, but this is typically not a critical concern. The mitigation of the stress concentration at the tip is much more critical than the loss of net section. As such, the hole should be as large as can be tolerated in terms of net section. Drill bits of 13/16-in and 1-1/16-in diameter are common due to their use for fabricating bolt holes. Larger diameter holes should be avoided to reduce loss of cross-sectional area.

If holes overlap, the sides of the slots should be ground smooth to remove any projecting surfaces. This will create one oblong hole.

3. Bolted Splices. Where rivets or bolts in a connection are replaced, or where a new connection is made as part of the rehabilitation effort, the strength of the connection should not be less than 75% of the capacity or the average of the resistance of and the factored force effect in the adjoining components. Almost exclusively, the connections are made with high-strength bolts (ASTM A325).

   This method can also be used to span a cracked flange or web, provided that such connection is designed to replace the tension part of the element or component.

   The preferred method of tightening bolts is by direct tension indicators or twist-off bolts. Regardless of the method used, all bolts in the group are first brought into a "snug-tight" condition and, then, the bolts are individually tightened to the specified tension.

4. Peening. Peening is an inelastic reshaping of the steel at the surface location of cracks, or of potential cracks, by using a mechanical hammer. This procedure not only smoothes and shapes the transition between weld and parent metal, it also introduces compressive residual stresses that inhibit the cracking. Peening is most commonly used at the ends of cover plates to reduce fatigue potential.

   A new computer-controlled peening process using high-speed peening called ultrasonic peening has been introduced, which removes the dependency of the quality of mechanical-hammer peening on the operator's proficiency. This process promises weld enhancement for unavoidable poor fatigue resistance details such as terminations of longitudinal stiffeners.

5. Ultrasonic Impact Treatment (UIT). This process is a relatively new and promising post-weld surface treatment technique. It involves the deformation of the weld toe by a mechanical hammering at a frequency of around 200Hz superposed by ultrasonic treatment at a frequency of 27 kHz. The objective of the treatment is to introduce beneficial compressive residual stresses at the weld toe by plastic deformation of the surface and to reduce stress concentration by smoothing the weld toe profile.

22.7.2.2 Section Losses

The following options are available to correct section losses by adding doubler plates:

1. Welding Doubler Plates. It is common practice to use welding for shop fabrication of steel members and for welding pieces in preparation for rehabilitation work. Field welding is often difficult to perform properly in high-stressed areas, and individuals with the necessary skill and physical ability are required. The proper inspection of field welds is equally difficult. A shop weld is preferred to a field weld. All welding, whether in the shop or in the field, must be performed by a certified welder using approved welding processes.

   Field welding should only be allowed on secondary members, for temporary repairs, or in areas where analysis shows minimal fatigue stress potentials.

2. Bolting Doubler Plates. Bolting doubler plates over a corroded section (that has been cleaned and painted to prevent further section loss) is a more long-term reliable solution.

22.7.2.3 Strengthening

The following options are available to strengthen a steel superstructure:

1. Add Cover Plates. If the deck is deteriorated and removed, adding cover plates to strengthen a girder becomes a viable alternative. The *LRFD Specifications* places fully welded cover plates into Fatigue Detail Categories E or E′, depending upon thickness, at the ends of the cover plates. These are the lowest fatigue categorizations and, therefore, the process may be counterproductive from the design perspective. If bolts designed in accordance with LRFD Article 6.10.12.2.3 are used at the end of the cover plates, Fatigue Detail Category B is applicable. Because this requires the presence of drilling equipment and work platforms, consider a fully bolted cover plate construction.

2. Introduce Composite Action. Introducing composite action between the deck and the supporting girders is a cost-effective method to increase the strength of the superstructure. The *LRFD Specifications* mandates the use of composite action wherever current technology permits. Composite action can be achieved by welded studs or high-strength bolts. Shear connectors shall be designed in accordance with LRFD Article 6.10.10.

   Composite action considerably improves the strength of the upper flange in positive moment areas, but its beneficial effect on the girder as a whole is only marginal. The combination of composite action in conjunction with selective cover plating of the lower flange is the most effective way of girder strengthening.

   Introducing composite action near joints prevents the deck from separating from the girders, thus increasing the service life of the deck. This should be performed on each bridge that will have its deck removed for other reasons.

3. Add New Girders. If the deck is removed, a new set of girders added to the existing bridge is one alternative to strengthen the superstructure. To ensure proper distribution of live load, the rigidity of the new girders should be close to that of the existing ones.

   The old girders may also need rehabilitation, in which case, it may be more feasible from a structural and economical standpoint to remove the girders. If the existing paint system contains heavy metals, it may make replacement more economically feasible. Using current deck designs and composite action, continuous girders with a large spacing should be explored as an alternative.

22.7.2.4 Bearings

The discussion in Section 22.6.2.3 also applies to steel superstructures. In addition, rock bearings and elastomeric bearings should not be mixed on the same pier or abutment because of difference in movement.

22.7.2.5 Painting

Technically, bridge painting is maintenance work and not rehabilitation work but, frequently, painting is considered in conjunction with rehabilitation work on steel structures. In general, bridge painting is not economical but, in some circumstances, it may be warranted on a specific project. When considering bridge painting options, three scenarios present themselves:

- full removal of existing paint and repainting,
- a complete recoat over the top of the existing paint (overcoat), or
- touch-up painting.

The most important factor with respect to painting bridges is that virtually all paint applied to bridges prior to 1977 contains heavy metals. To remove existing paint, the current state of practice is abrasive blast removal, full enclosure, and environmental and worker monitoring. The price for this work approaches, and at times exceeds, the cost of replacing the existing steel bridge members with new girders.

The paint industry has developed products that can be successfully applied over existing paints and marginally prepared surfaces. An overcoat may be an economic alternative to full removal and repainting where a uniform appearance for the structural members is desired at the conclusion of the rehabilitation. However, the problems associated with heavy metal-based paints are not solved, merely deferred until a subsequent rehabilitation or structure replacement. Touch-up painting neither provides a uniform appearance nor solves the long-term heavy metal problem. Touch-up painting may be appropriate in localized zones where corrosion could cause section loss.

Give careful consideration to the proper selection of paint for an overcoat. An improperly specified or improperly applied overcoat can cause failure of the original paint that was performing satisfactorily. Review the manufacturer’s literature on any paint’s service environment and recommended application environment. Proper surface preparation, application and field inspection is most of the challenge in applying paint.

22.7.2.6 Heat Straightening

This technique is restricted to hot-rolled steels. Steels deriving their strength from cold drawing or rolling tend to weaken when heated. The basic idea of heat straightening is that the steel, when heated to an appropriate temperature (usually cherry color), loses some of its elasticity and deforms plastically. This process rids the steel of built-up stresses. While at an elevated temperature, the steel can also be hot worked and forced into a desirable shape or straightness without loss of ductility. Special care should be exercised not to overheat the steel; accordingly, this technique should be implemented by those having experience with this process. Note also that the heating temporarily reduces the resistance of the structure. Measures such as vehicular restriction, temporary support, temporary post-tensioning, etc., may be applied as appropriate. Additional guidance on heat straightening can be found in *Heat-Straightening Repairs of Damaged Steel Bridges: A Technical Guide and Manual of Practice*, FHWA, 1998, and *Heat-Straightening Repair of Damaged Steel Bridge Girders: Fatigue and Fracture Performance*, NCHRP, 2008.

22.7.2.7 Beam Saddles

A beam or cap that has deteriorated from water, salt and corrosion or has been damaged by cracking in the bearing area such that the repair of the area would be inadequate or inadvisable, may be repaired using beam saddles. Consider the following:

- Evaluate the existing cap to determine whether it is capable of, or can be made capable of, supporting a loaded saddle.

- Design the saddle to support the maximum beam reactions. The designer must be sensitive to potential fatigue failure in the welding details. The dead load reactions from adjacent spans should be approximately equal, and the live load shear should be minimal to prevent rocking of the saddle assembly.

- Prefabricate the components of the saddle assembly and apply a paint system.

22.8 SUBSTRUCTURES

22.8.1 General

Chapter 18 provides a detailed discussion on the structural design of substructures for new bridges, and Chapter 17 applies to foundations. Many of the design and detailing practices provided in these Chapters also apply to the rehabilitation of the substructures of an existing bridge. Therefore, the designer should review Chapters 17 and 18 to determine its potential application to the bridge rehabilitation project.

22.8.2 Rehabilitation Techniques

Many of the rehabilitation measures previously described for bridge decks and superstructures can also be used to rehabilitate substructures. In addition, the following techniques may be appropriate and should be considered:

- Scour Mitigation
- External Pier Cap Post-Tensioning
- Micropile Underpinning
- Ground Anchorages
- Soil Stabilization

22.8.2.1 Scour Mitigation

The bridge designer will work with the Materials Division and Hydraulics Section to identify appropriate scour mitigation measures. Options that are often considered include tremie concrete encasement, grouting beneath undermined footings, and riprap or channel lining placement. Foundation load bearing requirements and impacts on the channel flow area must be addressed for any mitigation measures that are considered.

22.8.2.2 External Pier Cap Post-Tensioning

Inadequately reinforced concrete pier caps may require strengthening by external post-tensioning. The existing concrete in the cap must be evaluated to determine whether it will support the system. Tensioning strand or rods can be placed externally on the cap to add compression to the cap. Brackets, distribution plates and other components are needed to transfer the post-tensioning forces to the cap. If aesthetics are a concern, the cap can be widened with ducts placed internally for the post-tensioning.

Post-tensioning is usually symmetrical to the cap so that an eccentric force is not introduced. The designer must look at the stressing sequence to ensure that the cap is not overloaded eccentrically during post-tensioning operations.

22.8.2.3 Micropile Underpinning

Micropiles, also known as minipiles and pin piles, are small-diameter reinforced piles that are drilled and grouted to support structures. These piles may reach service loads up to 300 tons, can be installed to depths of approximately 200 ft, and usually use some type of steel bar or bars and/or steel casing pipe. The bars are grouted into the ground and/or the casing pipe is filled with grout. Although a conventional pile is generally quite large and requires heavy equipment and large staging areas for installation, micropiles can be used in applications where conventional piling is not convenient or possible, such as for underpinning or retrofitting existing bridges or structures. Micropiles have proven effective in many ground improvement applications by increasing the bearing capacity and reducing settlement, especially when strengthening the existing foundations.

22.8.2.4 Ground Anchorages

Ground anchors can be used to stabilize retaining walls and abutments that are experiencing lateral movement or rotation due to external earth pressures. They can also increase the footing resistance to uplift forces. Ground anchors consist of prestressing strand or rods grouted into a drilled hole and tensioned to a required force. The bridge designer determines the number of anchors and their location. The bridge designer must work closely with the geotechnical engineer to identify the forces acting on the structure, the length of anchor and the needed corrosion protection.

This repair technique changes the structure’s boundary conditions (e.g., a cantilever retaining wall to an anchored retaining wall) requiring an analysis and possible modification of the structure. Modifications are required to transfer ground anchor forces to the structure and may be required to redistribute external earth pressures.

22.8.2.5 Soil Stabilization

Soil stabilization, the chemical or mechanical treatment designed to increase or maintain the stability of a mass of soil, can improve the engineering properties of in-situ soils. Lime, fly ash or cement are typical chemical stabilization materials. Geotextiles, geogrids, compaction grouting and stone columns are examples of mechanical types of soil stabilization. Soil stabilization can be used as a corrective measure for settlement and for reducing liquefaction potential.

22.9 SEISMIC RETROFIT

22.9.1 Seismic Evaluation

The ability to predict the forces developed by earthquake-induced motion is limited by the complexity of predicting the acceleration and displacements of the underlying earth material and the response of the structure. The motion can generally be described as independent rotation, in any direction, of each bridge abutment or pier, in or out of phase with each other, combined with sudden vertical displacements. Ground between piers can distort elastically and in some cases rupture or liquefy.

The bridge failures induced by the motions of the abutments and piers stem from two major inadequacies of many existing bridge designs — the lack of adequate connections between segments of a bridge and inadequately reinforced columns. Other deficiencies include inadequately reinforced footings and pier caps.

Fortunately, tying the segments of an existing bridge together is an effective means of preventing the most prevalent failure mode — spans falling off the bearings, abutments or piers. It is also the least expensive of the inadequacies to correct. Bridges with single-column piers are particularly vulnerable where segments are not connected.

Columns inadequately reinforced, because of too few and improperly detailed ties and spirals or short-lapped splices, generally do not sufficiently confine the concrete. This is particularly critical in single-column piers.

Determining the retrofit technique to use involves these considerations:

- mode of failure anticipated,
- influence on other parts of the bridge under seismic and normal loadings,
- interference with traffic flow, and
- cost of fabrication and installation.

Some retrofit procedures are designed to correct inadequacies of bridges related to earthquake resistance. The procedures may be categorized by the function the retrofit serves, including:

- restraining uplift;
- restraining longitudinal motion;
- restraining hinges;
- widening bearing seats;
- strengthening columns, caps and/or footings;
- restraining transverse motion; and
- bearing upgrade.

NDOT policy is to completely upgrade any bridges identified in need of seismic retrofitting. Bridges that are selected for seismic retrofitting shall be investigated for the same basic criteria that are required for all new bridges, including minimum support length and minimum bearing force demands. Bridge failures have occurred at relatively low levels of ground motion. Specific details for seismic retrofitting may be found in the *Seismic Retrofitting Manual for Highway Structures: Part 1 - Bridges,* FHWA, 2006.

22.9.2 Application

For the rehabilitation of existing Statewide bridges, the designer is required to perform a seismic evaluation of the structure when major rehabilitation (i.e., deck replacement or superstructure widening) is anticipated.

Minor seismic retrofit will usually be limited to seismic restrainers, dynamic isolation bearings and widening of girder seats. Usually, it will be limited to work at or above the girder seats. Major seismic retrofit includes such items as strengthening columns, piers, pier caps, etc. It will generally include work below the level of the girder seats and may include work requiring cofferdams and traffic control.

22.9.3 Seismic Risk Ratings

Bridges designed to pre-1983 seismic design provisions are considered moderately to highly susceptible to sustain significant damage during an earthquake. A seismic prioritization study was conducted to evaluate the approximately 800 State bridges meeting the pre-1983 criteria. Based on the prioritization study, seismic risk ratings have been established for these bridges.

Two primary factors are determined in computing the structure risk rating — importance and seismic vulnerability. The importance factor considers various subfactors including route type, traffic volume, detour length, utility lines, defense route and railroad. The seismic vulnerability factor considers site seismicity and critical detailing (e.g., inadequate seat width, deficient column confinement).

Structures with risk values of 140 or higher are considered likely candidates for retrofitting. Further study/analysis must be performed to determine the need for and extent of retrofitting.

Contact the NDOT Structures Division for seismic risk rating information.

22.9.4 Seismic Retrofit Techniques

The remainder of Section 22.9 presents a brief discussion on those seismic retrofit techniques that may be considered:

- Column Jacketing
- Modifying Seismic Response
- Widening Bearing Seats
- Restrainers and Ties
- Bearing Replacement
- Seismic Isolation Bearings
- Cross Frames and Diaphragms
- Footing Strengthening

22.9.4.1 Column Jacketing

Jacketing consists of adding confinement to columns by covering with a grout-filled steel shell, fiberglass wrap or carbon fiber wrap. The steel jacket consists of structural steel welded over the column and grouted. The fiberglass and carbon fiber wraps are glued to the column in multiple layers. These are proprietary products. Non-circular columns can be retrofitted by jacketing, but the increased rigidity must be evaluated. A circular steel casing may be placed around the non-circular column and grouted.

Jacketing is generally located only at the points of potential column hinge formations. However, if more than half the total height of the column requires a jacket, consider extending the jacket full height for improved aesthetics. Jacketing increases column rigidity, amplifying global seismic forces and attracting more load to the column. This increased rigidity must be evaluated.

22.9.4.2 Modifying Seismic Response

The following techniques may be used to modify the seismic response of a bridge:

1. Flexural Reinforcement. Because of conservative provisions, concrete columns have often been both over-designed and over-reinforced in the past. Over-reinforcement means that the flexural steel is not expected to yield during the design event, resulting in both higher compressive and shear forces on the concrete. If other design criteria permit, some of the flexural steel may be cut to induce yield. If circumstances warrant, the flexural reinforcement may be increased. The vertical bars are located in a concrete jacket that is shear connected to the column by means of drilled and grouted dowels. This also increases the rigidity of the column, potentially rendering it counterproductive.

2. Infill Shear Wall. A concrete shear wall can be added between the individual columns of a frame pier. If the existing footing is not continuous, it should be made so. The wall should be connected to the columns by means of drilled and grouted dowels. This method substantially changes the seismic-response characteristics of the structure, requiring a complete reanalysis. The more rigid infill wall will attract more load, and this increase must be considered in the design.

22.9.4.3 Widening Bearing Seats

Seat width extensions allow larger relative displacements to occur between the superstructure and substructure before support is lost and the span collapses. The extensions are likely to be exposed to large impact forces due to the dropping span; therefore, they should either be directly supported by a footing or be adequately anchored to the abutment or pier cap. Provisions in the *LRFD Specifications*, relative to the design of seat widths, should be followed as practical.

22.9.4.4 Restrainers and Ties

In general, restrainers are add-on structural devices that do not participate in resisting other than seismic force effects. Typically, these components are made of steel, they should be designed to remain elastic during seismic action, and special care should be exercised to protect them against corrosion.

There are three types of restrainers — longitudinal, transverse and vertical. The purpose of the two former ones is to prevent unseating the superstructure. The objective of the third one is to preclude secondary dynamic (impact) forces that may result from the vertical separation of the superstructure.

The restraint devices should be compatible with the geometry, strength and detailing of the existing structure. The designer may need to create new devices if those reported in the literature are not suitable.

Ties are restrainers that connect only components of the superstructure together. They are activated only by seismic excitation.

22.9.4.5 Bearing Replacement

Bearings not adequately designed for seismic movements and damaged or malfunctioning bearings can fail during an earthquake. In addition, steel rocker and roller bearings may perform poorly in seismic events. One option is to replace these bearings with steel-reinforced elastomeric bearings. To maintain the existing girder elevation, either a steel assembly is inserted between the girder and the elastomeric bearing, or the elastomeric bearing is seated on a new concrete pedestal. Existing anchor bolts may assist in resisting shear between the pedestal and the pier. In both cases, the girder should be positively connected to the substructure by bolts, either directly or indirectly.

22.9.4.6 Seismic Isolation Bearings

There is a broad variety of patented seismic isolation bearings that are commercially available. They permit either rotation or translation or both. They have special characteristics by which the dynamic response of the bridge is altered, and some of the seismic energy is dissipated. The primary change in structural response is a substantial increase in the period of the structure's fundamental mode of vibration. The *LRFD Specifications* determines the equivalent lateral static design force as a function of this period. The devices are designed to perform elastically in response to normal service conditions and loads.

Accordingly, seismic isolation bearings normally contain an elastomeric element. The inelastic element is usually either a lead core or a viscous liquid damper whose resistance is a function of the velocity of load application. They are effective for seismic loads due to their high velocity. The liquid dampers are prone to leakage, thus requiring back-up safety devices.

The Chief Structures Engineer must approve the use of seismic isolation bearings. Their use is discussed in the AASHTO *Guide Specifications for Seismic Isolation Design* and the FHWA *Seismic Retrofitting Manual for Highway Structures*, FHWA, 2006.

22.9.4.7 Cross Frames and Diaphragms

The cross frames between steel girders and diaphragms between concrete girders at points of support can fail during a seismic event. Their capacity must be checked to ensure seismic forces from the superstructure are transmitted to the bearings and into the substructure.

22.9.4.8 Footing Strengthening

During a seismic event, the footing can fail before the column flexural capacity is reached. This retrofit approach can include the enlargement of the plan-view dimensions and the thickness of the footing, addition of top steel, and placement of dowels to connect the existing and new concrete.

Spread footings can also tilt, and one side may lift from the supporting soil during a seismic event causing a soil failure on the other side. Ground anchors or other hold-down devices can be used to keep the footing in contact with the soil.

22.10 BRIDGE WIDENING

22.10.1 General Approach

A bridge widening can present a multitude of challenges during the planning and design stages, during construction and throughout its service life. Special attention is required in both the overall design and detailing of the widening to minimize construction and maintenance problems.

This Section presents NDOT guidelines for widening existing bridges. The following briefly summarizes the basic objectives in bridge widening:

- Match the structural components of the existing structure, including splice locations.
- Match the existing bearing types in terms of fixity.
- Do not perpetuate fatigue-prone details.
- Always evaluate the need to replace the bearings and joints in the existing structure.
- Evaluate the load-carrying capacity of the existing structure.
- Evaluate the seismic resistance of the existing and widened structure. Incorporate retrofit measures if appropriate.
- Use the same structure frame on the widened portion as on the existing bridge.
- Match the flexibility of the existing and new superstructures.
- Use epoxy-coated steel reinforcing bars in a deck widening for all bridges, except those in Clark County.
- Flared columns may be considered for use on bridge widenings with the written approval of the Chief Structures Engineer.

22.10.2 Existing Structures

22.10.2.1 General

An existing structure may have been originally designed for either live loads or seismic loads less than those currently used for new bridges. If such a structure becomes a candidate for widening, consult the data available in the Nevada Bridge Inventory on the condition of the existing structure. A load rating for the existing bridge must be made to quantify the capacity of the existing bridge. Based on this information, the designer will determine whether the existing structure should be strengthened to increase load-carrying capacity. For the evaluation, the following should be considered, if appropriate:

- cost of strengthening existing structure;
- physical condition, operating characteristics and remaining service life of the structure;
- seismic resistance of structure;
- other site-specific conditions;
- only structure on route that restricts permit loading;

- width of widening; and
- traffic accommodation during construction.

22.10.2.2 AASHTO Standards

It is not normally warranted to modify the existing structure solely because it was designed to AASHTO Specifications prior to the adoption of the *LRFD Bridge Design Specifications* and its latest interim changes.

When preparing plans to modify existing structures, it is often necessary to know the live load and stress criteria used in the original design. Since approximately 1927, with few exceptions, structures on the Nevada highway system have been designed for loads and stresses specified by AASHTO.

The designer should be aware of the historical perspective of design criteria, such as live loads, allowable stresses, etc., when analyzing a rehabilitated structure. For accurate and complete information on specific structures, see the General Notes on as-built plans, old standard drawings and special provisions, and the appropriate editions of the AASHTO Specifications.

22.10.2.3 Rolled Steel Beams

Throughout the years, modifications to rolled steel beam sections have occurred. Designers should refer to the construction-year AISC steel tables for rolled beam properties and other data.

22.10.2.4 Survey of Existing Bridge

A survey of the existing bridge should be performed as one of the first work activities on a widening project. The as-built plans may not accurately identify the actual bridge geometrics and sizes of structural elements. This is especially relevant to cast-in-place bridges. The bridge designer should contact the Location Division for a survey of the existing bridge. Depending upon the complexity of the existing bridge, the Location Division may elect to use the LiDAR 3-D laser scanning system to perform the survey. The bridge designer should include a note on the contract plans to require the contractor to perform a survey of the existing bridge and verify controlling field dimensions prior to initiating construction.

22.10.2.5 Materials

For material properties of older structures, check the General Notes on the existing bridge plans, if they exist, or plans of comparable bridges of the day. Also, the NBI can be referenced for historical properties of materials.

Sometimes, the grade of reinforcing steel is indicated as “intermediate grade”; this terminology means Grade 40.

Up to approximately 1960, ASTM A7 was the primary structural steel used in bridge construction. The yield and tensile strengths of A7 may be taken as 33 ksi and 66 ksi, respectively.

22.10.3 Girder Type Selection

In selecting the type of girder for a structure widening, the widened portion of the structure should be a construction type and material type consistent with the existing structure. See Section 11.5 for guidance regarding appropriate superstructure types.

22.10.4 Longitudinal Deck Joints

Past performance indicates that longitudinal joints in bridge decks between a bridge widening and the existing bridge have a high likelihood of becoming a source of bridge maintenance problems. Therefore, as a general policy, no longitudinal deck joints should be employed. Section 16.2.7 provides guidance on where longitudinal deck joints may be necessary.

Experience has shown that a positive attachment of the widened and original decks provides a better riding deck, usually presents a better appearance and reduces maintenance problems. A positive attachment of the old and the new decks shall preferably be made.

The following recommendations should be considered when widening an existing girder-and-deck-type structure:

1. Structures with large overhangs should be widened by removing the concrete from the overhang to a width sufficient to develop adequate bond length for lapping the original transverse deck reinforcing to that of the widening.

2. Structures with small overhangs, where removal of the overhang will not provide sufficient bond length, should be either doweled to the widening or have transverse reinforcing steel exposed and extended by mechanical lap splice.

3. Structures with no overhangs should be attached by doweling the existing structure to the widening. Notching the existing exterior girder as a means of support has proven to be unsatisfactory and should be avoided.

4. Longitudinal construction joints should preferably not be located over the girder flanges.

5. Removal of the deck past the outside girder line will result in a cantilever slab condition. The design must ensure that the deck can resist the loadings anticipated during construction.

6. Longitudinal construction joints shall preferably be aligned with the permanent lane lines. These joints tend to be more visible than the pavement markings during adverse weather conditions.

22.10.5 Dead Load Deflection

It is recommended that, where the dead load deflection (combined with the post-tensioning deflection where post-tensioning is included) exceeds ¼ in, the widening should be allowed to deflect and a closure pour used to complete the attachment to the existing structure. A closure pour serves two useful purposes: It defers final connection to the existing structure until after the deflection from the deck slab weight has occurred; and it provides the width needed to make a smooth transition between differences in final grades that result from design or construction imperfections. The bridge plans should include a note indicating the required waiting period, if any, between deck concrete and closure concrete placement.

For the effects of dead load deflection, two groups of superstructure types can be distinguished — precast concrete girder or steel girder construction, where the largest percentage of deflection occurs when the deck concrete is placed and, for cast-in-place construction, where the deflection occurs after the falsework is released.

In the first group, dead load deflection after placing the deck is usually insignificant but, in cast-in-place structures, the dead load deflection continues for a lengthy time after the falsework is released. In conventionally reinforced concrete structures, approximately ⅔ to ¾ of the total deflection occurs over a four-year period after the falsework is released due to shrinkage and creep. A theoretical analysis of differential deflection that occurs between the new and existing structures after closure will usually demonstrate that it is difficult to design for this condition. Past performance indicates, however, that theoretical overstress in the connection reinforcing has not resulted in maintenance problems, and it is generally assumed that some of the additional load is distributed to the original structure with no difficulty or its effects are dissipated by inelastic relaxation. Good engineering practice dictates that the closure width should relate to the amount of dead load deflection that is expected to occur after the closure is placed. A minimum closure width of 30 in is recommended. A post-tensioned box girder or precast girders made continuous by post-tensioning is typically used for the widening, instead of a conventionally reinforced box girder, to minimize the differential deflection that occurs between the existing bridge and the widening.

22.10.6 Vehicular Vibration During Construction

All structures deflect when subjected to live loading, and many bridge widenings are constructed with traffic on the existing structure. Fresh concrete in the deck is subjected to deflections and vibrations caused by traffic. Studies such as NCHRP 86 *Effects of Traffic-Induced Vibrations on Bridge-Deck Repairs* have shown that:

- Good-quality reinforced concrete is not adversely affected by jarring and vibrations of low frequency and amplitude during the period of setting and early strength development.
- Traffic-induced vibrations do not cause relative movement between fresh concrete and embedded reinforcement.
- Investigations of the condition of widened bridges have shown the performance of attached widenings, with and without the use of a closure pour placed under traffic, to be satisfactory.

22.10.7 Substructures/Foundations

Foundation capacities of existing structures should be investigated if additional loads will be imposed on them by the widening. It is possible for newly constructed footings under a widened portion of a structure to settle. The new substructure could be tied to the existing substructure to reduce the potential for differential foundation settlements, provided that this does not adversely affect the existing substructure. If the new substructure is not tied to the existing substructure, suitable provisions should be made to prevent possible damage where such movements are anticipated. The bridge designer must work with the Materials Division to assess the compatibility of new and existing foundations and the potential for differential settlement.

When a bridge will be widened, a Foundation Report is required for the widening. Coordinate this with the Materials Division.

Chapter 23

MISCELLANEOUS STRUCTURAL ELEMENTS

NDOT STRUCTURES MANUAL

September 2008

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Chapter 23
MISCELLANEOUS STRUCTURAL ELEMENTS

In general, the Structures Division is responsible for all structural design for NDOT projects. Although the vast majority of the Division's work is for highway bridges, the transportation system includes a wide variety of other structural elements. Chapter 23 presents NDOT policies, practices and criteria for the structural design of these miscellaneous structural elements.

23.1 EARTH RETAINING SYSTEMS

23.1.1 General

NDOT uses earth retaining systems to provide lateral support for a variety of applications:

- cuts in slopes for roadway alignments;
- roadway widening where right-of-way is limited;
- grade separations;
- proximate live-load surcharge from buildings, highways, etc., that must remain in place;
- stabilization of slopes where instabilities have occurred;
- protection of environmentally sensitive areas; and
- excavation support.

Earth retaining systems are classified according to their construction method and the mechanism used to develop lateral support:

1. Construction Method. This may be either a "fill-wall" construction or "cut-wall" construction. Fill-wall construction is where the wall is constructed from the base of the wall to the top (i.e., "bottom-up" construction). Cut-wall construction is where the wall is constructed from the top of the wall to the base (i.e., "top-down" construction). Note that the "cut" and "fill" designations refer to how the wall is constructed, not the nature of the earthwork (i.e., cut or fill) associated with the project.

2. Lateral Load Support. The basic mechanism of lateral load support may be either "externally stabilized" or "internally stabilized." Externally stabilized wall systems use an external structural wall, against which stabilizing forces are mobilized. Internally stabilized wall systems employ reinforcement that extends within and beyond the potential failure mass.

Note that, because of their special nature and frequent use by NDOT, Section 23.2 discusses MSE walls separately from other earth retaining systems, which are discussed in Section 23.1.

23.1.2 Responsibilities

The type selection for an earth retaining system is a collaborative effort between the Structures Division and Geotechnical Section. The following identifies the basic responsibilities of the respective NDOT Units for the design of earth retaining systems, except MSE walls. See Section 23.2 for a discussion on the division of responsibilities for MSE walls.

23.1.2.1 Structures Division

The following summarizes the role of the Structures Division in the design of earth retaining systems:

1. <u>Design</u>. For CIP concrete cantilever walls, non-gravity cantilever (sheetpile) walls and anchored walls, the Structures Division will perform the internal stability design for the wall (e.g., wall dimensions and reinforcing configurations). For these walls, the Structures Division also performs the overturning, sliding and bearing checks using the geotechnical parameters provided by the Geotechnical Section. For soil nail and tie-back anchor walls, the Structures Division will design the reinforcing for the structural facing of the wall.

2. <u>Detailing</u>. The Structures Division provides all the construction details for the earth retaining system, including:

   - Plan views are provided to indicate the layout of the walls. The station and offset to the wall layout line (usually the front face) is provided at all locations needed for locating the wall.

   - Elevation views are provided to show the length and design height of wall segments, and top and bottom elevations of the wall. Top-of-wall elevations are provided at intervals necessary to build the walls. Provide elevations every 25 ft when the top of wall is not on a straight line. Footings are almost always level with the bottom and top of footing elevation shown for each step.

   - Typical sections are provided to show all additional information on the wall. This can include the dimensions of the footing and wall, approximate original ground line, finished ground line at the bottom and top of wall, bench at bottom of wall, slopes at the bottom and top of wall, drainage requirements and reinforcing steel.

   - Special details are used to show specialty items specific to a wall type and location, treatments at the top and bottom of wall and drainage details.

23.1.2.2 Geotechnical Section

For permanent earth retaining systems, the Geotechnical Section:

- performs the geotechnical investigations;

- provides recommendations for wall type;

- recommends the allowable soil bearing and lateral earth design coefficients for gravity, surcharge and seismic loading;

- performs the global and external stability checks;

- determines if there is a need for special drainage features due to the selected wall type and/or site conditions; and

- determines the size and spacing of soil nails and tie-back anchors.

The Geotechnical Section will also provide the following information to the Structures Division:

- earth pressure coefficients (k_a, k_o, k_p) and an estimate of the amount of deformation to develop the active and passive earth pressures and any recommendations on factors of safety;
- unit weight of the backfill material;
- allowable interface friction between cast-in-place concrete footing and soil;
- allowable bearing capacity;
- expected settlement;
- requirements for drainage control;
- testing requirements for anchored and soil nail walls; and
- special construction requirements for all walls.

23.1.2.3 Roadway Design Division

The Roadway Design Division identifies the need for an earth retaining system and provides the Geotechnical Section and the Structures Division with the alignment file and/or cross sections.

23.1.2.4 Other NDOT Units

Depending on the site, other NDOT Units may participate in the design and selection process for earth retaining systems. These include:

- Hydraulics Section, if the wall is located near flowing water or if it could be inundated during floods and the scour potential at the base of the wall.
- Environmental Services Division, if the wall will be located in or adjacent to wetlands or other environmentally sensitive areas or if the wall requires excavations in contaminated soils.

23.1.3 <u>References</u>

For further guidance on earth retaining systems, see:

- NDOT *Geotechnical Policy and Procedures Manual*
- *Foundations and Earth Structures*, Department of the Navy, Naval Facilities Engineering, NAVFAC DM 7.2, May 1982
- *Geotechnical Engineering Circular No. 2 – Earth Retaining Systems*, FHWA-SA-96-038
- Geotechnical Engineering Circular No. 4 – Ground Anchors and Anchored Systems, FHWA-IF-99-015

- *Geotechnical Engineering Circular No. 7 – Soil Nail Walls*, FHWA-IF-03-017
- *Training Course in Earth Retaining Structures*, FHWA-NHI-132036
- *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines*, FHWA-SA-96-071

23.1.4 <u>Types of Earth Retaining Systems</u>

23.1.4.1 Fill Walls

23.1.4.1.1 *MSE Walls*

MSE walls are constructed using reinforced layers of earth fill with non-extensible (metallic) reinforcing. Extensible (polymeric) reinforcing may be used when approved by the Chief Structures Engineer. The facing for the walls can be concrete panels, geotextile fabrics or exposed welded wire. The heights of these walls can extend to over 100 ft. Advantages of MSE walls include:

- They tolerate larger settlements than a CIP concrete cantilever wall.
- They are relatively fast to build.
- They are relatively low in cost.

See Section 23.2 for detailed information on MSE walls.

23.1.4.1.2 *CIP Concrete Cantilever Walls*

CIP concrete cantilever walls are best-suited for sites characterized by good bearing material and small long-term settlement. In soft soils or when settlement may be a problem, the semi-gravity walls can be pile supported. This adds to the cost, especially relative to a MSE wall. However, for short wall lengths, the CIP concrete cantilever wall may be the most cost-effective selection.

An important advantage of these walls is that they do not require special construction equipment, wall components or specialty contractors. They can be up to 30 ft in height, although most are less than 20 ft in height. The footing width for these walls is normally ½ to ⅔ the wall height.

CIP concrete cantilever walls can be used in cut slope locations. In this case, the slope behind the face of the wall requires excavation to provide clearance for the construction of the wall footing. Typically, excavation slopes should not be steeper than 1.5H:1V, which can result in significant excavations in sloped areas. In this case, a shored excavation may be required, or alternative wall types may be more suitable.

23.1.4.1.3 *Prefabricated Modular Walls*

Prefabricated modular walls include concrete and metal bin walls and concrete crib walls. These types of walls may occasionally be advantageous. For example, because the components are prefabricated before delivery to the field, prefabricated modular walls may be desirable where the time to build the wall is limited.

23.1.4.2 Cut Walls

23.1.4.2.1 *Soldier Pile Walls*

Soldier pile walls involve installing H-piles every 8 ft to 10 ft and spanning the space between the H-piles with lagging. The H-piles are usually installed by grouting the H-pile into a drilled hole; however, they can also be installed by driving. The advantage of installing the H-pile by drilling is that vibrations, and the potential for driving refusal, are usually avoided. The depth of the soldier pile is similar to the sheetpile wall; i.e., approximately two times the exposed height. Lagging can be either timber or concrete panels.

For most soldier pile walls, a concrete facing is cast in front of the soldier piles and lagging after the wall is at full height. Various architectural finishes can be used for the facing.

23.1.4.2.2 *Anchored Walls*

Ground-anchored wall systems (often called tie-back walls) typically consist of tensioned ground anchors connected to a concrete wall facing. Ground anchors may also be used to construct soldier pile walls of a taller height. Ground anchors consist of a high-strength steel bar or prestressing strand that is grouted into an inclined borehole and then tensioned to provide a reaction force at the wall face. These anchors are typically located at 8-ft to 10-ft horizontal and vertical spacing, depending on the required anchor capacity. Each anchor is proof tested to confirm its capacity.

Specialized equipment is required to install and test the anchors, resulting in a higher cost relative to conventional walls. An important consideration for this wall type can be the subsurface easement requirements for the anchoring system. The upper row of anchors can extend a distance equal to the wall height plus up to 40 ft behind the face of the wall.

23.1.4.2.3 *Soil Nail Walls*

A soil nail wall uses top-down construction. The typical construction methodology includes:

- a vertical cut of approximately 4 ft;
- drill, insert and grout soil nails;
- shotcrete exposed cut surface;
- repeat operation until total height of wall is complete; and
- for permanent applications, a reinforced concrete wall is cast over the entire surface.

A soil nail wall involves grouting large diameter rebar (e.g., #10 or larger) or strand into the soil at 4-ft to 6-ft spacing vertically and horizontally. The length of the rebar or strand will typically range from 0.7 times the wall height to 1.0 times the wall height, or more.

Specialty contractors are required when constructing this wall type. Soil nail walls can be difficult to construct in certain soil and groundwater conditions. For example, where seeps occur within the wall profile or in relatively clean sands and gravels, the soil may not stand at an exposed height for a sufficient time to install nails and apply shotcrete.

23.1.4.2.4 *Nongravity Cantilever (Sheet Pile) Walls*

Sheet pile walls are normally driven or vibrated into the ground with a pile driving hammer and are most suitable at sites where driving conditions are amenable to pile driving. Therefore, part of the design process requires performing a driveability analysis. Sites with shallow rock or consisting of significant amounts of cobbles and boulders are not suitable for sheet pile driving.

Generally, the sheet pile must be driven to a depth of 2 times the exposed height to meet stability requirements. Most sheet pile walls are 10 ft to 15 ft or less in height. Although higher walls are possible, the structural design and installation requirements increase significantly. Taller sheet pile walls are possible, but require ground anchors that are typically attached to a horizontal whaler beam installed across the face of the sheet piles.

23.2 MECHANICALLY-STABILIZED EARTH (MSE) WALLS

23.2.1 External Stability and Internal Stability

The Geotechnical Engineer is responsible for the external stability calculation. The approved wall suppliers, shown in the NDOT Qualified Products List (QPL), are responsible for the internal stability of the wall.

The external stability calculation should include a check for sliding, overturning, rotational failure and bearing pressure. The wall geometry (including the width of reinforcement and height) will be established based on these items for each height of wall. If the wall supplier must increase the reinforcement width or height of the backfill due to internal stability requirements, the contractor is not paid for quantity increases. Increases over that required for external stability must be verified by the Geotechnical Engineer to ensure that the increase is justified.

23.2.2 Spread Footings

Spread footings supported by MSE walls pose a special problem during construction. The *Standard Specifications* are not specific on how construction of the wall will occur when a spread footing will be supported. The *Specifications* require that the mechanically stabilized earth backfill be compacted to 95% of maximum density. It is almost impossible, however, to compact the material adjacent to the wall panel, particularly with bar mat systems. Therefore, a note on the plans or in the Special Provisions is needed. The following statement is suggested:

> *Special consideration shall be given the mechanically stabilized earth backfill placed adjacent to the back face of wall panel. The methods of placement and the materials used to meet the minimum compaction density specified in the Standard Specifications shall be approved by the Engineer.*

NDOT does not allow the placement of spread footings for bridges on embankments retained by MSE walls.

23.2.3 Piles Within MSE Walls

Piles placed within the mechanically stabilized earth backfill require special consideration. The piles must be placed prior to the construction of the wall. As the wall is constructed, the subsoils beneath the wall and the MSE wall itself may compress. The piles, however, are rigid. The compression of the soils will induce a load into the piles due to friction. Depending on site materials, these downdrag forces can be substantial. To reduce the friction on the piles and to mitigate the downdrag forces, a material such as “Yellowjacket” sleeves can be placed on the plies, or a slightly larger corrugated pipe can be placed over the pile prior to backfilling.

The soil reinforcement must be modified when piles are located within the wall. The soil reinforcement cannot be bent around the piles; they must remain linear to develop their strength. Also, the soil reinforcement cannot be attached to the piles. A skew of up to 15° from a line perpendicular to the wall face may be allowed provided that the design accounts for this.

Bar mats can be cut and skewed, but they must conform to the following:

- Do not allow single longitudinal wires.
- Bar mats develop their strength from the cross wires. At least two longitudinal wires are needed to make the cross wire effective.
- Cut segments must meet minimum pull-out capacity factors of safety. Testing of cut segments is required to show that their full strength is developed.

All cutting of reinforcement shall be done prior to the application of corrosion protection. Bridging frames for soil reinforcing may be required around the piling if cutting and skewing cannot resolve all conflicts. The bridging frames are designed to transfer all forces within the soil reinforcement and shall be corrosion protected. The bridging frame shall be designed by the wall supplier and verified by the bridge designer. Detail all bridging frames on the shop drawings.

23.2.4 Loads From Other Structures

MSE walls that support structures, such as soundwalls, must be designed for the lateral and vertical loads imposed on the MSE wall. These loads can be substantial. The magnitude of the force and where the force is applied on the MSE wall must be noted in the contract documents or a drawing provided.

23.2.5 Barrier Rails

MSE walls that incorporate a barrier rail at the top of wall require special attention. The top of MSE walls are not strong enough to resist traffic impacts. Traffic impacts must be transferred from the barrier rail into a reinforced concrete slab that is part of or located just below the roadway pavement. The concrete slab is sufficiently massive to keep vehicle impact forces from being transferred into the MSE wall. Size the concrete slab to resist sliding and overturning forces due to vehicle impacts, wind or seismic loading as appropriate.

23.2.6 Copings

All copings at the top of MSE walls shall be cast-in-place. The top of the walls generally project 1 ft to 2 ft above the top layer of soil reinforcement. The coping must be sufficiently large to hold together this unbraced section. Reinforcing steel from the top wall panels should extend into the coping.

23.2.7 Shop Drawings

The wall supplier prepares the shop drawings and supportive calculations. NDOT or the design consultant approves the shop drawings. See Appendix 25A for an MSE wall checklist.

23.2.8 Responsibilities

23.2.8.1 Geotechnical Section

The Geotechnical Engineer will conduct global stability analyses and provide design recommendations including depth of embedment and width of reinforcement. The Geotechnical

Engineer will conduct external stability analyses with respect to sliding, overturning, slope stability and bearing pressure failures.

The Geotechnical Engineer uses publication No. FHWA-NHI-00-043 *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines* and/or the latest edition of the *LRFD Specifications* to design and determine the minimum reinforcement lengths of the wall. Typical information and details to be provided by the Geotechnical Section to the Structures Division include the following:

- depth of embedment of leveling pad;
- total height of wall from top of leveling pad to top of coping/bottom of barrier rail;
- minimum 4-ft wide bench in front of walls located on top of slopes;
- no steeper than 2H:1V slopes in front of or on top of walls;
- strength properties of soils supporting the wall (i.e., foundation soils), MSE backfill and retained backfill;
- magnitude of anticipated total and differential settlement;
- recommended waiting period prior to the construction of barrier rails, copings, concrete anchor slabs and roadway surface;
- the minimum required reinforcement lengths for the entire length of the wall; and
- surcharges.

23.2.8.2 Wall Supplier

The approved wall supplier will check the external stability with respect to sliding, overturning and bearing pressure to confirm NDOT's proposed minimum reinforcement lengths. The Geotechnical Engineer will determine the need for any changes indicated by the contractor's external stability analysis. NDOT is responsible for all costs associated with changes to the wall due to external stability.

The wall supplier is responsible for internal stability. The wall supplier is responsible for all costs associated with modifications to the overall wall geometry due to internal stability design or construction convenience.

23.2.8.3 Structures Division

The Structures Division will estimate the quantities and prepare the contract documents for MSE walls. See Chapter 5 for a discussion on MSE contract documents.

23.2.8.4 Roadway Design Division

The discussion in Section 23.1.2.3 also applies to MSE walls.

23.3 BURIED STRUCTURES

23.3.1 Reinforced Concrete Boxes

The structural design of reinforced concrete boxes is based on Section 12 of the *LRFD Specifications*. The Hydraulics Section is primarily responsible for these drainage appurtenances, and NDOT has developed standard designs that will apply in most cases. See the *NDOT Standard Plans*. Occasionally, the Hydraulics Section may request that the Structures Division verify the structural adequacy of a proposed or existing reinforced concrete box. A special design is required when:

- The box geometry or height of soil above the reinforced concrete box exceeds the values in the *Standard Plans*.
- Loads are imposed on the reinforced concrete box from other structures.
- The sequence of backfilling the sides of the reinforced concrete box will not allow equal loading.
- Special inlet, outlet, confluence or other special hydraulic structure is needed for which a standard does not exist.

23.3.2 Concrete Arch Culverts

The use of concrete arch culverts is only permitted when approved by the Chief Structures Engineer, the Chief Hydraulic Engineer and the applicable District Engineer. The following will apply:

1. The design must meet the requirements of Section 12 of the *LRFD Specifications*.
2. The arch must be doubly reinforced meeting AASHTO LRFD requirements for minimum reinforcing and service limit state (crack control) criteria.
3. The arch must be poured in-place using Class A (AA) modified or Class D (DA) modified concrete. Shotcrete construction is not permitted.
4. Design verification using the CANDE computer program is necessary.
5. Provide design verification for a potential future condition that would require excavation of backfill material along one edge of the arch.
6. The arch culvert must be constructed with a concrete invert.
7. The “saddle” area of multiple-cell arch structures must be constructed with a waterproofing system and a suitable drainage system to control ponding and saturation of backfill soils. Provide weepholes in exterior walls at 50-ft maximum spacing. All drains and weepholes shall be a minimum 3-in diameter.
8. A technical representative of the arch culvert supplier shall be on-site and shall supply the necessary technical assistance during the initial completion of major work activities including, but not limited to, the placement of reinforcing, forming, concrete placement, form removal, waterproofing and backfilling.

9. Upon completion of the culvert, provide certification from the Engineer of record that the arch was constructed with materials and procedures consistent with what was used for the arch design.

10. Only rigid formwork and falsework conforming to Section 502 of the *Standard Specifications* shall be used.

23.4 SOUND BARRIERS

The Environmental Services Division will determine the warrants for, locations of and minimum heights for sound barrier walls. The following discusses the basic NDOT requirements for conventional concrete (precast or cast-in-place) or masonry sound barrier walls. Proprietary wall systems and/or walls composed of other materials are subject to evaluation and approval under NDOT's Product Evaluation Program before they may be used. Refer to the NDOT "Soundwall System Evaluation Manual" for requirements for alternative wall systems.

23.4.1 Design Criteria

NDOT's practices for the structural design of sound barriers are:

- The design standards are the current editions of the AASHTO *Standard Specifications for Highway Bridges* and *Guide Specifications for Structural Design of Sound Barriers.*
- Wind pressure as determined by the AASHTO *Guide Specifications* using a minimum wind velocity of 80 mph and minimum B-2 exposure (20 psf minimum wind pressure).
- Seismic load as determined by the AASHTO *Guide Specifications* using the appropriate site acceleration coefficient but, in no case, shall the coefficient be less than 0.15.
- Ice load where applicable as determined by the AASHTO *Guide Specifications*.
- 10-kip vehicular impact load when the sound wall is constructed within the roadway clear zone (as defined by the current edition of the AASHTO *Roadside Design Guide*) and is not otherwise protected by a traffic barrier. Walls constructed within this clear zone should incorporate a suitable safety barrier.
- A Nevada-registered professional civil or structural engineer shall stamp the structural calculations and plan sheets.
- Foundation design should be based on the recommendations of a Nevada-registered civil or structural engineer. A copy of the geotechnical investigation report should be included with the project documents.
- Masonry design shall be based on a specified 28-day compressive strength ($f'm$) of 1.5 ksi without special inspection (use one-half reduction). Grout all cells and provide minimum reinforcing consisting of #4 bars at 16-in vertical and 20-in horizontal spacings.

23.4.2 Material Requirements

The following applies:

- Minimum concrete 28-day compressive strength shall be 4 ksi. Concrete used at locations subject to roadway salting shall be Class EA with a minimum 28-day compressive strength of 4.5 ksi for those elements located within the salt-affected area, defined as 12-in below or 48-in above the adjacent roadway surface.

- All reinforcing steel shall conform to AASHTO M31 Grade 60. Welded wire reinforcement may also be used; see Section 14.3.2. Reinforcing in concrete elements within the salt-affected area shall be epoxy coated.

- Structural steel shall conform to AASHTO M270 Grade 36 or 50 and shall be galvanized unless embedded in concrete.

- Masonry units shall conform to ASTM C90, Type I. Construct masonry walls level using running bond and provide cap blocks of 2-in minimum thickness.

23.5 SIGN, SIGNAL AND LIGHTING STRUCTURES

For these structures, NDOT has adopted the AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals.* The NDOT Traffic Engineering Section is primarily responsible for these structures, and NDOT has developed standard designs that will apply in most cases. Occasionally, the Structures Division will become involved in the design of structural supports for these roadside appurtenances.

The Structures Division is responsible for the design of all structures in the *NDOT Standard Plans* and for developing the designs for special installations. The Structures Division is also responsible for approving shop drawings on these structures. See Appendix 25A.

23.6 PEDESTRIAN/BICYCLE BRIDGES

23.6.1 Preliminary Design

The preliminary design for a pedestrian or bicycle bridge is intended to determine the most appropriate structure type and configuration for a given site. The Preliminary Design Report must be approved before initiating final design. The Report should address the following.

23.6.1.1 Geometrics

The geometrics of the bridge and the approach transitions shall meet the requirements of the AASHTO *Guide Specifications for Design of Pedestrian Bridges*. A minimum vertical clearance of 18′-0″ is required over NDOT highway facilities. Clearances over other facilities will be determined on a project-by-project basis. For pedestrian/bicycle bridges over waterways, the Hydraulics Section will determine the necessary hydraulic opening.

23.6.1.2 Structure Type

Generally accepted structure types for pedestrian and bicycle bridges include:

- cast-in-place, post-tensioned box girders;
- precast, prestressed concrete girders with a cast-in-place concrete deck;
- CIP concrete slabs;
- steel girders with a cast-in-place concrete deck; and
- tubular steel pony trusses.

Additional structure types may be considered as deemed appropriate for the given site. An evaluation of structure types must include a consideration of constructibility, aesthetics, use of falsework, construction costs, etc.

23.6.2 Final Design

The design shall conform to the latest edition of the AASHTO *Standard Specifications for Highway Bridges*, except as modified by the AASHTO *Guide Specifications for Design of Pedestrian Bridges* and as noted herein.

23.6.2.1 Seismic

AASHTO seismic provisions shall apply to pedestrian and bicycle bridges, as modified by the *NDOT Structures Manual*. See Section 13.3.

23.6.2.2 Fatigue

All tension members shall meet minimum V-notch toughness requirements for Zone 2.

23.6.2.3 Tubular Steel Pony Trusses

Tubular steel pony trusses are generally designed and fabricated by a company specializing in this type of pedestrian/bicycle bridge. A generic detail of the pony truss shall be shown in the contract documents. All applicable design standards also need to be shown in the documents.

The contract documents shall include provisions for the design, detailing and submittal of shop drawings. A Nevada Professional Engineer registered in civil or structural engineering must perform the design. Stamped calculations and design drawings must be submitted for review and approval prior to the start of fabrication. Shop drawings must also be submitted for review and approval. If the design drawings and shop drawings are combined in one submittal, they must also be stamped by the Registered Professional Engineer.

Chapter 24

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Chapter 25

CONSTRUCTION SUPPORT

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September 2008

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Chapter 25
CONSTRUCTION SUPPORT

The Structures Division is involved in many activities related to the construction of structural elements. These are all accomplished in coordination with NDOT's Resident Engineer responsible for the construction project. The majority of this Chapter addresses construction support activities related to the bridge designer. Chapter 26 addresses construction-related activities associated with the NDT Squad.

The bridge designer must be responsive to the field crew and give priority to contract-related questions, requests for information and submittal reviews. Delays caused by an untimely review can result in contractor claims for time and/or compensation.

The Resident Engineer has the responsibility and authority for construction contract administration. To maintain contract efficiency, the bridge designer will have direct communication with suppliers and fabricators to discuss technical issues. However, the bridge designer should not provide direct opinions or interpretation of contract documents. Supplier and fabricator questions on the direction needed on contract documents should be submitted as a Request for Information through the prime contractor to the Resident Engineer. This process is necessary to keep both the Resident Engineer and prime contractor informed of contract document issues and to allow these parties to negotiate a solution.

25.1 REVIEW OF SHOP DRAWINGS

25.1.1 General

Shop drawings (or working drawings) are intended to transform the general structural design, as presented in the contract documents, into detailed working drawings from which each individual structural member or component can be fabricated and/or constructed. If modifications to the structural design are necessary for fabrication and/or construction of structural components, NDOT must approve any changes before fabrication.

25.1.2 Responsibility

In general, it is the responsibility of NDOT to verify that the fabricator and constructor of structural elements is supplying the items as specified by the contract; it is the contractor's responsibility to ensure that all structural items are fabricated or constructed to the correct dimensions, to use the correct materials, and to conform to the contract documents.

25.1.3 Procedures

The following procedures apply to NDOT's review and approval of shop drawings.

25.1.3.1 Contractor Submittals

Section 105.02 "Plans and Working Drawings" of the *NDOT Standard Specifications for Road and Bridge Construction* documents the general requirements with respect to shop drawing submittals. Additional items of work requiring shop drawing submittal and review may be included in the Special Provisions.

The *Standard Specifications* specifies the number of submittal copies, but this may not be adequate for some projects. Copies of the approved shop drawings are needed for the Resident Engineer, prime contractor and bridge designer. Additional copies are needed for the NDT Squad, supplier/fabricator, design consultant and other involved entities. The number of required submittal copies should be reflected in the Special Provisions if different from the *Standard Specifications*.

25.1.3.2 Division Review

In its review of shop drawings, the Structures Division will take one of the following actions on NDOT-designed projects:

1. No Corrections. If everything is correct on the shop drawing, the checker will stamp "APPROVED" on the drawings.

2. Minor Corrections. If the basic concept of the shop drawing is acceptable with only the need for minor corrections, the checker will stamp "APPROVED EXCEPT AS NOTED" on the drawings. The contractor will not be required to resubmit the plans for NDOT review and approval.

3. Major Corrections. If the shop drawings contain major discrepancies and errors, the checker will stamp "RETURNED FOR CORRECTION" on the drawings. The contractor must revise the shop drawings and resubmit the drawings to NDOT with the corrections clearly noted.

On consultant-designed projects, the consultant is responsible for reviewing the shop drawings, determining their acceptability and placing the applicable stamp on the drawings as indicated above. The Structures Division will independently review the shop drawings and convey any suggestions to the consultant for inclusion. The Structures Division will stamp "REVIEWED" on the drawings. The Division's review is usually conducted concurrently with the consultant's review.

The Traffic Division and Structures Division are both responsible for sign, signal and high-mast lighting shop drawings. The Traffic Division will review these submittals for conformance with layout and electrical requirements; the Structures Division will review the structural details.

Discussion of shop drawings issues between the reviewer and supplier/producer is typically conducted through the Resident Engineer and prime contractor. To expedite a resolution of issues and shop drawing approval, the Structures Division will at times correspond directly with the supplier/producer. The Resident Engineer and prime contractor must first approve direct communication with the fabricator. The reviewer must provide copies of all correspondence to the Resident Engineer.

25.1.3.3 Review Periods

The *Standard Specifications* provides 30 calendar days for review and approval of shop drawings. If Railroad approval is also required, the time is increased to 90 days. The Railroad will only review shop drawings that have first been reviewed and approved by the Structures Division. The bridge designer should increase the review time for complex structures or on projects with multiple submittals that require more review time than the standard 30 days. The Special Provisions should explicitly define the increased number of days needed for review and approval and any requirements for scheduling multiple submittals for large projects with several structures.

25.1.3.4 Distribution

After the Structures Division or consultant has approved the shop drawings, distribute copies as outlined below. Copies for the contractor, subcontractors, vendors and suppliers are generally routed through the Resident Engineer. For some submittals (e.g., structural steel, precast girders), copies of approved drawings may be returned directly to the supplier to expedite fabrication. Such direct delivery must first be approved by the Resident Engineer and prime contractor.

The typical distribution for copies of approved shop drawings is as follows:

- one copy for Structures Division files;
- two copies for the NDT Squad, as applicable;
- one copy for the design consultant, as applicable;
- one copy for the Resident Engineer;
- one copy for the prime contractor;
- two copies for the structural steel fabricator or precast concrete producer;
- one copy for other subcontractors, vendors or suppliers; and
- additional prints distributed as needed (e.g., railroad, utility, local agency).

25.1.4 <u>Checklists</u>

Appendix 25A presents the standardized shop drawing checklists used by NDOT. The bridge designer (and others as appropriate) will complete those checklists applicable to a project. It may be determined, on a project-by-project basis, that other shop drawings are required if so noted in the contract documents.

25.2 CONSTRUCTION FIELD AND SHOP INSPECTIONS

25.2.1 General

The Structures Division has an active and significant role in conducting field and shop inspections for the construction of structural items. The bridge designer has a unique perspective and knowledge of the structure design, and this knowledge can help ensure that construction problems are avoided. Therefore, the Division must make a conscientious effort to participate in the construction of structural elements through attendance at project meetings and periodic site visits.

The NDT Squad performs actual construction quality assurance inspection and approval of fabricated structural steel and precast, prestressed concrete members. Chapter 26 presents a more detailed description of the NDT functions.

25.2.2 Responsibilities

The following briefly describes the responsibilities of the various NDOT units in construction inspections of structural items.

25.2.2.1 Resident Engineer (RE)

The RE is NDOT's field representative on construction projects and is responsible for:

- assessing the compatibility of the design with site conditions;
- administering the construction project in accordance with established policies and procedures;
- monitoring projects to ensure compliance with the plans;
- conducting preconstruction conferences and pre-pour conferences;
- enforcing specifications, controlling inspection and testing and ensuring proper documentation;
- resolving issues and disputes with the contractor; and
- preparing contract change orders.

The RE will notify the Structures Division of any issues related to structural items and will coordinate submittal reviews.

25.2.2.2 Assistant District Engineer – Construction

The Assistant District Engineer – Construction is responsible for:

- managing the overall administration of construction projects in the District;
- evaluating, processing and approving change orders;
- resolving disputes and potential claims;

- coordinating construction activities with other District operations; and
- providing day-to-day supervision of the Resident Engineer.

25.2.2.3 Construction Division

The Construction Division within the Headquarters Office is responsible for:

- assigning, as practical, its available staff to conduct field inspections on major areas of structural construction;

- ensuring that the Assistant District Engineers – Construction conduct periodic field inspections on all major structures;

- processing construction progress payments;

- administering contract compliance;

- providing constructibility reviews;

- coordinating change orders initiated by the various NDOT Divisions; and

- processing and responding to contractor claims.

25.2.2.4 Structures Division

25.2.2.4.1 *Bridge Design Squads*

The Bridge Design Squads are responsible for:

- providing technical assistance to the Resident Engineer on construction issues;
- participating in project meetings as necessary;
- participating in construction field and shop inspections; and
- reviewing and approving shop drawings.

25.2.2.4.2 *NDT Squad*

The NDT Squad provides quality assurance in structural steel and precast, prestressed concrete fabrication shops, and observes field welding, erection, post-tensioning and grouting operations in the field. See Chapter 26 for more information.

25.2.3 <u>Field Coordination</u>

The bridge designer can make an initial field visit prior to the start of construction concurrent with the preconstruction conference. This is an opportunity to meet with the RE and the RE's crew to review the contract documents. In addition and as applicable, the bridge designer typically participates in the following field activities to assist and support the RE during construction:

- attend pre- and post-construction meetings;

- attend pre-pour conferences ahead of planned major concrete pours;
- observe concrete placement on major pours;
- accompany the Materials Division to observe foundation subgrade preparation and foundation construction activities;
- on major structural projects, conduct routine field visits approximately once a month (these may be performed in conjunction with regularly scheduled project meetings);
- for cast-in-place structures supported by falsework, observe the construction operation at some point during falsework placement or before the concrete pour is scheduled; and
- provide support for processing change orders and resolving claims and disputes.

Notify the Resident Engineer of planned visits or any issues or problems observed and follow the appropriate safety procedures.

The NDT Squad, as applicable, typically participates in the following field activities to support the Resident Engineer during construction:

- inspect the placement of precast concrete and structural steel girders (Note: The bridge designer may also be on-site during girder placement);
- inspect stressing and grouting operations (Note: The bridge designer may also assist); and
- inspect the steel and precast concrete fabrication sites to provide a full-time quality assurance presence and continuous audit of the fabricator's procedures. (Note: The bridge designer may visit the fabrication sites to observe plant operations).

See Chapter 26 for more discussion on the NDT Squad participation in construction operations.

25.3 MISCELLANEOUS ISSUES

25.3.1 <u>Construction Change Orders</u>

25.3.1.1 Objectives

During construction operations, construction change orders will occasionally be necessary. At times, the Structures Division is responsible for initiating change orders, and reviewing and approving all change orders related to structural items. The objectives of the Division in its review are to:

- determine its agreement with and acceptance of the change order, and
- calculate and verify the quantities and costs.

25.3.1.2 Procedures

Construction change orders will normally be processed with the following basic procedure:

1. <u>Resident Engineer</u>. The RE will notify the Structures Division of the need to process a change order on a structural item.

2. <u>Structures Division</u>. If in agreement, the Chief Structures Engineer will verbally inform the RE of tentative approval. The Principal Structures Engineer – Design will prepare a memorandum for signature by the Chief Structures Engineer to the Chief Construction Engineer, Construction Division, Headquarters. The memorandum will fully address the justification for the change order and will include plans, details and quantities, as applicable.

3. <u>Construction Division</u>. The Construction Division obtains approval to initiate the change order from the Assistant Director – Operations and then authorizes the RE to prepare the change order.

4. <u>Resident Engineer</u>. The RE prepares the change order. The RE then obtains the signatures of the contractor and the District Engineer. The RE submits both copies of the change order to the Construction Division in Headquarters.

5. <u>Chief Structures Engineer</u>. The Construction Division will submit the change order to the Structures Division. If in agreement with the change order, the Chief Structures Engineer, or the Assistant Structures Engineer – Design, will sign the change order. The change order is then sent to the Construction Division. If the Structures Division does not agree with the change order, the Chief Structures Engineer or the Assistant Chief Structures Engineer – Design will sign the change order as “reviewed” but not “reviewed and approved.” In addition, a memorandum is prepared outlining the reasons for not agreeing with the change order.

6. <u>FHWA</u>. If applicable, the Construction Division provides the FHWA with a copy of the change order for review and concurrence.

7. <u>Construction Division</u>. The Construction Division signs and submits the change order to NDOT’s upper management for signature. After signature, the Construction Division circulates all needed copies, including one to the Structures Division.

25.3.2 Bridge Deck Contour Sheets

Bridge deck contour plan sheets are not provided in the contract documents. However, the contour plots may provide a benefit to the field construction personnel. Therefore, bridge deck contour plots may be provided upon the request of the Resident Engineer.

25.3.3 Value Engineering Proposals

Section 105.18 of the *Standard Specifications* allows contractors to submit Value Engineering (VE) Proposals to NDOT "for modifying the plans, specifications or other requirements of the contract for the purpose of reducing the total cost of construction without reducing design capacity or quality of the finished product." The *Standard Specifications* presents the procedures that a contractor must follow for a VE Proposal, which is processed as a Change Order.

The Construction Division will seek input from the Structures Division for any VE Proposals related to structural items. In general, the bridge designer that reviews the Proposal must recognize that the contract documents represent one solution to accomplishing the project objectives. For a variety of reasons (e.g., equipment, specialized contractor expertise, field conditions), this solution may not be the most economical. In the review of the Proposal, the bridge designer must ensure that the proposed design is at least equal to the functionality, durability and longevity of the design presented in the contract documents.

25.3.4 Plan Revisions

25.3.4.1 Requests for Information

During the advertisement period, prospective bidders may submit Requests for Information (RFIs) to NDOT's Project Manager or Project Coordinator. If related to structural items, these RFIs will be forwarded to the Structures Division for a response. If changes to the contract documents are necessary, the bridge designer must coordinate with the Project Manager or Project Coordinator to initiate a Supplemental Notice. At this stage, post all responses to RFIs on the NDOT website.

During construction, the Resident Engineer and/or Contractor may submit RFIs to the Structures Division seeking clarification on provisions, design details, etc., in the contract documents. The bridge designer will respond to these as needed.

For consultant-designed projects, the Structures Division will coordinate with the consultant to respond to RFIs.

25.3.4.2 Revisions During Construction

NDOT construction personnel must ensure that all design changes, whether related to formal change orders or the practical realities of construction, are documented. The Resident Engineer will produce a set of as-built plans that are transmitted to Central Records for storage. These as-built plans are essential for any future bridge rehabilitation projects and for use in future bridge inspections conducted by the Bridge Inspection Squad.

Appendix 25A

SHOP DRAWING CHECKLISTS

Appendix 25A presents the following checklists for each of the following shop drawings:

- cast-in-place, post-tensioned concrete structures (503);
- structural steel (506);
- precast, prestressed concrete I-girders (503);
- falsework (502);
- stay-in-place forms (502);
- overhang forms (502);
- girder erection (503, 506);
- bearings (high-load, multi-rotational) (502);
- expansion joints (502);
- temporary shoring (206);
- MSE walls (640);
- sign/signal/lighting structures (623); and
- precast, concrete box culverts (502).

The applicable Section of the NDOT *Standard Specifications* is noted in parentheses next to the checklist title. The bridge designer must verify compliance with the shop drawing requirements noted in the *Standard Specifications* and contract Special Provisions.

Contract No. ____________________________ Date: ____________________________

Structure No. ____________________________ Reviewer(s): ____________________________

CAST-IN-PLACE, POST-TENSIONED CONCRETE STRUCTURES (503)

Are the following items properly included on the shop drawings for cast-in-place, post-tensioned concrete structures?	**Yes**	**No**	**N/A**
Prestressing Submittal			
1. Drawings.			
a. Prestressing system details.	☐	☐	☐
b. Anchor head.			
i. Anchorage blockout and pour back details.	☐	☐	☐
ii. Necessary local zone reinforcement.	☐	☐	☐
iii. Anchorage set.	☐	☐	☐
c. Standard plan reinforcing and any supplemental reinforcing.	☐	☐	☐
d. Prestressing steel, ducts, tendons and anchorage layout and geometry.	☐	☐	☐
e. Grout vent types and locations.	☐	☐	☐
f. Grout cap details.	☐	☐	☐
g. Stressing location(s) and sequence.	☐	☐	☐
h. Jacking forces.	☐	☐	☐
2. Calculations.			
a. Anchor set losses.	☐	☐	☐

Are the following items properly included on the shop drawings for cast-in-place, post-tensioned concrete structures?	**Yes**	**No**	**N/A**
b. Friction calculations.	☐	☐	☐
c. Bearing stresses at anchorage (if applicable).	☐	☐	☐
d. Elongation.	☐	☐	☐
3. Procedures.			
a. Stressing equipment and procedures.	☐	☐	☐
b. Detensioning procedures.	☐	☐	☐
c. Grouting operations.	☐	☐	☐
4. Certifications.			
a. Anchorage tests and acceptances.	☐	☐	☐
b. Certified grouting technician(s).	☐	☐	☐
Grouting Operations Plan			
1. Equipment information and procedures.	☐	☐	☐
2. Type, quality and brand of materials used.	☐	☐	☐
3. Theoretical grout volumes for each typical duct.	☐	☐	☐
4. Group cap and vent information.			
a. Vent types and locations.	☐	☐	☐
b. Direction of grouting.	☐	☐	☐

Are the following items properly included on the shop drawings for cast-in-place, post-tensioned concrete structures?	**Yes**	**No**	**N/A**
c. Proposed blockouts for vents.	☐	☐	☐
5. Grouting procedures.			
a. Sequence of use of inlets and outlets.	☐	☐	☐
b. Handling blockages.	☐	☐	☐
6. Procedure compliance with specifications.	☐	☐	☐
7. Vertical rise calculations.	☐	☐	☐
8. Cleaning and proofing equipment.	☐	☐	☐
9. Certifications for materials and equipment.	☐	☐	☐

Contract No. ______________________ Date: ______________________

Structure No. ______________________ Reviewer(s): ______________________

STRUCTURAL STEEL (506)

Are the following items properly included on the shop drawings for structural steel?	**Yes**	**No**	**N/A**
1. Principal controlling dimensions and materials.			
a. Length of span adjusted to accommodate roadway profile grade.	☐	☐	☐
b. Length, thickness and width of plates in primary members and splices.	☐	☐	☐
c. Primary dimensions and/or weight per length of rolled shapes.	☐	☐	☐
d. Diameter, specification and grade of mechanical fasteners and coating on faying surfaces (if any), if required.	☐	☐	☐
e. Specification, grade and toughness testing requirements for steel components.	☐	☐	☐
f. Size of fillet welds and partial joint penetration welds; appropriate partial and complete joint penetration weld configurations.	☐	☐	☐
2. Web and flange plates of welded members and rolled girder stringers.			
a. Weld designations.	☐	☐	☐
b. Shop butt weld splice locations.	☐	☐	☐
c. Flange and web tapers and haunches (controlling dimensions only).	☐	☐	☐
d. Cover plate dimensions and termination details.	☐	☐	☐
e. Location of tension and compression zones in welded members.	☐	☐	☐
3. Stiffener and connection plates.			

Are the following items properly included on the shop drawings for structural steel?	Yes	No	N/A
a. Width, thickness, material grade, and if toughness testing required.	☐	☐	☐
b. Weld size and termination details and bolting to web and flange details.	☐	☐	☐
c. Appropriate spacing of intermediate stiffeners.	☐	☐	☐
d. Avoiding interference with shop web and flange splice locations.	☐	☐	☐
e. Fit and location of stiffeners.	☐	☐	☐
f. Bolt hole edge distances and compatibility with diaphragm/ cross-frame connections.	☐	☐	☐
4. Bolted splices.			
a. Flange and web splice plate thickness and dimensions.	☐	☐	☐
b. Number, size and spacing of bolts and holes in splice material.	☐	☐	☐
c. Fill plates if necessary.	☐	☐	☐
d. Proper bolt hole edge distances.	☐	☐	☐
5. Cross-frames and diaphragms.			
a. Member dimensions and orientation.	☐	☐	☐
b. Number and spacing of connection plate bolts and types of holes, especially for slip-critical connections or details required for differential deflections.	☐	☐	☐
c. Size, designation and length of welded connections. Proper weld termination details.	☐	☐	☐
6. Camber and/or mid-ordinate for cambered rolled girders or girder sections.	☐	☐	☐
7. Procedures and sequence for shop assembly including handling methods.	☐	☐	☐

Are the following items properly included on the shop drawings for structural steel?	Yes	No	N/A
8. Elevation at center of span or segment, field splice, abutment and pier ordinates on shop assembly diagrams.	☐	☐	☐
9. Number and spacing of bolts in floor girder and cross girder connections and special attachments (brackets, pot bearings, etc.).	☐	☐	☐
10. Necessary provisions for overhang jack assemblies and fall protection systems are depicted.	☐	☐	☐
11. Bearings and Expansion Joints — typically provided as separate submittals (see applicable checklist).	☐	☐	☐
12. Erection plan, showing general layout of structural steel framing and erection equipment. (Note: This is typically a separate submittal; see Girder Erection checklist).	☐	☐	☐
13. General Notes and Detail Sheets relative to cleaning and painting.			
a. Corner preparation (if required for cut edges).	☐	☐	☐
b. Cleaning, required surface preparation and profile depth (if specified).	☐	☐	☐
c. Shop primer: type; manufacturer; wet or dry film thickness; verification of cure before shop application of subsequent coatings; applicable restrictions on field contact (faying) surfaces; any requirements for pre-priming shop contact surfaces before assembly; and designation of any field weld areas to be left unprimed.	☐	☐	☐
d. Field and top coat(s); shop or field; type; manufacturer; wet or dry film thickness; intermediate coat cure times and/or recoating "window" (time) specified by the contract documents or paint manufacturer's data sheet; any blockout areas where shop top coats are not permitted (e.g., field splices, diaphragm/cross-frame connections, bearings, etc.).	☐	☐	☐
14. Designation of material, tension zones, and welds for fracture-critical members (FCMs), including applicable nondestructive testing.	☐	☐	☐
15. Material and material testing.			
a. Material specified in accordance with the contract documents.	☐	☐	☐
b. Proposed material substitutions.	☐	☐	☐
16. Incorporation of all necessary revisions into the Shop Drawings.			

Are the following items properly included on the shop drawings for structural steel?	Yes	No	N/A
a. Errors or discrepancies in the contract plans discovered during Shop Drawing preparation or review.	☐	☐	☐
b. All construction changes that affect the Shop Drawings.	☐	☐	☐
c. Fabricator-proposed modifications approved by NDOT and Contractor.	☐	☐	☐
17. Framing plan details.			
a. Basic span lengths and, where appropriate, transverse girder spacing.	☐	☐	☐
b. Pier and abutment identifications.	☐	☐	☐
c. Orientation of structure (north arrow), skew(s), spot checks of curve or flare geometry, if applicable.	☐	☐	☐
d. Piecemarks indicated for every element, and their relative location is shown to clarify member orientation.	☐	☐	☐
18. Answer or acknowledge all appropriate questions noted on Shop Drawings as “Engineer verify” (does not include “Contractor verify” or “Field verify” queries that must be resolved by others before final Shop Drawing approval).	☐	☐	☐
19. Verification of fabricator certification.	☐	☐	☐
20. Compliance with project-specific requirements that may supersede the requirement of this checklist (such as utility attachments, special connections or connection materials (pins, links, cables), and stage removal and construction).	☐	☐	☐

Contract No. ____________________________ Date: ___________________________________

Structure No. ___________________________ Reviewer(s): ____________________________

PRECAST, PRESTRESSED CONCRETE I-GIRDERS (503)

Are the following items properly included on the shop drawings for precast, prestressed concrete I-girders?	**Yes**	**No**	**N/A**
1. All dimensions including total length of girder adjusted to accommodate roadway profile grade and including 0.0075 in per ft of girder length for elastic shortening.	☐	☐	☐
2. Erection plan, showing general layout of the concrete elements including diaphragm locations (Note: This may be a separate submittal; see Girder Erection checklist).	☐	☐	☐
3. The number and size of all members (completed girders shall be marked with an assigned production number).	☐	☐	☐
4. The number, size and type of prestressing strands, their locations and the forces in these prestressing elements.	☐	☐	☐
5. Girder end details, including size of blockouts at abutments, location and diameter of holes or inserts and embedded bearing plates.	☐	☐	☐
6. Bearing details showing mark number, number required, size, type, materials, including anchor bolts and sole plates (Note: This may be a separate submittal).	☐	☐	☐
7. The location and details of lifting devices and support points if the girder will not rest on its bearings while being stored or transported.	☐	☐	☐
8. The location and type of any inserts required for attachments.	☐	☐	☐
9. The layout of the casting bed to be used for casting the prestressed girders showing the location of hold-down devices for any harped strands.	☐	☐	☐
10. Methods for providing and controlling required girder camber during casting, transport and erection.	☐	☐	☐
11. The location and length of any de-bonded prestressing strands.	☐	☐	☐
12. Jacking forces, number of strands and sequence of harping and detensioning.	☐	☐	☐
13. Tendon path showing straight and harped strands, including deflecting saddles (provide details and required number).	☐	☐	☐
14. The details and type of reinforcing steel, rebar mark number, rebar size, number per girder, total number, length each, total length, total weight, bent rebar, minimum lap for size of bar used and grade of rebar used.	☐	☐	☐

Are the following items properly included on the shop drawings for precast, prestressed concrete I-girders?	Yes	No	N/A
15. All general notes and construction notes presented in the contract plans properly reflected in the shop drawings.	☐	☐	☐
16. Verification of fabricator certification.	☐	☐	☐
17. Girder design and stress calculations (stamped by a Nevada registered civil or structural professional engineer) for proposed modifications to girders to accommodate fabricator's operations.	☐	☐	☐
18. Girder curing methods.	☐	☐	☐
19. Concrete mix design submitted and approved by Materials Division.	☐	☐	☐

Contract No. ___________________________ Date: ________________________________

Structure No. __________________________ Reviewer(s): __________________________

FALSEWORK (502)

Are the following items properly included on the shop drawings for falsework?	**Yes**	**No**	**N/A**
1. General layout including plan and elevation views plus adequate typical sections and special details.	☐	☐	☐
2. Horizontal and vertical clearances; adequate openings for pedestrian and vehicular traffic.	☐	☐	☐
3. Provisions for overheight vehicle protection.	☐	☐	☐
4. Material designation; member sizes and spacing.	☐	☐	☐
5. Concrete placement sequence and construction joint locations.	☐	☐	☐
6. Provisions for grade adjustments and superstructure camber.	☐	☐	☐
7. Determination of nominal soil bearing resistance for wet and dry conditions.	☐	☐	☐
8. Data for proprietary systems or manufactured assemblies (e.g., overhang jacks, metal scaffolding); verify acceptable working loads and deflections.	☐	☐	☐
9. Design calculations (stamped by a Nevada registered civil or structural professional engineer) conforming to contract documents.			
a. Design loads and member stresses.	☐	☐	☐
b. Lost deck formwork.	☐	☐	☐
c. Connections and bracing.	☐	☐	☐
d. Settlements and deflections.	☐	☐	☐
e. Local bending and buckling effects.	☐	☐	☐
f. Mud sills/footings meet soil bearing requirements.	☐	☐	☐

Are the following items properly included on the shop drawings for falsework?	**Yes**	**No**	**N/A**
10. Provision for accessing low-point drains in prestressing ducts.	☐	☐	☐
11. Provisions for metal stay-in-place forms (Note: This may be a separate submittal).	☐	☐	☐
12. Provisions for deck overhang supports/jacks (Note: This may be a separate submittal).	☐	☐	☐
13. Sequence and method for falsework removal.	☐	☐	☐

Contract No. ____________________________ Date: __________________________________

Structure No. ____________________________ Reviewer(s): ___________________________

STAY-IN-PLACE FORMS (502)

Are the following items properly included on the shop drawings for stay-in-place forms?	**Yes**	**No**	**N/A**
1. General plan layout including structure support locations, girder lines and location of SIP forms; if applicable, clearly designate use of varying SIP form sections throughout the structure (irregular girder spacing).	☐	☐	☐
2. Typical section(s) and dimensions.	☐	☐	☐
3. Deck thickness and minimum concrete cover.	☐	☐	☐
4. Material designations and galvanized coating.	☐	☐	☐
5. Maximum allowable form weight including corrugation fill.	☐	☐	☐
6. Design loading and SIP form capacity data and/or calculations.	☐	☐	☐
7. Design calculations (stamped by a Nevada registered civil or structural professional engineer) conforming to contract documents.	☐	☐	☐
8. Maximum allowable deflection not exceeded.	☐	☐	☐
9. Girder connection details; no welding to structural steel girders.	☐	☐	☐

Contract No. ____________________________ Date: ________________________________

Structure No. ___________________________ Reviewer(s): __________________________

OVERHANG FORMS (502)

Are the following items properly included on the shop drawings for overhang forms?	**Yes**	**No**	**N/A**
1. General plan layout, typical section(s) and dimensions.	☐	☐	☐
2. Material designations; member sizes and spacing.	☐	☐	☐
3. Finishing machine rail support located beyond the perimeter of the bridge deck.	☐	☐	☐
4. Data for proprietary systems or manufactured assemblies; verify acceptable working loads and deflections.	☐	☐	☐
5. Design calculations (stamped by a Nevada registered civil or structural professional engineer) conforming to contract documents.			
a. Design loads, member stresses and allowable deflections.	☐	☐	☐
b. Local girder bending and buckling effects; girder bracing.	☐	☐	☐

Contract No. ____________________ Date: ____________________

Structure No. ____________________ Reviewer(s): ____________________

GIRDER ERECTION (503, 506)

Are the following items properly included on the shop drawings for erection?	Yes	No	N/A
1. Span lengths along base line; splice locations; degree/direction of curve and skew.	☐	☐	☐
2. Spacings (beams, diaphragms, cross bracings, anchor bolts).	☐	☐	☐
3. Match marking diagram and weight of each piece identified.	☐	☐	☐
4. General logistics and sequence of girder erection.	☐	☐	☐
5. Erection towers identified and details provided.	☐	☐	☐
6. Girder pick-points identified consistent with girder fabrication drawings; description of pick-up devices (cables, clamps, etc.) and method of protecting girders.	☐	☐	☐
7. Crane location(s); location(s) relative to existing facilities (rail lines, utilities, etc.).	☐	☐	☐
8. Verify crane capacity based on swing and reach requirements.	☐	☐	☐
9. Girder bracing/blocking.	☐	☐	☐
10. Evaluation of existing bridge to support loaded delivery vehicle and/or crane; special structure protection details.	☐	☐	☐
11. Calculations to verify that girders are not overstressed due to erection procedures (pick-up points not consistent with girder fabrication drawings).	☐	☐	☐
12. Verification of erector certification, if required.	☐	☐	☐

Contract No. ____________________________ Date: ________________________________

Structure No. ___________________________ Reviewer(s): __________________________

BEARINGS (502)

Are the following items properly included on the shop drawings for bearings?	**Yes**	**No**	**N/A**
1. Location diagram showing the general layout of the structure with the locations and orientation of the bearings.	☐	☐	☐
2. The number, size and types of all bearings.	☐	☐	☐
3. Plan and elevation of bearings showing dimensions, tolerances and fabrication details; details of all components.	☐	☐	☐
4. Bearing fabrication and assembly details; welding details.	☐	☐	☐
5. Material designations and testing requirements are noted.	☐	☐	☐
6. Steel surface preparation and shop coating details.	☐	☐	☐
7. Design calculations conforming to contract documents, if necessary.	☐	☐	☐

Contract No. ____________________ Date: ____________________

Structure No. ____________________ Reviewer(s): ____________________

EXPANSION JOINTS (502)

Are the following items properly included on the shop drawings for expansion joints?	**Yes**	**No**	**N/A**
1. Product is on the Qualified Products List.	☐	☐	☐
2. General layout and dimensions (overall length, skew angle). Orientation of expansion joint components within the joint blockout.	☐	☐	☐
3. The number, size (movement rating) and types of all expansion joints.	☐	☐	☐
4. Plan and elevation views and sections for all components.	☐	☐	☐
5. Material designations for all components; coatings.	☐	☐	☐
6. Installation widths (minimum and maximum) noted including provisions for temperature variations.	☐	☐	☐
7. Manufacturer recommendations for installation methods and procedures. Required attendance of manufacturer's technical representative noted, if applicable.	☐	☐	☐
8. Where applicable, strip seal glands or joint fillers are provided as one continuous piece. Details for method of splicing glands or joint fillers for non-continuous installations.	☐	☐	☐
9. Details for shop and field welding of steel joint components.	☐	☐	☐
10. Supplementary items for modular expansion joints.			
a. AISC certified fabricator.	☐	☐	☐
b. Installation Manual: Installation sequence and procedures, lifting locations and mechanisms, leveling assemblies details, adjustments for temperature changes, temporary and permanent anchorage to bridges, and shipping and storage requirements.	☐	☐	☐
c. Maintenance Manual: Maintenance plan, parts list, parts replacement schedule and inspection instructions.	☐	☐	☐
d. Design calculations and fatigue testing conforming to contract documents.	☐	☐	☐

Contract No. ___________________________ Date: ________________________________

Structure No. __________________________ Reviewer(s): __________________________

TEMPORARY SHORING (206)

Are the following items properly included on the shop drawings for temporary shoring?	**Yes**	**No**	**N/A**
1. Elevation view showing existing and proposed ground elevations.	☐	☐	☐
2. Plan view showing the beginning and ending stations and alignment.	☐	☐	☐
3. Typical section view showing composition and configuration of the shoring system with dimensions, member sizes, embedment length and height of retained fill.	☐	☐	☐
4. Material designations for all shoring system components.	☐	☐	☐
5. Connection details for various system components including weld details and specifications.	☐	☐	☐
6. Design soil properties including unit weight, coefficients of active and passive pressure, live load surcharge, railroad surcharge (as applicable) and location of groundwater table.	☐	☐	☐
7. Design calculations (stamped by a Nevada registered civil or structural professional engineer) conforming to contract documents.	☐	☐	☐
8. Calculations and details demonstrate conformance to OSHA regulations.	☐	☐	☐
9. Shoring construction sequence.	☐	☐	☐
10. Geotechnical Section review and recommendation for approval.	☐	☐	☐

Contract No. ____________________ Date: ____________________

Structure No. ____________________ Reviewer(s): ____________________

MSE WALLS (640)

Are the following items properly included on the shop drawings for MSE walls?	**Yes**	**No**	**N/A**
1. Product is on the Qualified Products List, QPL remarks are addressed.	☐	☐	☐
2. Plan view indicating beginning and ending stations, offset to front face of wall and stationing for any change in wall alignment.	☐	☐	☐
3. Elevation view indicating beginning and ending stations, elevations at top of leveling pad for each step, panel layout and designation.	☐	☐	☐
4. Typical wall section(s) showing overall height and length of reinforcement.	☐	☐	☐
5. Soil reinforcing layout and limits of MSE backfill.	☐	☐	☐
6. Soil reinforcing corrosion protection.	☐	☐	☐
7. Typical section of wall coping, anchor slab and barrier rail.	☐	☐	☐
8. Typical details.			
a. Wall panel details with sections at top, bottom, left and right sides; details of the vertical and horizontal joints; maximum panel size.	☐	☐	☐
b. Leveling pad including steps.	☐	☐	☐
c. Panel to soil reinforcement connections.	☐	☐	☐
d. Aesthetic treatments.	☐	☐	☐
e. Filter cloth.	☐	☐	☐
9. Material designations for soil reinforcing, concrete embedments and connection hardware.	☐	☐	☐

Are the following items properly included on the shop drawings for MSE walls?	Yes	No	N/A
10. Design calculations (stamped by a Nevada civil or structural professional engineer) conforming to contract documents.			
a External and internal stability design verified (reviewed by geotechnical engineer).	☐	☐	☐
b. Wall panels.	☐	☐	☐
c. Reinforcement bridging frames; reinforcing layout and capacity adjustments to accommodate obstructions.	☐	☐	☐
11. Notes indicating special construction procedures recommended or required by the wall manufacturer.	☐	☐	☐

Contract No. ____________________________ Date: ________________________________

Structure No. ___________________________ Reviewer(s): ___________________________

SIGN/SIGNAL/LIGHTING STRUCTURES (623)

Are the following items properly included on the shop drawings for sign/signal/lighting structures?	**Yes**	**No**	**N/A**
1. Plan and elevation views showing structure configuration and orientation to roadway.	☐	☐	☐
2. Minimum vertical clearance.	☐	☐	☐
3. Material designations.	☐	☐	☐
4. Member sizes, dimensions and coatings.	☐	☐	☐
5. Complete fabrication details, connections details, provisions for accommodating structure deflection or providing structure camber.	☐	☐	☐
6. Anchor bolts/assemblies.	☐	☐	☐
7. For deviations from Standard Plans or contract document details or for designated contractor designed installations — Design calculations (stamped by a Nevada registered civil or structural professional engineer) conforming to contract documents.	☐	☐	☐
8. Special installation requirements.	☐	☐	☐

Contract No. ____________________ Date: ____________________

Structure No. ____________________ Reviewer(s): ____________________

PRECAST, CONCRETE BOX CULVERTS (502)

Are the following items properly included on the shop drawings for precast, concrete box culverts?	**Yes**	**No**	**N/A**
1. Design calculations conforming to *NDOT Standard Specifications*.	☐	☐	☐
2. Concrete box dimensions and height of earth fill.	☐	☐	☐
3. Reinforcing materials, sizes, dimensions, orientation and minimum concrete cover.	☐	☐	☐
4. Concrete 28-day compressive strength and mix design. Identify casting process and sources and mass proportions of all materials.	☐	☐	☐
5. Joint details and materials.	☐	☐	☐
6. Laying schedule with section piece-marks, section lengths, total length, horizontal and vertical alignment, stationing and invert elevations.	☐	☐	☐
7. All general notes and construction notes presented in the contract plans properly reflected in the shop drawings.	☐	☐	☐

Chapter 26

NON-DESTRUCTIVE TESTING (NDT)

NDOT STRUCTURES MANUAL

September 2008

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Chapter 26
NON-DESTRUCTIVE TESTING (NDT)

26.1 NDT SQUAD

26.1.1 <u>Responsibilities</u>

26.1.1.1 General

The Non-Destructive Testing (NDT) Squad is in the Inventory/Inspection Section within the Structures Division. In general, the Squad provides all in-house NDT services required by NDOT for a variety of applications. The following Sections summarize these services.

26.1.1.2 Construction Support

The NDT Squad performs the following activities in support of the NDOT Construction Program:

- quality assurance of shop fabrication for structural steel and precast, prestressed concrete;
- inspections for field welding, erection, post-tensioning and grouting operations;
- QA for field verification and production testing of soil nails;
- QA for field testing and loading of tie-backs; and
- bridge deck evaluation.

See Section 26.2 for more discussion.

26.1.1.3 Existing Structures

The NDT Squad performs non-destructive testing to evaluate the condition of existing structures in support of the following:

- the Nevada Bridge Inspection Program, and
- bridge rehabilitation projects.

In addition, the NDT Squad can perform certain maintenance repairs on existing bridges (e.g., grinding and/or drilling of fatigue cracks). See Section 26.3 for more discussion.

26.1.2 <u>Qualifications of NDT Personnel</u>

In general, all NDOT personnel performing non-destructive testing must meet the requirements of NDOT/ASNT CP-011-2005 "NDOT Standard for Qualification and Certification of Non-Destructive Testing Personnel." Section 28.2.2 discusses the positions and responsibilities of NDT Squad personnel specifically for the Nevada Bridge Inspection Program.

26.2 CONSTRUCTION SUPPORT

26.2.1 Quality Assurance (QA) of Shop Steel Fabrication

The NDT Squad provides a continuous audit of the contractor's procedures during shop fabrication. Guidance on Quality Assurance policies and procedures for steel-bridge components can be found in AASHTO/NSBA Steel Bridge Collaboration S4.1-2002 "Steel Bridge Fabrication QC/QA Guide Specification."

26.2.1.1 Preparation

The following procedures will apply to steel shop fabrication inspection:

1. Prior to fabrication, the following shall have been received, reviewed and approved:

    - Shop drawings,
    - Welding procedures and welder qualifications,
    - AISC Certifications,
    - *Fabricators Quality Control Manual*,
    - Request to perform radiographic testing after normal work hours,
    - Notice of beginning of work,
    - Heat cambering and straightening procedure, and
    - Fracture Control Plan, if fabricating fracture critical members.

26.2.1.2 Pre-Operation Activities and Checks

The NDT Inspector will:

1. Meet with the QC and shop supervisor and discuss QC/QA shop protocol and fabrication protocol in general. Discuss additional requirements if fabricating fracture critical members.

2. Review all Material Certifications and record on appropriate NDOT form. Visually inspect material for any deficiencies.

3. Record all Heat Numbers for the structural steel used in individual girders and verify with Mill Certifications. Verify that all components are stamped with heat numbers for traceability.

4. Prior to welding, inspect welding machines for proper operation and record results. Check electrode oven and wire and flux storage area.

5. Prior to welding, randomly verify joint fit-up and verify that the configuration for full-penetration welds and web-to-flange fillet weld complies with drawings and Welding Procedure.

26.2.1.3 Welding and Painting Operations

The NDT Inspector will:

1. During welding, verify that welder is qualified to make specific weld(s). Monitor preheat, interpass temperature and travel speed.

2. Inspect all finished welds visually and record results on appropriate NDOT form.

3. Witness all QC Non-Destructive Testing of weldments. Obtain a copy of the QC test report in a timely manner and record results on appropriate NDOT form. QC personnel shall perform all NDT in accordance with the *Standard Specifications for Road and Bridge Construction,* Section 506, latest addition of AWS D1.5 and QC NDT Procedure.

4. Perform the required QA Non-Destructive Testing per the *Standard Specifications for Road and Bridge Construction*, Section 506, latest addition of AWS D1.5 and Special Provisions. Record all results on appropriate NDOT form and report results to fabricator's QC personnel.

5. Repair all welds in accordance with approved weld procedure and re-inspect repaired welds. Obtain QC test report and record results on appropriate NDOT form. If welds are fracture critical, verify that additional requirements are met.

6. Upon completion of welding, verify that flange tilt, web flatness and sweep complies with the latest addition of AWS D1.5. Record results on appropriate NDOT form.

7. During shop assembly, verify that camber and stiffener locations comply with shop drawings and AWS D1.5. Record results on appropriate NDOT form. Note: If heating is required for cambering, verify that the temperature of base metal is appropriate, and that blocking is in accordance with the Heat Straightening and Cambering Procedure.

8. Verify that shop and field high-strength bolts comply with the *Standard Specifications for Road and Bridge Construction,* Section 506.03.07 and Special Provisions. Record test results on appropriate NDOT form. Reseal field bolt containers and identify each container.

9. Prior to painting, verify that all welds and base metal repairs have been made and accepted.

10. Verify that painting complies with *Standard Specifications for Road and Bridge Construction*, Section 614, manufacturer's recommendations and Special Provisions for paint system. Verify dry-film thickness on each coat, cure time and finished product. Make necessary repairs. Record dry-film thickness, surface finish, humidity, ambient air and surface temperature results for each girder on appropriate NDOT form.

11. Verify that all welding, dimensional tolerances, bolting and painting complies with *Standard Specifications for Road and Bridge Construction*, Special Provisions and shop drawings. Verify that all non-conformances have been corrected. Ensure that all NDT results, dimensional measurements, bolting tests and paint measurements are recorded on the appropriate Final NDOT Inspection Report.

12. Upon compliance with items mentioned in #11 above, release girders for shipment using appropriate Inspection Shipping List Form.

26.2.2 Post-Tensioning Inspections

NDT Squad members and bridge designers should be familiar with the following procedures applicable to post-tensioning inspections. The objective of post-tensioning inspections is to ensure conformance to the design requirements.

26.2.2.1 Preparation

Before the start of any post-tensioning operation, it is the responsibility of the NDT Inspector to:

- review the *NDOT Standard Specifications* and Special Provisions, specifically Section 503;
- review the approved shop drawings to obtain the jacking forces, stressing sequence and elongation requirements;
- review grouting plan if prepackaged grout will be used;
- meet with bridge design personnel to discuss the stressing operation and to obtain pertinent information (e.g., the number of strands that can be lost prior to the replacement of a tendon and by how many kips to reduce the force if this occurs);
- discuss safety procedures needed for working around post-tensioning and grouting operations;
- check grout certifications; and
- ensure that all strands have been tested and reels checked by NDOT.

26.2.2.2 Materials

The NDT Inspector should have the following items upon arrival at the job site:

- electric hydraulic pressure cell,
- indicator readout unit,
- approved shop drawings,
- diary,
- *NDOT Standard Specifications* and Special Provisions,
- jack calibration charts,
- post-tensioning field worksheet (see Figure 26.2-A),
- approved grouting procedure,
- grout cone,
- strand-testing results,
- detensioning procedures, and
- Tendon Grout Acceptance Checklist.

NEVADA DEPARTMENT OF TRANSPORTATION
STRUCTURES DIVISION – NDT POST-TENSIONING FIELD WORKSHEET

CONTRACT NO.:	STRUCTURE NO.:	
DATE:	INSPECTOR:	
STRAND TESTED: DATE: (SEE TEST LOG)	RAM NO.	GAUGE NO.
NAME OF SYSTEM:	STRESSING FOREMAN:	
SPECS = JACKING FORCE Kn/kips		
CALCULATED ELONGATION (mm/in) (80%)	THEORETICAL SEATING LOSS (mm/in)	
PAGE 1 OF		

TENDON/ STRAND NO.	JACKING FORCE kips 20%	JACKING FORCE kips 100%	CALCULATED psi 20%	CALCULATED psi 100%	ACTUAL psi 20%	ACTUAL psi 100%	ELONGATION abut no. in/mm	ANCHOR SET in/mm	COMMENTS

POST TENSIONING FIELD WORKSHEET

Figure 26.2-A

26.2.2.3 Resident Engineer Discussions

Upon arrival at the job site, the NDT Inspector should meet with the Resident Engineer to discuss the following:

- if the requirements on the age of the concrete and the compressive strength have been met;

- when placement of strands begin and corrosion inhibitor requirement (when necessary, a certification is required); and

- whether ducts have passed the required pressure tests.

The NDT Inspector shall also meet with the subcontractor's Stressing Foreman to discuss or obtain the following:

- work schedule,
- a copy of the calibration charts for the gauge and ram to be used,
- stressing procedure,
- grouting operations plan, and
- confirm required backup equipment is on-site.

26.2.2.4 Pre-Operation Checks

Before the start of the stressing operation, it is the responsibility of the Inspector to check the following:

- All ducts have the required number of strands.

- All strands have wedges on them.

- The ram and jack for the Identification Number are available (to ensure they match the calibration charts submitted).

- Grout inlets and outlets are functioning properly and tagged.

- The ram, hoses and stressing equipment are in good operating condition.

26.2.2.5 Stressing Operation

During the stressing operation, it is the responsibility of the NDT Inspector to perform the following:

- Calibrate the readout unit and pressure cell before incorporating it into the stressing system.

- Calculate jacking forces using current ram calibration charts and compare calculations with stressing supervisor’s calculations.

- Ensure that all stressing personnel are aware of the area where the tendons must be painted for elongation and slippage.

- When the stressing begins, the tendon is first pulled to 20% of the total jacking force and a mark made either on the tendon itself or the ram. The jacking end of the tendon is also painted to check for wedge slippage. The tendon is then stressed until the total jacking force is reached; the gauge pressure is then recorded. At this time, the previously made mark is measured from the face of the jack. The measured value is compared to 80% of the total calculated elongation. After this has been recorded, the jack is backed off to the 20% point and another measurement is taken. The difference between this measurement and the last one will be the anchor set. Record the measured anchor set. After the jack has been backed off completely and before it is removed, the paint marks at the stressing end should be checked to ensure that none of the wedges have slipped. After stressing is completed, both ends of the tendons should be checked to ensure that all wedges are seated and there has been no slippage.

- Perform an in-place Modulus of Elasticity Test and Friction Test if elongations do not meet expected values.

- Flame cutting of strand in upper tendons before completion of all stressing is not permitted. The NDT Inspector shall monitor the sawing to see that abrasive methods are used and no damage occurs to the unstressed strand.

26.2.2.6 Grouting Operation

During the grouting operation, it is the responsibility of the NDT Inspector to ensure the following:

- All equipment meets the requirements of the approved Grouting Operations Plan including the grout mixer, grout storage hopper, grout pump and standby flushing equipment.

- Grouting technicians have the required training and documentation.

- A joint meeting has been held with the contractor, grouting technician and NDOT personnel to discuss testing, corrective procedures and other relevant issues.

- Conduct a Tendon Grout Trial Batch Test.

- Conduct a duct pressure test.

- Grout is prepackaged and conforms to the requirements for a Class C grout, as defined by the Post-Tensioning Institute (PTI) Specification for Grouting of Post-Tensioned Structures.

- The grout must be continually agitated during pumping.

- The grouting mixture must be checked using a flow cone. The flow cone is plumbed and filled with a known quantity of grout and the time required to empty the cone is dependent upon the type of grout; see specifications for efflux time requirements.

- The grouting equipment must be capable of pumping at a pressure of at least 150 psi but not more than 250 psi.

26.2.2.7 Potential Issues

The NDT Inspector should be aware of the following potential issues:

1. Variations in Elongation. The measured elongation should not vary by more than 5% from the calculated value. If the measured elongation is not close to the calculated value, the method of measurement should be closely watched. Ensure that 20% of the final tendon stress is being used for the initial stress. Ensure that the contractor is accurately marking and measuring the elongations, and all wedges are properly installed. If problems continue, contact the Structures Division.

2. Strand Breakage. If a strand breaks, immediately stop the stressing operation and record the force in the tendon. The Structures Division will recommend a new jacking force or will recommend the complete removal of the tendon. Ensure that the stress limits were not exceeded. Check the actual stresses in the strand and compare with the stress limits prior to seating, immediately after seating, and at the end of the seating loss zone. For example:

    - 10-strand, ½-in diameter tendon stressed to $0.75f_{pu}$
    - P_j = (0.75)(270 ksi)(10 strands)(0.153 in^2) = 309.8 kips
    - Assume one strand breaks when the jacking force is 305 kips
    - Stress in remaining 9 strands = (305 kips)/((0.153 in^2)(9 strands)) = 221.5 ksi
    - Stress limit = $0.9f_{py}$ = 0.9 ($0.9f_{pu}$) = (0.9)(0.9)(270 ksi) = 218.7 ksi
    - Entire tendon must be rejected because the stress in the remaining 9 strands is greater than the stress limit.

    Note: Check other critical stress limits also.

3. Wedge Slippage. If wedges are slipping during the stressing operation, the stressing operation should be stopped. The contractor shall correct the slippage before continuing. Some slippage is acceptable at low levels of force. If slippage occurs at a high force, the Structures Division should be contacted.

4. Equipment. If the contractor's equipment does not perform in a satisfactory working manner (pump surges, leaky lines, faulty gauges, etc.), the stressing operation shall be stopped and not continued until the contractor has fixed or replaced the faulty equipment.

26.2.2.8 Record Keeping

The NDT Inspector shall maintain the following records for post tensioning:

1. Daily Diary. The NDT Inspector should keep accurate records on the number of strands stressed or grouted that day. Any item discussed with the Resident Engineer or Stressing Foreman shall be noted plus any problem area that may arise.

2. Post-Tensioning. Post-tensioning report forms shall be completed daily.

3. <u>Final Report</u>. It is the responsibility of the Inspector to submit a final report on all post-tensioning.

26.2.3 <u>Precast/Prestressed Concrete Girder Fabrication</u>

26.2.3.1 Preparation

The NDT Inspector is responsible for the following:

1. Prior to fabrication, the following items are submitted and approved:
   - shop drawings,
   - mix design,
   - curing method,
   - PCI Certification,
   - *Fabricators Quality Control Manual*, and
   - current ram calibration certifications.
2. Holding a Pre-fabrication Meeting to discuss fabrication procedures, QC testing, stressing and curing procedures.
3. Reviewing all material certifications and strand test results from NDOT.

26.2.3.2 Pre-Operation Checks

The NDT Inspector is responsible for the following:

1. Verifying that forms are free of rust and dents and are in good shape and clean.
2. Verifying that strand placement complies with the *Standard Specifications for Road and Bridge Construction*, Section 503, Special Provisions and shop drawings.
3. Tracking heat/reel numbers for each girder. Rejecting any strand that does not comply with the above-mentioned specifications.
4. Verifying that the stressing operation complies with the *Standard Specifications for Road and Bridge Construction*, Section 503, Special Provisions and shop drawings.

26.2.3.3 Prestressing Operations

The NDT Inspector is responsible for the following:

1. Monitoring stressing operation using a calibrated hydraulic pressure cell with a strain gage indicator. Recording stressing results on appropriate NDOT form.
2. Verifying that reinforcing steel, inserts and bearing placement complies with shop drawings.
3. Recording inspection results on daily construction reports. Making any operational repairs or adjustments to comply with drawings.

4. Verifying that concrete complies with mix design and that concrete testing is conducted in compliance with specifications.

5. Verifying that concrete curing complies with the *Standard Specifications for Road and Bridge Construction*, Section 503 and Special Provisions. Recording all concrete test results, curing temperatures and deficiencies on appropriate NDOT form.

6. Verifying that concrete transfer strength has been achieved prior to release of strand (cut-down).

7. Verifying condition of girder after forms are removed. Documenting any defects for repairs on appropriate NDOT form.

8. Verifying that dimensional tolerances comply with the *Standard Specifications for Road and Bridge Construction*, Section 503, and Special Provisions. Recording deficiencies on appropriate NDOT form.

9. Reviewing all test data, dimensional checks and non-conformance reports for final compliance to the *Standard Specifications for Road and Bridge Construction*, Section 503, Special Provisions and shop drawings. Releasing girders for shipping if girders meet above-mentioned specifications and drawings.

26.3 EXISTING STRUCTURES

The NDT Squad will perform the following tests and work on existing structures, as needed, in support of the Nevada Bridge Inspection Program or to assist the bridge designer in identifying any necessary work for a proposed bridge rehabilitation project.

26.3.1 Testing Methods for Cracking in Metals

The extent and size of cracks should be established to determine the appropriate remedial action, if visual inspection by the bridge inspector reveals cracking in steel components. The following are the most common test methods performed by the NDT Squad to locate cracks in steel components and measure their extent and size:

1. Dye-Penetrant Testing (PT). The surface of the steel is cleaned, then painted with a red dye. The dye is allowed time to "dwell" on the area and then is wiped off. If a crack is present, the dye penetrates the crack through capillary action. A white developer is the painted on the cleaned steel and any cracks are indicated where the red dye "bleeds" from the crack.

2. Magnetic-Particle Testing (MT). The surface of the steel is cleaned and sprinkled with fine iron filings while a strong magnetic field is induced in the steel. A crack causes an interruption in the lines of magnetic flux, allowing them to "leak" from the metal, thereby attracting the metal filings, which form a trace along the line of the crack.

3. Ultrasonic Testing (UT). Testing devices that use high-frequency sound waves to detect cracks, discontinuities and flaws in materials. The accuracy of UT depends upon the expertise of the individual conducting the test and interpreting the results.

4. Eddy Current Testing (ET). Eddy Current testing uses a phenomenon called electromagnetic induction to detect flaws in conductive materials. This form of testing detects flux leakage emanating from a discontinuity in metal when an eddy current is passed through the material. Eddy Current testing can detect very small cracks in or near the surface of the material, the surfaces need minimal preparation, and physically complex geometries can be quickly investigated.

All tests must be conducted by, at a minimum, a Level II ASNT certified technician. For more information, see *Detection and Repair of Fatigue Damage in Welded Highway Bridges*, NCHRP Report 206, July 1979.

26.3.2 Fatigue-Damage Retrofits

When cracking is discovered during in-service inspections, the NDT Squad may retrofit minor cracking. The retrofits consist mainly of grinding or drilling holes (see Chapter 22 for more details on these retrofit procedures, especially Section 22.7.2.1). When lead-based paint is encountered during a retrofit, the NDT Inspector shall adhere to the following procedures.

26.3.2.1 Policy/Procedure Statement for Fatigue Crack Repair Involving Lead-Based Paint

Fatigue damage may be found in steel members during bridge inspections. Often, fatigue cracks are found that are small in size and of such a nature that they may be easily removed or stabilized by grinding or drilling methods. NDOT bridge inspection or NDT personnel, while still on-site, may retrofit this type of fatigue damage during the bridge inspection. Retrofittable fatigue damage may exist in bridges with coating systems containing lead or other heavy metals. The mechanical action of grinding or drilling may cause particles of lead-based paint or coating to become airborne. Accordingly, work in this environment must comply with the OSHA Lead Construction Standard (1926.62), in addition to all applicable NDOT Transportation Policies and safety procedures. This policy/procedure covers both the safety and technical aspects of work involving minor fatigue damage retrofit with paint systems containing inorganic lead.

26.3.2.2 Applicable Safety Standards and Transportation Policies

The following standards, policies and procedures shall be followed when working in environments in which lead-based coating materials are removed:

- OSHA 29CFR1926.62 “Lead in Construction”
- OSHA 29CFR1910.134 “Respiratory Protection”
- OSHA 29CFR1926.59 “Hazard Communication”
- OSHA 29CFR1910.1000 “Air Contaminants”
- OSHA 29CFR1910.146 “Permit-required Confined Spaces”
- *NDOT Employee Safety Manual*
- NDOT Policies and Procedures Manual TP 1-6-26, “Respiratory Protection”
- NDOT Policies and Procedures Manual TP 1-6-28, “Confined Spaces”
- NDOT Policies and Procedures Manual TP 1-7-2, “Chemical Hazard Communication”

26.3.2.3 Required Training and Testing

Prior to performing any work that may fall under this policy, all affected employees must have completed the following training and testing:

- NDOT Respiratory Protection Program Training.

- Respirator Fit-Testing.

- Biennial physical exam, in accordance with NDOT Medical Surveillance Program.

- Lead awareness training, as per CFR1926.59 (h) “Employee Information and Training,” CFR1926.62(1) “Employee Information and Training” and CFR1910.52(1) “Employee Information and Training.”

- Baseline blood sampling and analysis for blood lead (PbB).

- Confined Space Awareness Training, as outlined in NDOT TP 1-6-28.

26.3.2.4 Required Personal Protective Equipment (PPE)

The following PPE must be worn at all times during fatigue damage retrofit work involving coatings containing lead or other heavy metals:

- Half or full-face respirators with organic vapor, acid gas and HEPA filter cartridges.
- Disposable over-garments, including hooded-coveralls, booties and gloves. Garbage bags will be on-site for collection of used garments for proper disposal.
- Safety goggles or face shields (if full-face respirators are not used).
- Work shoes or boots (no tennis shoes).
- In lieu of wearing gloves, hands must be washed with soap and water immediately after completing the procedure, or wiped with towelettes until soap and water are available. At no time shall food be consumed or tobacco, gum or cosmetics be used unless hands are washed with soap and water.

26.3.2.5 Confined Space Work

Any retrofit that must be performed within a confined space (e.g., a steel tub girder) as defined in the OSHA regulations must be conducted in accordance with NDOT Policies and Procedures Manual TP 1-6-28, "Confined Spaces." The introduction of any chemical or material (e.g, Carbomastic 15 Low Odor epoxy mastic) into a confined space that may create a hazard must be reviewed by the on-site NDOT supervisor or engineer before any employee is allowed to enter. Additional equipment, including but not limited to communications equipment, air sampling monitors, safety harnesses and tethers, may be required. The on-site supervisor or engineer will ensure that the proper procedures are observed when confined spaces must be entered.

26.3.2.6 Inspection/Retrofit Equipment

Equipment to be used to perform retrofits of fatigue damage typically consists of the following:

- "Peel-Away" brand lead-based-paint remover and neutralizer, or equivalent;
- HEPA vacuum-blast abrasive paint removal system;
- "Hougen Rotabroach" magnetic drill, or equivalent;
- die grinder or hand held flat grinder;
- hand scrapers;
- non-destructive testing equipment including, but not limited to, Magnetic Particle, Ultrasonic and/or Liquid (Dye) Penetrant equipment; and
- conventional hand tools, as commonly used in bridge inspection.

26.3.2.7 Engineering Controls

The use of "Peel-Away" brand lead-based-paint remover and/or the HEPA vacuum-blast abrasive paint removal system shall be used whenever possible to minimize the release of airborne lead or other heavy metal particles into the environment or the breathing zone of employees.

26.3.2.8 Contractor Safety (Multi-Employer Worksite)

On-site personnel that are not NDOT Structures Division Bridge Inspection/NDT Squad employees must not be allowed to be exposed to airborne lead or other heavy metals unless:

- they comply with all aspects of this program, and
- they are approved to enter the work zone by the on-site NDOT supervisor or engineer.

26.3.2.9 Retrofit Procedures

The following procedures shall be used when completing retrofits of minor fatigue damage when coatings containing lead or other heavy metals are present:

26.3.2.9.1 *Preparation*

1. Prior to the start of any retrofit work, a member of the work group or other on-site person who meets the qualification of a competent person [as per CFR 1926.32(f)], and qualified person [as per 1926.32(m)], shall identify any material hazard present. This shall involve a review of the bridge plans and a review of all painting materials listed in the original construction documents.

2. Consultation with a Certified Industrial Hygienist (CIH) will occur on an as-needed basis and may include evaluation of work-space atmospheric concentrations of lead or other heavy metals, including air sampling in the employee's Personal Breathing Zone. The NDOT Environmental Services Division will provide sampling and testing of coatings on an as-needed basis and maintain analytical data on bridges tested.

3. Prior to the start of work, Material Safety Data Sheets (MSDS) for any products to be used in the repair procedure shall be made available to, and reviewed by, all parties involved in the repair effort. MSDS shall also be available on-site.

4. Adequate traffic control and access to the work site shall be provided by NDOT District personnel, as per Part 6 of the *Manual on Uniform Traffic Control Devices* (MUTCD).

5. Personnel performing the retrofits must have completed all applicable testing and training and must wear all required PPE, as outlined above.

26.3.2.9.2 *Coating Removal*

1. Areas of fatigue damage shall be identified and surface contaminants (e.g., dirt, cobwebs) removed.

2. When a lead-based-paint removal paste is used, the product shall be applied to the member following the manufacturer's directions. The product shall be applied at the tip(s) of each crack, if drilling methods will be used, or to the entire length of each crack, if grinding methods will be used. If multiple cracks will be repaired, paint remover should be applied to all cracks before subsequent repair work is completed, to allow time for the remover to work (dwell time). Following an adequate dwell period, the loosened coating/paint remover paste residue shall be carefully scraped away and collected in approved sampling containers for delivery, using chain-of-custody procedures, to a State

of Nevada certified laboratory for analysis to evaluate disposal options. Remove paint down to bare metal.

3. If HEPA vacuum blast equipment is used, the manufacturer's directions should be followed that result in coating removal down to bare metal in the areas surrounding either the tip(s) of each crack if drilling methods will be used, or the entire length of each crack if grinding methods will be used. All abrasive/paint residues from the operation shall be collected in approved sampling containers for delivery, using chain-of-custody procedures, to a State of Nevada certified laboratory for analysis to evaluate disposal options.

26.3.2.9.3 *Crack Examination*

1. Once the surface coating has been removed, the tip(s) of each crack shall be identified using NDT methods including, but not limited to, Magnetic Particle Testing (MT), Ultrasonic Testing (UT) or Dye Penetrant Testing (PT). See Section 26.3.1. Personnel conducting the NDT testing shall be qualified to perform such tests.

2. Once the NDT test has been performed, the NDT inspector and/or the engineer on-site shall identify the tip(s) of each crack.

26.3.2.9.4 *Drilling Procedures*

1. If crack stabilization by the hole-drilling method is selected, a ¼-in diameter pilot hole shall first be drilled at the tip of each crack. This hole serves to center the cutter used by the magnetic drill.

2. Using the magnetic drill, drill a hole completely through the damaged member, which intercepts the crack tip. Situate the hole such that one-half of the diameter of the cutter extends past the detectable tip of the crack to cover undetectable crack propagation. Use dry lubricant on the cutter where practical. Cutter size to be used may vary between ½-in and 1-in diameter, depending upon the size of the crack, location and stress level.

3. When multiple, closely spaced cracks exist, position pilot holes to intercept multiple crack tips, as directed by the engineer or NDT Inspector.

4. All drilled holes that overlap or exhibit less than ¼ in of material between the edges of adjacent holes shall be elongated by grinding, as directed by the engineer or NDT Inspector.

5. Collect drilling chips, clean each drill hole, and remove all traces of cutter lubricant adjacent to the drill hole(s).

6. Examine drill core(s) and interior of each drill hole, using appropriate NDT and visual methods, to ensure that each crack tip was properly intercepted. Improperly positioned holes may need to be slightly enlarged to properly intercept the crack tip, as directed by the engineer or NDT inspector.

7. Properly placed holes shall be polished with emery cloth, removing all gouges, nicks and burrs.

8. Satisfactory retrofit holes not receiving high-strength bolts shall be coated using Carbomastic 15 Low Odor epoxy mastic or equivalent.

9. Fill drill holes with high-strength bolts conforming to ASTM A325 when space permits. The bolt's clamping force induces a compressive stress in the area of the drill hole that resists further crack growth. The diameter of the fasteners should be 1/16 in smaller than the diameter of the drill hole. Tighten bolts using the "turn-of-the-nut" method. Fasteners shall be coated with Carbomastic 15 Low Odor epoxy mastic, or equivalent, following installation.

26.3.2.9.5 *Grinding Procedures*

1. If grinding retrofits are specified, a die grinder or hand-held flat grinder shall be used to remove all visible portions of the crack.

2. The individual performing the grinding procedure has the greatest potential for exposure. All other personnel should remain at least 6 ft away.

3. Upon completion of the grinding, the recess shall be examined, using appropriate NDT and visual methods, to ensure that all traces of the crack were completely removed.

4. After the crack has been completely removed, the recess shall be polished with emery cloth, removing all gouges, nicks and burrs.

5. Satisfactory repairs shall be coated using Carbomastic 15 Low Odor epoxy mastic or equivalent.

26.3.3 Concrete Bridge Deck Condition Surveys

The NDT Squad conducts the following concrete bridge deck condition tests:

- Delamination Sounding (see Section 22.4.2.3)
- Chloride Analysis (see Section 22.4.2.4)
- Pachometer Readings (see Section 22.4.2.5)
- Ground Penetrating Radar (see Section 22.4.2.6)
- Coring (see Section 22.4.2.8)

Each condition test method is discussed in the referenced section, including a test description (with any ASTM or other specification reference), the test purpose, when to use the test and an analysis of the test results.

Chapter 27

RESERVED

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Chapter 28

NEVADA BRIDGE INSPECTION PROGRAM

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Chapter 28
NEVADA BRIDGE INSPECTION PROGRAM

Chapter 28 discusses the National Bridge Inspection Standards (NBIS) and NDOT's implementation through its Bridge Inspection Program.

28.1 FEDERAL BRIDGE INSPECTION PROGRAM

28.1.1 Importance of Bridge Inspections

The State of Nevada contains approximately 1850 bridges on its public roads and streets; approximately 1050 of these are on the State highway system. In general, bridges are designed and constructed with the intent of providing a large margin of safety and a long service life (75 years) for the traveling public. This is accomplished through the application of stringent design criteria and construction specifications. Nevertheless, all structural elements deteriorate over time, sometimes prematurely, and, if left unchecked, will eventually present a hazard to bridge users. Therefore, a systematic program of periodic bridge inspections is necessary to evaluate condition and functionality, to detect structural problems and to extend the useful life of the bridge. Of course, the program must be developed recognizing economic constraints.

28.1.2 National Bridge Inspection Standards (NBIS)

The collapse of the Silver Bridge over the Ohio River in 1967 prompted the United States Congress to enact legislation requiring the establishment of the National Bridge Inspection Standards (NBIS), creating a nationwide bridge inspection and inventory program. The Federal Highway Administration has promulgated regulations to establish the specific criteria that each State transportation department must meet; i.e., the State DOTs are the administrators of the NBIS for all bridges located within the geographic boundaries of the State. For convenience, Appendix 28A duplicates the regulations from 23CFR Part 650, Subpart C "National Bridge Inspection Standards."

28.1.2.1 Primary Constituents

The following summarizes the primary constituents of the National Bridge Inspection Standards:

- NBIS requires the periodic inspection of all "bridges" (which are defined as having a roadway centerline length of greater than 20 ft) located on all highway facilities open to the public.
- NBIS does not mandate the inspection of pedestrian bridges, railroad bridges, privately owned bridges, or those bridges or culverts having a roadway centerline length of 20 ft or less. Further, the NBIS does not mandate the inspection of sign structures, traffic signals, luminaire supports, etc.
- NBIS establishes the basic requirements for each component of a State DOT Bridge Inspection Program:

+ Inspection Frequency/Procedures/Reports,
+ Qualifications of Personnel, and
+ Maintenance of the State's Bridge Inventory.

28.1.2.2 Operational Elements

The following presents a brief discussion on the operational elements of the NBIS:

1. Frequency of Inspections. The basic NBIS requirement is that each bridge be inspected at regular intervals not to exceed 24 months. Examples of structures requiring more frequent inspections may include:

   - unique structure types;
   - those with details that have no performance history;
   - those with potential foundation or scour problems;
   - non-redundant structures;
   - steel structures with fatigue-prone details;
   - steel structures with cracks or crack repairs;
   - structures experiencing heavy traffic loadings;
   - old structures; and
   - structures with known, significant structural problems.

2. Qualifications of Personnel. One of the most important elements of a State DOT bridge inspection program is the qualifications of its inspection personnel. This includes both the individual in charge of the overall organization and the field inspection personnel. §650.309 of the NBIS lists the minimum requirements for all bridge inspection personnel. In addition to education and experience requirements, the field inspectors must be physically fit and must have basic language, mathematical and mechanical skills.

3. Inspection Procedures and Reports. Each State must have a systematic strategy for conducting field inspections and reporting their findings. It must be clear to the inspection team which structural elements to check and what to look for. The bridge inspection report should accurately and clearly record all findings from the inspections and should include photographs of the overall structure and any significant defects.

4. Records. Each State must have a systematic means of entering, storing and retrieving bridge inspection data. The records should contain a full history of the structure including:

   - all inspections,
   - recommendations for maintenance or repair work,
   - any maintenance or repair work performed,
   - structure ratings,
   - calculations,
   - the Structure Inventory and Appraisal (SI&A) data, and
   - communications.

5. Load Ratings. All bridges must be load rated to determine their structural capacity. This includes the calculation of both the Operating and Inventory Ratings (see Section 28.3 for definitions). The ratings provide an indication of the bridge's safe load-carrying capacity. This information also assists in the determination of necessary load restriction

posting, the issuance of special overload permits, and the scheduling for rehabilitation or replacement.

28.1.3 National References

Several references have been developed at the national level for the implementation of the NBIS. The following briefly describes the most important references, and the discussion includes a brief statement on its status and application within NDOT.

28.1.3.1 *AASHTO Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*

The AASHTO *Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* serves as a standard and provides uniformity in the procedures and policies for determining the physical condition, maintenance needs and load capacity of highway bridges in the United States. This publication assists bridge owners by establishing inspection procedures and load rating practices that meet the National Bridge Inspection Standards (NBIS). The load rating procedures are based upon the LRFR methodology.

AASHTO has approved but not yet published the *Manual for Bridge Evaluation* (MBE). The MBE is an updated version of the *AASHTO Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* incorporating the LRFR methodology plus the traditional ASR and LFR methodologies.

28.1.3.2 FHWA *Bridge Inspector's Reference Manual* (BIRM)

The FHWA *BIRM* provides guidelines for training bridge inspectors. The *BIRM* presents a fundamental discussion on the inspection and evaluation of specific bridge components, and it discusses field inspection procedures and reporting requirements. In addition, the *BIRM* discusses the basic qualifications of bridge inspectors and field safety procedures.

The *BIRM* is used as a primary reference in the comprehensive training program on bridge inspection presented by the National Highway Institute (NHI). NDOT uses the *BIRM* as a primary field and office reference for NDOT's bridge inspectors.

28.1.3.3 FHWA *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* (Coding Guide)

The FHWA *Coding Guide* has been prepared for use by the State DOTs to record and code the specific data items that are stored in the National Bridge Inventory (NBI) database. The NBI data is used to prepare legislatively required reports to Congress. The *Coding Guide* also provides the data necessary for FHWA and the Military Traffic Management Command to identify and classify the Strategic Highway Corridor Network and its connectors for national defense.

FHWA mandates that bridge inventory data be submitted to the Agency in a standardized format as required by NBIS. Therefore, NDOT has adopted the conventions, terminology, etc., within the FHWA *Coding Guide* for the collection, recording and reporting of bridge inspection data.

28.1.3.4 AASHTO *Guide for Commonly Recognized (CoRe) Structural Elements* (Guide)

The AASHTO *Guide* provides a description of structural elements that are commonly used in highway bridge construction and evaluated during bridge inspections. These elements are termed "Commonly Recognized" (CoRe) structural elements because of their nationwide recognition and use. Although the element descriptions were originally developed for PONTIS (AASHTOWare bridge management software used by NDOT), these descriptions are not considered unique to PONTIS; rather, they provide a uniform basis for data collection for any bridge management system and enable the sharing of data among States. NDOT uses the CoRe element descriptions in setting up data collection procedures for the NDOT Bridge Management System. See Chapter 29.

28.2 NEVADA BRIDGE INSPECTION PROGRAM

28.2.1 General

28.2.1.1 Compliance with NBIS

Section 28.2 describes the Nevada Bridge Inspection Program, which is NDOT's implementation of the National Bridge Inspection Standards (NBIS) for all public bridges in the State of Nevada not owned by Federal agencies. In general, the Nevada Bridge Inspection Program meets or exceeds the requirements of the NBIS. The following sections discuss specific procedures and criteria adopted by NDOT for its implementation of the Nevada Bridge Inspection Program.

28.2.1.2 Coding Bridge Inspection Data

FHWA and AASHTO have developed rating systems to aid in the inspection of bridges. The two primary rating systems currently in use are the National Bridge Inventory (NBI) rating system and the PONTIS (or element-level) rating system. Both rating systems are intended to create uniformity for rating the structural condition of a bridge. Each rating system relates the material distress found at the bridge to its effect on the structure's strength and safety. NDOT requires that the bridge inspector complete both an NBI and PONTIS inspection of each bridge inspected.

A PONTIS element-level inspection identifies each bridge component as a separate element, based not only upon function but also material type, and it evaluates each element by sub-dividing its total quantity into different "Condition States," or states of physical deterioration or damage. The AASHTO *Guide for Commonly Recognized (CoRe) Structural Elements* describes the PONTIS element-based rating system.

In a PONTIS inspection, each bridge element is assigned an element number and a standard description. The total quantity for an element is then sub-divided among up to 5 available Condition States, 1 to 5, where Condition State 1 indicates the best possible condition. Additional elements called "Smart Flags" are used to track event-driven damage, such as traffic impacts or fatigue cracks that are unique to a specific bridge.

In contrast, an NBI inspection evaluates each bridge component based only on function, and assigns a single "Condition Rating" representing the condition of the component type as a whole, regardless of quantity. Condition Rating codes range from 9 to 0, where 9 is the best rating possible ("excellent condition") and 0 is the worst rating possible ("failed condition"). A narrative description with quantities is used to describe the condition of the functional group.

The NBI inspection also includes the assignment of "Overall (condition) Ratings," representing the overall condition of the Deck, Superstructure, Substructure and Culvert. These specific condition ratings are incorporated into the Structure Inventory and Appraisal (SI&A) sheet, which serves as a comprehensive listing of all NBI data for any given bridge. The Overall Ratings are used, in part, to determine the Sufficiency Rating (SR) for the bridge, which is a numerical indicator of the bridge's sufficiency to remain in service.

The PONTIS rating system incorporates a database that, over time, can be used to estimate deterioration rates based on the material and the bridge environment. This allows bridge owners to schedule preventive and corrective actions more uniformly. In this way, NDOT can make informed decisions to optimize the expenditure of funds to prioritize funds, when to take action, and what type of action to take. See Chapter 29 for more discussion on bridge management.

28.2.2 Responsibilities/Qualifications

28.2.2.1 NDOT Structures Division

In compliance with §650.307 of the NBIS, the Inventory/Inspection Section is responsible for the Nevada Bridge Inspection Program. Section 1.3.2 briefly summarizes the functional responsibilities of each Unit within the Section. Section 28.2.2 elaborates on the responsibilities and qualifications of the NDOT staff within the Inventory/Inspection Section.

28.2.2.2 Assistant Chief Structures Engineer – Inventory/Inspection

The Assistant Chief Structures Engineer – Inventory/Inspection (ACSE – I/I) serves as the Program Manager (PM) for implementation of the National Bridge Inspection Standards. §650.305 defines the Program Manager as:

> *The individual in charge of the program that has been assigned or delegated the duties and responsibilities for bridge inspection, reporting and inventory. The program manager provides overall leadership and is available to inspection team leaders to provide guidance.*

In addition, the PM assists applicable NDOT staff in determining any maintenance or repair actions that are appropriate. Decisions on load posting and bridge closures require the approval of the PM.

§650.309(a) identifies the minimum qualifications of the PM. The PM must have a sound background in structure inspections, rehabilitation and maintenance to be an effective manager. On occasion, specialized knowledge and skills in fields such as structural design, construction, mechanical systems, electrical systems, soils, construction materials and emergency repair techniques will be required.

The PM is the primary liaison between FHWA and NDOT, and the PM is responsible for ensuring that NDOT complies with Federal directives for structure inspection and maintenance. This includes ensuring that all structures are inspected at the proper intervals and that NDOT files remain up-to-date and accurate. The Program Manager has overall responsibility for personnel supervision; scheduling structure inspections and maintenance; and scheduling the use of NDOT-owned specialized equipment. The responsibilities of the PM also include:

- Overseeing quality assurance reviews.
- Overseeing coordination with Federal, State and local governmental agencies.
- Monitoring an in-depth inspection program for structures with fracture critical members, underwater members, or unique or special features requiring additional attention during inspection to assure the safety of such structures.
- Oversight of coordination with the Railroad Companies operating in Nevada to inspect NDOT-owned bridges over railroads.

- Confirming that load-posted structures receive interim inspections as required by Federal and State laws, rules and policy.

- Developing, monitoring and updating training programs for State and private consultant inspectors in structure inspection, maintenance and repair techniques.

- Retaining the services of private consultants or contractors to supplement NDOT staff, as needed, to perform specialized inspection, testing or repair techniques.

- Analyzing Federal and State legislation, administrative rules, and national and industry standards for incorporation into NDOT programs and policies.

- As appropriate, recommending revisions to State of Nevada laws and participating in the development of new legislation.

- Being responsible for prompt, decisive and effective responses to emergencies (e.g., earthquakes, major bridge damage, bridge failures).

- Developing and administering the Nevada Bridge Inspection Program budget for the Inventory/Inspection Section.

28.2.2.3 Principal Structures Engineer – Inspection

The Principal Structures Engineer – Inspection (PSE) serves as the Bridge Inspection Squad's supervisor and Assistant Program Manager. The Principal Structures Engineer – Inspection assists the Assistant Chief Structures Engineer – Inventory/Inspection in fulfilling the responsibilities of the Program Manager including the following duties:

- Providing the day-to-day supervisory management for the Nevada Bridge Inspection Program.

- Managing the work of all consultants used to perform bridge inspections.

- Reviewing and approving all Bridge Inspection Reports.

- Coordinating with Federal, State and local governmental agencies.

- Directing an in-depth inspection program for structures with fracture critical members, underwater members, or unique or special features requiring additional attention during inspection to assure the safety of such structures.

- Coordinating with the Railroad Companies operating in Nevada to inspect NDOT-owned bridges over railroads.

- Directing the interim inspections of load-posted structures as required by Federal and State laws, rules and policies.

- Managing a technology transfer program for NDOT and consultant inspectors for the inspection of bridges.

- Providing training for personnel on proper access, equipment operation and safety procedures.

28.2.2.4 Professional Engineer

28.2.2.4.1 *Bridge Inspection Squad*

The Professional Engineer in the Bridge Inspection Squad serves as the Inspection Team Leader (TL) for the Nevada Bridge Inspection Program. §650.305 of the NBIS defines the Inspection Team Leader as the:

> *Individual in charge of an inspection team responsible for planning, preparing, and performing field inspection of the bridge.*

§650.309(b) of the NBIS identifies the qualifications of the Inspection Team Leader; see Appendix 28A. A TL has the authority to sign and process the Bridge Inspection Reports.

The TL must be at the structure site at all times during each field inspection. The TL shall be proficient with the NBIS, relevant FHWA and AASHTO publications, and this *Manual*. The TL should have a strong background in structural engineering, structure behavior trends, and bridge maintenance and rehabilitation techniques. The TL is also responsible for the general safety of the work site. Safety items include planning and monitoring any required traffic control and ensuring that each team member complies with all NDOT safety procedures, proper use of access equipment, etc.

28.2.2.4.2 *Load Rating/Over-Dimensional/Overweight Permitting Squad*

The Professional Engineer in the Load Rating/Over-Dimensional/Overweight Permitting Squad is responsible for calculating bridge Inventory and Operating Ratings, recommending load posting for existing bridges, and analyzing over-weight and/or over-dimensional vehicles for operating permit purposes. See Section 28.3.

28.2.2.5 Staff III Associate Engineer

28.2.2.5.1 *Bridge Inventory Management Squad*

The Staff III Associate Engineer in the Bridge Inventory Management Squad serves as the Squad's supervisor. The responsibilities of this position include:

- Act as the liaison with non-State bridge owners to receive bridge plans, ascertain new bridge locations, etc.
- Update Statewide bridge location maps.
- Conduct Inventory Inspections of new bridges, Statewide.
- Receive completed Bridge Inspection Reports from both in-house and consultant staff; disseminate reports to owners and file NDOT copies.
- Manage National Bridge Inventory (NBI) data, and disseminate to Structures Division personnel, consultant inspectors and the Federal Highway Administration.

The Staff III Associate Engineer in the Bridge Inventory Management Squad also serves as an Inspection Team Leader (TL) for the Nevada Bridge Inspection Program. See Section 28.2.2.4.1 for TL qualifications and responsibilities.

28.2.2.5.2 *Non Destructive Testing (NDT) Squad*

The Staff III Associate Engineer in the Non Destructive Testing Squad serves as the NDT Squad supervisor. The responsibilities of the NDT Squad with respect to the Nevada Bridge Inspection Program include providing NDT inspection of structural steel. See Chapter 26.

The Staff III Associate Engineer in the NDT Section also serves as an Assistant Inspector (AI) in the Nevada Bridge Inspection Program. When serving as the AI, the Staff III Assistant Engineer – NDT assists the TL in the field. It is expected that this individual, at a minimum, is familiar with this *Manual* and has a competency level sufficient to follow the directives of the TL.

28.2.2.6 Staff II Associate Engineer – Inventory Management

The Staff II Associate Engineer in the Bridge Inventory Management Squad assists the Staff III Associate Engineer – Inventory Management Squad in fulfilling the responsibilities of the Squad. See Section 28.2.2.5.1.

The Staff II Associate Engineer also services as an AI. See Section 28.2.2.5.2 for the AI responsibilities.

28.2.2.7 Staff I Associate Engineer – Non-Destructive Testing Squad

The Staff I Associate Engineer in the Non-Destructive Testing Squad (NDT) serves as an NDT specialist. NDT specialists perform non-destructive testing for NDOT within the context of the Nevada Bridge Inspection Program. See Chapter 26 for non-destructive testing in more detail and other responsibilities. The Staff I Associate Engineer in the NDT Squad also serves as the AI. See Section 28.2.2.5.2 for AI responsibilities.

28.2.2.8 Special Equipment Operator III – Bridge Inspection Squad

The Special Equipment Operator III in the Bridge Inspection Squad is in charge of the transport, operation and maintenance of the bridge inspection vehicles. The Special Equipment Operator III must possess a valid Class A or Class B Commercial Drivers License in the State of Nevada.

The Special Equipment Operator III has the following responsibilities:

- coordinates access-required inspections with in-house and consultant TLs,
- arranges for traffic control from District Offices,
- oversees the operation of the bridge inspection units,
- conducts minor bridge repairs as needed,
- coordinates and performs inspection vehicle maintenance,
- manages inventory of inspection hand tools and disposable equipment/supplies, and
- provides Under-Bridge Inspection Truck operator training.

28.2.2.9 Special Equipment Operator II – Bridge Inspection Squad

The Special Equipment Operator II in the Bridge Inspection Squad assists the Special Equipment Operator III in fulfilling the requirements of the Nevada Bridge Inspection Program. See Section 28.2.2.8 for requirements.

28.2.3 District Office

Nevada is divided into three districts that administer the transportation program at the local level. Each District Office has an Assistant District Engineer – Maintenance that oversees the maintenance operations in that District. The Assistant District Engineer – Maintenance is the primary contact for coordination between the District Office and the Structures Division for the Nevada Bridge Inspection Program. Most day-to-day coordination, however, occurs with the District Bridge Maintenance Crew. District involvement in the Nevada Bridge Inspection Program includes the following:

- The Bridge Inspection Squad submits the bridge inspection schedule for bridge inspections to the District (see Section 28.2.8).
- The District arranges traffic control for all bridge inspections as required (see Section 28.2.10.2).
- The District provides assistance during the bridge inspection as requested by the Bridge Inspection Squad.
- The Bridge Inventory Management Squad submits copies of all Bridge Inspection Reports for State-owned bridges to the appropriate District Office, and identifies which bridges require maintenance.
- The District may participate in Quality Assurance Field Inspections (see Section 28.2.13).
- The District responds to "critical maintenance" findings for State-owned bridges (see Section 28.2.6.8).
- The District collaborates with the ACSE – I/I to authorize bridge closures on State routes.
- The District performs routine bridge maintenance activities identified in the Bridge Inspection Reports.
- The PSE and District coordinate their activities to respond collaboratively to emergencies (see Section 28.2.6.10).

28.2.4 Consultant Program

28.2.4.1 General

NDOT does not have the staffing necessary to achieve all Program requirements and, therefore, uses the services of consultants to supplement in-house staff. When work cannot be performed consistent with the schedule for the Program, or when the work requires specialized professional or technical talents not readily available within NDOT, consultants may be employed. In addition, NDOT has a vested interest in retaining a contingent of qualified consultants that are available when needed.

Consultants are considered an extension of the NDOT staff for the implementation of the Nevada Bridge Inspection Program. During a field inspection, consultant employees are expected to represent NDOT in their interface with the public, and they must comply with all applicable NDOT requirements (e.g., wearing NDOT attire).

28.2.4.2 Special Uses

In addition to bridge inspections, NDOT uses consultants for the following specialized elements of its Bridge Inspection Program:

- load rating analyses of existing bridges;
- underwater inspections that require diving;
- overhead sign, signal and high-mast lighting inspection; and
- bridge scour evaluation and developing Plans of Action.

28.2.4.3 Operational Issues

In general, consultants are responsible for complying with all NDOT requirements in the implementation of the Nevada Bridge Inspection Program. The following discusses a few specific issues:

1. Registered Professional Engineer. A registered professional civil or structural engineer in the State of Nevada is required to be in overall management of the consultant's project. This is an NDOT requirement and not an NBIS requirement.

2. Team Leader (TL). NDOT does not mandate that the consultant TL be a Registered Professional Engineer. NDOT and the consultant must mutually agree on the acceptability of the proposed TL. The consultant must submit a resume for each proposed TL. The Principal Structures Engineer – Inspection will devote the time deemed necessary with each proposed TL to evaluate the TL's credentials, experience and performance.

3. Underwater Divers. The consultant is required to provide only certified Commercial Divers. Underwater TLs must meet the TL qualifications in Section 28.2.2.4.1. All diving operations must be conducted in compliance with OSHA 29 CFR 1910 Subpart T - Commercial Diving Operations (including OSHA Directive CPL 02-00-143) and Association of Diving Contractors International "CONSENSUS STANDARDS For Commercial Diving Operations."

4. Scheduling. NDOT provides a list of bridges for consultant inspection, and the consultant submits a schedule of inspections to NDOT for review and approval.

5. Coordination. Consultants are required to participate in the coordination with entities external to NDOT (e.g., Railroad Companies, State and local entities).

6. Field Inspections. The consultant Inspection Team and NDOT representatives (AIs, Special Equipment Operators, District maintenance personnel) work in tandem to perform the field inspection. NDOT operates in a support capacity (e.g., provide traffic control, provide special equipment).

7. Submission of Reports. Consultant Bridge Inspection Reports must be signed and sealed by a Registered Professional Engineer prior to submittal to NDOT.

8. NDOT QA Review. The Principal Structures Engineer – Inspection or his designated representative will review all Bridge Inspection Reports submitted by consultants for completeness and accuracy. The nature of the NDOT review is considered a Quality Assurance review, not an "approval"; i.e., the burden of responsibility for technical

content remains with the consultant. However, the consultant is required to respond to any written comments from NDOT.

28.2.5 NDOT References

NDOT has prepared the following references to assist in the implementation of its Bridge Inspection Program:

1. *NDOT Bridge Inspection Manual*. This reference provides instructions and guidance to all bridge inspection personnel on NDOT bridge inspection policies and procedures. Topics presented include:

    - Bridge Inspection Report format,
    - Condition Rating application,
    - rating of NBI data items, and
    - Maintenance Report coding instructions.

2. *NDOT PONTIS Coding Guide*. This reference provides instructions and guidance on NDOT procedures for conducting PONTIS element-level bridge inspections. Topics presented include:

    - unit of measurement conventions,
    - girder tabulation conventions,
    - condition state assignment conventions,
    - primer for PONTIS CoRe elements and Smart Flags, and
    - comprehensive listing of all PONTIS Elements used in Nevada.

28.2.6 Types of Inspections

28.2.6.1 General

The following identifies a few of the basic parameters for bridge inspections:

1. Inspection Team Composition. All Teams must have an Inspection Team Leader (TL). The minimum crew size is two, including the TL.

2. Inspection References. §650.313(a) of the NBIS requires that each State DOT:

    > *Inspect each bridge in accordance with the inspection procedures in the AASHTO Manual (i.e., the Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges).*

    In addition, NDOT uses the FHWA *Bridge Inspector's Reference Manual* (BIRM) and the FHWA *Coding Guide* as primary references for bridge inspections.

3. Units of Measurement. All PONTIS Element data and Structure Inventory and Appraisal (SI&A) data are reported in SI (metric) units of measurement (see Chapter 29). All other components of the NBIP are based on US Customary units of measurement.

28.2.6.2 Inventory Inspections

§650.305 of the NBIS defines the Inventory (or Initial) Inspection as:

> *The first inspection of a bridge as it becomes a part of the bridge file to provide all Structure Inventory and Appraisal (SI&A) data and other relevant data and to determine baseline structural conditions.*

An Inventory Inspection is the baseline inspection that must be completed for every new structure before it can be entered into the Nevada Bridge Inventory. An Inventory Inspection is a fully documented inspection, using the bridge plans, to determine basic data for a specific structure for entry into the file. The bridge inspector conducting the Inventory Inspection must complete a SI&A Sheet as part of the Inventory Inspection process. Data gathered for Inventory Inspections should include the following:

- an analytical determination of load capacity,
- all Structure Inventory and Appraisal (SI&A) data required by FHWA regulations,
- baseline structural conditions and quantities,
- any existing problems or locations in the structure that may have potential problems,
- the location and condition of any fracture critical members or details, and
- any recommendations for corrective action.

The Inventory Inspection must be complete and have Structure Inventory and Appraisal Data (SI&A) entered into the State Bridge Inventory within 90 days for State-owned bridges and within 180 days for all others. In addition, as part of the Inventory Inspection, bridge inspectors must evaluate the structure and identify other foreseeable types of inspections that the structure will require throughout its life. For example, a bridge spanning a waterway may in the future require an Underwater Inspection. The bridge should also be assessed for needing Fracture Critical or Complex Bridge Inspections, with Fracture Critical Members and special/complex inspection methodologies identified. Once the inspection types the structure will require have been identified, the bridge inspector should document the associated inspection frequencies as part of the Inventory Inspection. The inspector shall also document any special inspection equipment and access equipment that is needed to perform future inspections.

The Inventory Inspection shall be performed at arm’s length. Because it is a baseline inspection, all deficiencies, cracks, construction errors, alignment problems, etc., should be quantified and documented.

Inventory Inspections are also used when a structure is discovered that has never been inventoried. For example, some short-span bridges and culverts that span more than 20 ft have never been inventoried or classified as bridges when they were built. Inventory Inspections are also performed when the configuration or geometry of a structure changes (e.g., widening, lengthening, change in vertical clearance) or when structural improvements are made (e.g., rehabilitation).

28.2.6.3 Routine Inspections

§650.305 defines a Routine Inspection as:

> *Regularly scheduled inspection consisting of observations and/or measurements needed to determine the physical and functional condition of the bridge to identify any changes from initial or previously recorded conditions, and to ensure that the structure continues to satisfy present service requirements.*

Routine Inspections are generally conducted from the deck, ground or water level or from permanent work platforms and walkways, if present. The bridge inspector shall closely inspect the critical load-carrying members (e.g., steel and concrete girders, decks, slabs, piers, bearings, abutments) and shall more closely examine any element that appears distressed. Fatigue prone and fracture critical details or elements shall be examined with a detailed, close-up (arm's length) inspection.

For some bridges, it may be necessary to schedule the use of special equipment (lift or under bridge inspection vehicle) to gain the needed access to perform a Routine Inspection. For example:

- bridges with one or more spans that are inaccessible due to stream characteristics; and
- bridges that are more than 30 ft above highways, railroads, etc.

For Routine Inspections, inspecting underwater portions of the substructure is limited to observations during low-flow periods and/or probing for signs of undermining. If substructure units are either immediately adjacent to or in the water, waterway soundings should be documented to detect changes in the channel. Substructure elements continuously submerged in greater than 3 ft of water shall be placed on the Underwater Inspection bridge list and applicable SI&A data coded. Document any structural changes or deterioration that could affect previously recorded bridge load ratings in the Report. See Section 28.2.6.6 for a discussion on mandatory requirements for Underwater Inspections.

During a Routine Inspection, the bridge inspector should evaluate traffic and pedestrian safety features in addition to structural items. Provide special attention to the condition of parapets, railings, pedestrian fencing, guardrail, sidewalks, etc. The following are examples of conditions that may warrant documentation in the Bridge Inspection Report:

- tripping hazards, severe approach roadway settlements or large spalls on sidewalks;
- rebar protruding from decks, walks or parapets;
- loose, missing or damaged railings or parapets;
- missing or damaged guardrail;
- loose concrete that could fall onto the traveled way, sidewalk, waterway or railroad; and
- any other condition that the inspector perceives as a threat to public safety.

28.2.6.4 In-Depth Inspections

28.2.6.4.1 *General*

§650.305 defines an In-Depth Inspection as:

> *A close-up inspection of one or more members above or below the water level to identify any deficiencies not readily detectible using routine Inspection procedures; hands-on inspection may be necessary at some locations.*

The PSE shall determine the need for In-Depth Inspections. Conditions that prompt an In-Depth Inspection are often identified during a Routine Inspection. These include:

- need for specialized access,
- need for special inspection/testing techniques and equipment, and
- need for increased inspection of an element.

28.2.6.4.2 *Description*

An In-Depth Inspection is a hands-on, close-up visual inspection that often requires special access equipment. Each element under investigation should be within arm's reach of the inspector. Non-destructive field tests and/or other material tests may be required. The inspection may include a recommendation for a load rating to assess the residual capacity of damaged or deteriorated members, depending on the extent of the damage or deterioration. Non-destructive load tests may be conducted to assist in determining a safe bridge load-carrying capacity.

The visual examination should reveal information including but not limited to:

- distortion, crippling and buckling of members;
- spalling and cracking of concrete;
- decaying, splitting and physical attack of timber;
- corrosion and cracking of steel;
- collision damage;
- paint or finish and bearing failures; and
- joint failures.

In-depth inspections may also consist of:

- sounding of concrete elements to determine the limits of delamination/deterioration;
- sounding and probing/drilling of timber elements to determine the limits of internal deterioration, rot and decay;
- connection inspections (bolts, rivets and welds) to identify failing welds/rivets and loose/failing bolts;
- section loss measurements (as practical) for steel elements; and
- inspection of bearings, paints or finishes and other miscellaneous structural elements.

Where loose bolts are found during connection inspections, these bolts shall be tightened in non-critical areas (e.g., cross-frames) and shall be marked for replacement in critical areas (e.g., girder splices).

Thoroughly document the activities, procedures and findings of In-Depth Inspections with the appropriate photographs, a location plan of deficiencies, test results, measurements and a written report. Any changes in the condition of the structure should be entered into the Bridge Inspection Report, and the Report shall also contain maintenance recommendations. If a bridge element condition is sufficiently severe, the In-Depth Inspection data can be used to develop rehabilitation plans for the bridge.

28.2.6.5 Special Inspections

A Special Inspection is performed when a bridge requires more frequent inspections than is provided by the Routine Inspection cycle. This is an inspection scheduled at the discretion of the PSE. A Special Inspection is typically used to monitor a specific known or suspected deficiency (e.g., foundation settlement, scour, member conditions and the public's use of a load-posted bridge) sufficiently severe to warrant heightened scrutiny. Other examples may include actively settling or rotating substructures, advanced section loss, and structures with any NBI Item 58 through 62 that has a rating of 4 or less.

The bridge inspector must make sufficient measurements and observations to evaluate the structure's physical and functional conditions, and denote changes in the known or suspected deficiency. The results of the Special Inspection should be documented in the Bridge Inspection Report. If the deficiency has become more severe, it may be necessary to reevaluate the structure's load rating.

The following provides guidance on the frequency of performing Special Inspections:

1. Load Posted Bridges. Any bridge not capable of carrying State legal loads requires inspection at least once every twelve months.

2. Bridges with an NBI Rating of 4 or Less. All bridges with an NBI rating of 4 or less for the deck, superstructure or substructure require inspection at least once every twelve months.

3. Special Cases. Bridges having advanced deterioration and/or unusual movement will be inspected at a frequency determined by the PSE.

28.2.6.6 Underwater Inspections

§650.305 defines an Underwater Inspection as:

> *Inspection of the underwater portion of a bridge substructure and the surrounding channel, which cannot be inspected visually at low water by wading or probing, generally requiring diving or other appropriate techniques.*

Inspectors can observe structural conditions above water well in advance of failure. However, significant underwater structural conditions cannot be readily observed above water until the defect has progressed to where distress is evident.

In general, an Underwater Inspection is required if, during a Routine Inspection, water conditions exist at the structure that prohibit access to all portions of an element by visual or tactile means that would ensure a level of certainty. See §650.313(e)(2) of the NBIS. However, many of Nevada's bridges spanning "waterways" are over dry land during some portion of each year. These structures are typically inspected during these "low-flow/no-flow" periods (i.e., where the water depth is 3 ft or less) and, therefore, do not require a true Underwater Inspection. Therefore, for those structures passing over waterways with substructure components continuously submerged in water equal to or greater than 3 ft, the following inspection procedures will apply:

1. Regulation and Certification. Underwater diving operations shall be conducted in compliance with OSHA 29 CFR 1910 Subpart T – Commercial Diving Operations (including OSHA Directive CPL-02-00-143) and Association of Diving Contractors International (ADCI) Consensus Standards for Commercial Diving Operations. All diving team personnel shall be commercial divers, and at least one team member shall be trained as an NBIS Inspection Team Leader (TL).

2. Extent of Inspections. Inspections shall consist of a visual/tactile examination of all structural elements and a scour evaluation using mechanical (probing rods, rules, etc.)

and/or electronic methods. As practical, the section loss of submerged steel elements shall also be ascertained.

3. Results of Inspections. A complete Bridge Inspection Report (including NBI and PONTIS) shall be completed for each Underwater Inspection. The Underwater Inspection Report shall include:

- a description and location of the elements inspected,
- the procedure employed for the inspection of these elements, and
- the frequency of the inspection.

28.2.6.7 Fracture Critical Member (FCM) Inspections

28.2.6.7.1 *General*

Fracture Critical Member (FCM) Inspections shall be conducted in conjunction with the Routine Inspections of the bridge. FCMs are steel tension members or portions of steel members in tension whose failure would likely result in a total or partial bridge collapse. FCMs require more thorough and detailed inspections than the members of non-fracture critical bridges.

A FCM Inspection is a "hands-on" inspection; i.e., a visual/manual inspection made at a distance no greater than arm's length from the entire member/member component surface. Every square foot of the member/member component must be examined. The observations and/or measurements are used to determine the structural capacity of the member/member component, to identify any changes from previous FCM Inspections and to ensure that the structure continues to satisfy present safety and service requirements. Under-bridge access equipment is often required to move the inspector within arms length of the critical members. NDT examination is commonly used to examine potential deficiencies on fracture critical members.

Essentially, Fracture Critical Member (FCM) Inspections are a special type of In-Depth Inspection. The following will apply to the inspection of bridges with fracture critical members:

1. Fracture Critical Bridges. The ACSE – I/I maintains an inventory of bridges with fracture critical members. The PSE, with assistance from the Bridge Inventory/Inspection staff, is responsible for identifying structural members that are fracture critical.

2. Frequency. As required by §650.313(e)(1) of the NBIS, bridges containing fracture critical members shall be inspected in-depth at least every 24 months. A 24-month frequency will be employed until the inspection indicates a need to increase the frequency.

28.2.6.7.2 *Condition Rating of Cracked Steel Members*

The bridge inspector must devote special attention to the proper application of condition ratings to primary load-carrying steel bridge members that have experienced fatigue or weld cracking. Issues include both the consistency maintained in rating similar damage found at different bridges and the degree of conservatism used in the selection of condition ratings. The following provides guidance to aid the bridge inspector in rating fatigue damage.

To rate fatigue damage or weld cracking in primary steel superstructure elements, NDOT uses both "major" items (e.g., Items 59.2 "Stringer," 59.3 "Girder or Beams," 59.4 "Floor Beams") and the “secondary” Item 59.8 “Welds-Cracks.” Item 59.8 highlights or “weights” the magnitude and severity of cracking in steel superstructure elements or cracks in weldments. The numerical rating for this Item that best represents the extent and severity of cracking shall be selected in accordance with the Modified Condition Ratings found in Appendix A of the *NDOT Bridge Inspection Manual*. This rating may then be considered in establishing a Timing Code for repair of the cracking in the Bridge Inspection Report. In most cases, where noteworthy cracking is found, a Welds-Cracks condition rating of “5” or below is appropriate, with values of “4” or “3” being most often applied. These rating values can then be used to establish corresponding repair Timing Codes, varying from “within the next two years” to “as soon as possible.” This is justified because it is best to stabilize or repair fatigue or impact-related cracks as soon as practical, whether or not the load-carrying capability of the member itself has been reduced.

The condition rating for the “major” item(s) shall be assigned based on the Condition Rating definitions in the FHWA *Coding Guide*. These ratings shall be downgraded to an appropriate level, based upon the existing or potential threat to the member(s) imposed by the fatigue damage. For example, when fatigue cracking exists that has propagated out of a weldment and into the base metal of a member, the major item rating shall be “4” or less (with Welds-Cracks rated “3” or less). However, when cracks are found that are confined to a weldment, and the degree of threat to the base metal of the member is low (such as in a compression zone), then the major rating for the member may be raised to a value as high as “6,” and the Welds-Cracks rating used to signify the extent of cracking as discussed previously.

Inspection frequencies should be established based upon the degree of threat to the primary member. For those structures experiencing cracking that has propagated into base metal, the frequency of inspection should be set at a maximum of 6 months. For those structures where cracking is confined to a weldment, and propagation of the cracking into base metal is highly improbable, a 12-month inspection frequency should be established. Bi-ennial (24-month) inspection frequencies may be maintained only:

- where cracks exist that have previously been arrested/repaired;
- where cracks exist in secondary members (e.g., within the individual members of a cross-frame); or
- where cracks exist in non-structural welds (e.g., tack welds) where no potential for propagation outside of the weldment exists.

Communication and documentation of fatigue/weld crack findings is vitally important. Mandatory NDOT policy is that ANY cracking found in primary load-carrying steel members must be reported immediately to the ACSE – I/I. In this manner, findings can be discussed and “surprises” avoided. Further, findings should be documented in sufficient detail such that the precise location of the deficiency can be determined and the size of the deficiency “tracked” over time until a repair is completed. The use of framing sheets from the bridge plans to document fatigue damage is recommended.

28.2.6.8 Critical Maintenance Inspections

A critical maintenance inspection is a type of Special Inspection conducted to monitor significant structural damage or deterioration in a primary member. If a structure is determined to need “critical” maintenance, the structure is inspected at maximum intervals of six months for as long as the condition exists. A critical maintenance deficiency is defined as any deficiency in a primary load-carrying member requiring maintenance with a Maintenance Report Timing Code of “1” (as soon as possible) and a Criticality Code of “3” or “4” (i.e., having major structural significance).

When a critical maintenance condition is identified, the following actions occur:

1. The Inspection Team Leader (TL) notifies the Principal Structures Engineer (PSE) within 24 hours of the finding. This notification typically occurs by cell phone from the bridge site, immediately upon the finding.

2. The PSE will then immediately notify the Assistant Chief Structures Engineer – Inventory/Inspection (ACSE – I/I) of the finding. The PSE must also notify the District Office, if the bridge is State-owned, or the bridge owner, if the bridge is non-State owned.

3. The TL must prepare a Critical Maintenance Memorandum and submit it to the PSE within 24 hours of the finding. Copies of the Memorandum are sent to the:

    - Chief Structures Engineer;
    - Assistant Chief Structures Engineer – Inventory/Inspection;
    - Assistant Chief Structures Engineer – Design;
    - District, if a State owned bridge; and
    - bridge owner, if a non-State owned bridge.

4. For State owned bridges, the PSE will meet with the ACSE – I/I and Assistant Chief Structures Engineer – Design (ACSE – D) to determine the best course of action for the bridge repair. The determination of the repair strategy will depend upon the severity of the damage/deterioration, capability of NDT/District personnel and their workload, etc. Repairs can be accomplished using one of the following:

    - <u>NDT Squad</u>. The NDT Squad can perform certain repairs using bridge access equipment. These repairs can include drilling of fatigue cracks. See Section 26.3.2. The PSE is responsible for these repairs.

    - <u>District Personnel</u>. District maintenance personnel can perform certain repairs using their own or rented equipment. These repairs can include filling sinkholes in approach roadways and repairing undermined footings. The PSE is responsible for coordinating these repairs with the District.

    - <u>District Contract – Normal Procedures</u>. A conventional NDOT contract administered at the District level can be used to repair bridges in which the scope is not large but beyond the capabilities of the NDT Squad and District maintenance personnel. These repair contracts must also not be time sensitive or require special construction oversight necessitating the services of an NDOT construction crew. These repairs may include impact damage or fatigue damage of a less critical nature. The ACSE – D is responsible for coordinating these repairs with the District, which may include preparing contract drawings and specifications. The PSE remains in communication with the ACSE – D and District.

    - <u>District Contract – Emergency Procedures.</u> An emergency contract administered at the District level is used when the repairs are not large in scope, do not require special construction oversight necessitating the services of an NDOT construction crew, but are time sensitive. These repairs can include critical bridge impacts and fatigue damage. The ACSE – D is responsible for coordinating these repairs with the District, which may include preparing contract drawings and specifications. The ACSE – D also works with the District to declare the project an emergency. The PSE remains in communication with the ACSE – D and District.

- Headquarters Contract – Normal Procedures. A conventional NDOT contract administered through Headquarters can be used when the repairs are large in scope and require special construction oversight but are not time sensitive. These repairs could include less critical but pervasive fatigue damage or replacement of a non-critical bridge. The ACSE – D is responsible for programming the project, assigning a bridge design squad to prepare contract documents, and placing the project in the NDOT work program. The PSE remains in communication with the ACSE – D and District.

- Headquarters Contract – Emergency Procedures. An emergency NDOT contract administered through Headquarters can be used to perform repairs on large projects that require special construction oversight and the services of an NDOT construction crew, and the project is time sensitive. These repairs could include critical and pervasive fatigue damage, impact damage that requires closure of an important bridge, or replacement of an important bridge. The ACSE – D is responsible for programming the project, assigning a bridge design squad to prepare contract documents, and including the project in the NDOT work program. The ACSE – D also works with the District to declare the project an emergency. The PSE remains in communications with the ACSE – D and District.

After determining the best course of action, the PSE will contact the District for concurrence on the recommendation.

5. For non-State owned bridges, the PSE will request that the bridge owner submit a corrective action plan as soon as practical. The PSE will follow up with the bridge owner to determine the status of the repair, and continue to do so depending upon the severity of the damage/deterioration.

6. The status of critical-maintenance bridges is tracked using both the PONTIS database and a Critical Maintenance spreadsheet. The spreadsheet is shared between the ACSE – I/I, PSE and TL. The PSE is responsible for tracking the maintenance progress.

7. NDOT submits a Quarterly Report to FHWA detailing the status of all bridges requiring critical maintenance and those recently repaired.

8. A follow-up inspection shall be conducted after the critical maintenance repairs are complete. A Bridge Inspection Report shall be completed detailing the repair and shall include revised condition ratings justifying the removal of the bridge from Critical Maintenance status.

28.2.6.9 Complex Bridge Inspections

§650.305 of the NBIS defines a complex bridge as “movable, suspension, cable stayed, and other bridges with unusual characteristics.” §650.313(f) discusses the NBIS requirements for their inspection.

NDOT assigns its senior staff to the inspection of complex bridges. In general, these inspections require more equipment and more time and often require assistance from the Non-Destructive Testing Squad. The Bridge Inspection Report for a complex bridge shall include:

- specialized inspection procedures employed, and
- the additional/specialized inspector training/experience required.

28.2.6.10 Damage Inspection

§650.305 defines a Damage Inspection as:

> *An unscheduled inspection to assess structural damage resulting from environmental factors or human actions.*

Common examples of events that may require a Damage Inspection include earthquakes, floods, vehicular impacts, fire damage and marine vessel impacts.

The scope of the inspection must be sufficient to determine the need for emergency load restrictions or closure of the bridge to traffic and to assess the level of effort necessary to implement a repair. The level of effort for a Damage Inspection can vary significantly and depends on the severity of the damage.

The Damage Inspection is often succeeded by an In-Depth Inspection to better document the extent of damage and the urgency and scope of repairs. Follow-up activities include proper documentation, verification of field measurements and calculations and, perhaps, a more refined analysis to establish or adjust interim load restrictions.

The impetus for a Damage Inspection is often due to an “Emergency.” NDOT Policy TP 1-3-12 “Emergency Response” defines an emergency as “an unexpected or sudden event which causes serious damage which must be corrected immediately.” TP 1-3-12 also documents NDOT’s procedures for responding to an emergency. In addition, the NDOT publication “Role in Disasters” presents NDOT’s Emergency Operations Plan (EOP) pursuant to the “State of Nevada Comprehensive Emergency Management Plan” (SCEMP).

28.2.6.11 Overhead Sign/Signal/High-Mast Lighting, Bridge-Mounted Signs and Bridge-Mounted Utility Inspections

Although the NBIS does not require the inspection and inventory of these appurtenances, NDOT has developed an overhead sign, signal and high-mast lighting inventory that is separate from the Nevada Bridge Inventory. Bridge-mounted signs, luminaire supports and utilities are inspected as part of the overall bridge inspection. Inspections of these elements are visual and, where appropriate, NDT methods are used (e.g., ultrasonic). See Section 26.3.

NDOT has developed a separate data reporting form for the inspection of bridge-mounted signs, luminaire supports and utilities. The form is part of the Bridge Inspection Report. The bridge inspector should provide special attention to the attachments of these structures to the bridge.

28.2.6.12 Pedestrian Structures

NBIS does not require the periodic inspection of bridge structures that carry only pedestrian or bicycle traffic, and pedestrian structures are not included in the Nevada Bridge Inventory.

However, NDOT performs routine inspections on publicly owned pedestrian structures on a 24-month frequency, acknowledging concern for the roadway and motorists passing underneath.

28.2.6.13 Miscellaneous Structures

Although NBIS does not require the inspection of the following structures, the following presents NDOT practices for the inspection of these miscellaneous structures:

1. Culverts. Culverts less than or equal to 20 ft in length do not require scheduled inspections. These structures shall receive periodic inspections when warranted. A noticeable dip in the roadway, breakdown of the roadway shoulder, excessive pavement cracking, or restricted water flows are all signs of possible structural problems with the culvert. Culvert structures shall be assigned a Structure Identification Number beginning with the prefix "C" and shall be identified on structure location maps.

2. Retaining Walls. Retaining walls constructed to retain approach roadway fills adjacent to bridges are inspected as an integral part of the Routine Inspection process. Retaining walls shall also be inspected when they show visual signs of distress. Where the retaining wall retains the roadway, excessive roadway settlement and pavement cracking may indicate a problem with the wall. Retaining walls shall be checked for plumbness, excessive movement, spalling and heavy rust staining on the front face. Check for proper drainage behind the wall. Inadequate drainage can result in overstress of the wall.

3. Sound Barriers. Bridge-mounted sound barriers shall be inspected with the bridge. Sound barriers should be checked for collision damage, plumbness, corrosion and wall panel deterioration. The inspector shall also closely observe the connection details of the wall to the bridge.

4. Privately Owned Structures. Privately owned structures open to unrestricted public access are inspected and remain in the Nevada Bridge Inventory. Those structures restricted to open traffic are not inspected.

5. Tunnels. Tunnels carrying unrestricted highway traffic receive routine inspections every 24 months. An inspection report shall be produced for all tunnel inspections, which shall include condition ratings, written documentation of findings and maintenance recommendations.

28.2.6.14 Estimated Remaining Life

In general, NDOT determines the frequency of its bridge inspections based on the Estimated Remaining Life (ERL) of the structure. NDOT's criteria are:

1. ERL > 5 Years. All bridges with an ERL greater than 5 years are inspected at least once every 24 months.

2. ERL ≤ 5 Years. All bridges with an ERL of 5 years or less are inspected at least every 12 months.

Estimated Remaining Life determinations will be made according to the following criteria:

1. New and Reconstructed Structures. ERL of new and reconstructed structures shall be as follows:

   Bridges (including box culverts): 75 years
   Tunnels: 75 years

2. ERL Reduction. Reduction of ERL of bridges shall be on a year-for-year basis from the date of construction. If a bridge has been hit or otherwise damaged or if a bridge is found to have substandard materials or workmanship, the ERL may be reduced accordingly.

3. Limit of Reduction. Once the bridge ERL has been reduced to 15 years, it shall remain at 15 years until further deterioration of the structure indicates a need to continue reducing the ERL.

4. Rehabilitated Bridges. Rehabilitated bridges will be assigned an ERL value of 25 years with the year of rehabilitation as the base year. An exception to this is when the bridge rehabilitation occurred during the first 25 years of the bridge life. In this case, the ERL shall be based on the original 75-year life. The ERL of rehabilitated bridges shall be reduced as specified in Nos. 2 and 3. (Note: The distinction between reconstruction and rehabilitation will be made by the Structures Division on a case-by-case basis. A subjective evaluation will be made on how much the bridge improvement work adds to the Estimated Remaining Life of the structure).

5. Repaired Bridges. Any added bridge life resulting from bridge repairs will be determined on a case-by-case basis.

6. Tunnels. The ERL of tunnels shall be reduced on a year-for-year basis from the date of construction. If the tunnel has been damaged, the ERL may be reduced accordingly. Once the ERL of a tunnel has been reduced to 15 years, it shall remain at 15 years until further deterioration indicates a need to continue reducing the ERL.

7. Rehabilitated Tunnels. Rehabilitated tunnels will be assigned an ERL value of 50 years with the year of rehabilitation as the base year. The only exception to this rule is when the rehabilitation occurred during the first 25 years of the tunnel's life. In this case, the ERL shall be based on the original 75-year life. The ERL of rehabilitated tunnels shall be reduced as specified in No. 6.

28.2.7 Scour Critical Bridges

28.2.7.1 General

Scour is the movement of channel bed material by the action of the moving water. This movement may result in degradation, or erosion of material or aggradation, or accumulation of material. Degradation of the channel bed may lead to structure instability, posing an often unseen threat to safety. Scour is generally most severe during periods of high flow. When flows recede to normal levels, the presence of scour is often hidden by silt or debris, making detection difficult. Scour is the leading cause of bridge failures.

To address this concern, NDOT maintains bottom profile records of all Nevada bridges over waterways. The records include "local" channel bottom elevations along the upstream fascias of the bridge. Additionally, for bridges over large waterways, local channel bottom elevations are collected around the perimeter of all substructure elements in the water. These records are obtained in conjunction with both the Routine and Underwater Bridge Inspections.

Additionally, all Routine and Underwater Inspections include an evaluation of substructure foundation exposure, including the assessment of any foundation undermining found during the inspection. Often, foundation undermining can only be found or assessed using divers.

Finally, all Routine and Underwater Inspections include an evaluation of the waterway adjacent to the bridge. This evaluation includes an assessment of channel scour in the vertical orientation and channel embankment erosion/lateral channel migration in the horizontal orientation. Vegetation intrusion and channel bottom material aggradation adjacent to the bridge are also evaluated, as is the effectiveness of channel embankment protective measures (e.g., riprap, slope pavement).

28.2.7.2 Scour Evaluation and Plans of Action

§650.313(e)(3) of the NBIS requires that each State DOT must:

> *Prepare a plan of action to monitor known and potential deficiencies and to address critical findings. Monitor bridges that are scour critical in accordance with the plan.*

The following describes the NDOT Policy, which the Structures Division and Hydraulics Section developed and implemented jointly, to comply with NBIS Plan of Action (POA) requirements:

- For all bridges over waterways, a multi-disciplinary team of engineers (i.e., geotechnical, hydraulic, structural) shall perform a scour analysis. The multi-disciplinary team will develop a scour POA for use by the Bridge Inspection Squad.
- The ACSE – I/I maintains a list of those bridges in the State of Nevada that have been determined to be scour critical.
- Each scour critical bridge has a unique POA based on the specific hydraulic, geotechnical and structural characteristics for that bridge site.
- NDOT is responsible for ensuring that POAs are developed for all scour critical bridges.
- The POA includes:
    + the discharge of concern that will trigger an Underwater Inspection,
    + specific actions that must be performed during a bridge inspection to monitor the foundation,
    + scour mitigation measures that are deemed appropriate, and
    + threshold events that will justify closure of the bridge.

28.2.7.3 Scour Evaluation During Routine Inspections

A local channel bottom evaluation shall be conducted for all bridges over water. The minimum recommended channel bottom measurements to be obtained during a Routine Inspection should include:

- channel bottom elevations along upstream fascia of the bridge taken at each substructure unit or element and at mid-span points at a minimum, and

- additional channel bottom elevation measurements around substructure units as deemed necessary by the TL.

An evaluation of substructure foundation exposure/undermining shall also be conducted, using visual, wading and probing methods, as applicable. When exposures/undermining conditions cannot be adequately assessed using these methods, an underwater diving inspection shall be recommended.

The waterway adjacent to the bridge shall also be evaluated, largely using visual methods. This evaluation shall include the following assessments:

- channel scour in the vertical orientation;

- channel embankment erosion and lateral channel migration in the horizontal orientation;

- vegetative growth throughout the channel, including along the channel banks;

- material aggradation in the channel, both upstream and downstream of the bridge; and

- effectiveness of channel embankment protective measures (e.g., riprap, slope pavement). Where protection is warranted but lacking, it should be recommended.

28.2.7.4 Scour Evaluation During Underwater Inspections

In addition to evaluations required during the Routine Inspection, water depth measurements during an Underwater Inspection should also include the elevation measurements obtained in concentric rings at the ends and quarter points of the element at distances of 0 ft, 5 ft, 10 ft and 15 ft from the element face.

28.2.8 <u>Scheduling Inspections</u>

28.2.8.1 General

§650.311 presents the inspection frequencies based on the type of inspection (e.g., Routine Inspections) and/or special inspections (e.g., Fracture Critical Member Inspection). In general, NDOT schedules inspections during the year by District as follows:

- <u>District I</u>. October through February.
- <u>District II</u>. March through June.
- <u>District III</u>. July through September.

28.2.8.2 Routine Inspections

§650.311(a) requires that Routine Inspections for each bridge be scheduled at regular intervals not to exceed 24 months. NDOT shall use the following in scheduling Routine Inspections:

- scheduled based on the generated date of next inspection;
- completed no sooner than 30 days prior to, and no later than, the scheduled inspection date; and
- may be rescheduled in the event of an emergency, inclement weather, safety concerns or other unforeseen circumstance with the approval of the PSE. Inspection may be rescheduled no later than 30 days following the date of the next inspection.

28.2.8.3 Access-Required Routine Inspections

Figure 28.2-A will determine the frequency of inspections using access equipment. Due to resource limitations, not all bridges requiring access can be so accommodated in each inspection cycle. As a consequence, if time/resource constraints require the elimination of scheduled access-required inspections, those structures with 96-month frequencies shall be eliminated first and, if further eliminations are necessary, they shall be taken from the groups listed in Figure 28.2-A in the following sequence:

- concrete slab, box girder or filled-deck arch bridges (48 months);
- concrete girder or T-beam bridges;
- concrete open-spandrel or through-arch bridges; and
- steel bridges.

Candidate bridges so eliminated will be inspected without the use of access equipment.

28.2.8.4 In-Depth Inspections

In-Depth Inspections will be scheduled by the PSE on an as-needed basis with scheduling as close as practical following the Routine Inspection.

28.2.8.5 Complex Bridge Inspections

A Complex Bridge Inspection follows the requirements for Routine Inspections regarding the frequency date of the next inspection. Complex Bridge Inspections may be scheduled separately for defined segments of the bridge or for designated groups of elements, connections or details that can be efficiently addressed by the same or similar inspection techniques. If the latter option is chosen, each defined bridge segment and/or each designated group of elements, connections or details should be clearly identified and recorded, and each should be assigned a frequency for re-inspection.

Bridge Type	Minimum Inspection Frequency
All steel bridges	48 months or 24 months
Concrete girder or T-beam bridges	48 months
Concrete slab, box girder or filled deck arch	96 months or 48 months
Concrete open-spandrel arch or through arch	48 months
Concrete frames, tunnels or other bridges	As needed

Notes:

1. *Steel bridges should be inspected using access equipment on a 48-month minimum frequency, if they are basically "totally accessible" (i.e., every span has a vertical clearance not exceeding 25 ft and is in the dry or otherwise able to be examined by walking under each span). If these structures are not "totally accessible," a 24-month minimum inspection frequency should be used.*

2. *Concrete refers to both prestressed and conventionally reinforced concrete.*

3. *Concrete slab, box beam or filled deck arch structures should be inspected using access equipment on a 96-month minimum frequency, if they are basically "totally accessible" (i.e., every span has a vertical clearance not exceeding 25 ft and is in the dry or otherwise able to be examined by walking under each span). If these structures are not "totally accessible," a 48-month minimum inspection frequency should be used.*

ACCESS-REQUIRED ROUTINE INSPECTIONS

Figure 28.2-A

28.2.8.6 Special Inspections

Special Inspections are used when a structure requires more frequent inspection than that of the Routine Inspection. Special Inspection frequency should reflect the severity of the deficiency. Special Inspections might be used to inspect load-posted bridges, monitor severely deteriorated conditions or when an In-Depth Inspection is warranted. Special Inspections will be scheduled by the PSE on a case-by-case basis.

28.2.8.7 Underwater Inspections

An inspection of permanently submerged structural elements shall be performed once every 48 months. *Note: FHWA requires an Underwater Inspection once every 60 months.* These inspections shall be scheduled to coincide with periods of low-flow to minimize the extent to which elements are submerged. The exceptions to the above shall be during flooding conditions. Submerged structural elements threatened by flood waters shall be inspected as soon as it is safe to enter the water following the flood.

28.2.8.8 Fracture Critical Member (FCM) Inspections

Fracture Critical Member Inspections (FCM) should be scheduled at regular intervals not to exceed 24 months, and are usually scheduled to coincide with the Routine Inspection of a bridge. These Inspections often require the scheduling of special access equipment, traffic control and NDT examination, and on normally conducted simultaneous Routine Inspections.

28.2.9 Inspection Preparation Procedures

The Inspection Team Leader (TL) should perform the following office procedures to prepare for the bridge inspection:

1. Document Preparation. Check the original bridge plans and rehabilitation plans, preferably "As-Built" plan (if available), which will determine the type of bridge, bridge components and foundation that will be inspected. Check the bridge files to review previous inspection reports and to determine the deficiencies that were noted. Prepare copies of relevant plan sheets and previous inspection report documentation, which will be used as reference materials during the inspection.

2. Equipment. Determine the access equipment and inspection equipment that will be needed for the group of bridges that will be inspected and where this equipment is located. Make the necessary arrangements to relocate the equipment to where it is needed. Make arrangements for NDT examination and traffic control if applicable.

3. Coordination. The following applies:

   a. Coordination Within NDOT. Notify all appropriate NDOT Divisions and Districts of the times that personnel from the Structures Division will be in the area to inspect bridges and to confirm staff and equipment availability, ability to provide traffic control, etc. Coordinate with the Public Information Officer for advance public notification as needed.

   b. Coordination Outside NDOT. Notify all outside agencies (e.g., local owners, NHP, media) of pertinent bridge inspections and arrange a mutually satisfactory time for their personnel to be present, if requested or necessary. Provide the traveling public with advance warning of lane or ramp closures, as deemed necessary by NDOT District and Public Information Officials. Make arrangements with private property owners when necessary to complete the inspection.

   c. Railroad Coordination. NDOT must coordinate with the Railroad Company when its Inspection Team will be working within 25 ft of the centerline of tracks. The Railroad Company will provide an employee to assist the Inspection Team with track control. NDOT provides the Railroad Company with ample advance notice (typically, 4 to 6 weeks) of the inspection schedules for bridges requiring Railroad track control. When on site, the TL will provide the Railroad employee with a two-way radio to notify the NDOT Team of an approaching train. NDOT inspectors must follow directives relating to railroad safety provided by the Railroad track control representative.

4. Inspection Plan. Develop an inspection plan for each bridge on the inspection schedule. Check the bridge folder to determine if an inspection plan has already been developed, and update the plan as needed.

28.2.10 Field Inspection Procedures

28.2.10.1 General

In general, NDOT requires that all bridge inspections in the field be consistent with the recommendations and guidance contained in the FHWA *BIRM* and AASHTO *Manual*. In addition, the *NDOT Bridge Inspection Manual* and FHWA *Coding Guide* present specific information for the bridge inspector's use in completing the field inspection and processing the resulting Bridge Inspection Reports. Both NBI and PONTIS element level inspections are required to be completed for each bridge. In addition, the following are recommended field procedures for all inspections:

1. Safety Briefing. When non-NDOT personnel are present, they shall be briefed on NDOT safety requirements before starting the inspection. Do not proceed with any inspection without the proper personnel being present and having received a safety briefing.

2. Inspection Plan Review. Examine the detailed inspection plan to determine where to position equipment. Modify the inspection plan by noting the location of piers, abutment slopes and any other obstructions under the bridge.

3. Equipment Check. Verify that the necessary equipment has been assembled and is on site.

28.2.10.2 Traffic Control

The District Offices are responsible for planning and implementing traffic control procedures for all Access-Required Bridge Inspections. Assessment of specific lane closures shall be determined by the TL in collaboration with the Special Equipment Operator and District Bridge Maintenance Crew leader.

Special considerations, such as restricting the time of inspections to low-volume periods, permit a more thorough inspection of the entire bridge. The District Bridge Maintenance Crew leader shall assess these options in conjunction with the PSA and the TL. These considerations shall be implemented where feasible.

28.2.10.3 Safety

In general, all bridge inspection activities shall conform to NDOT safety policies. NDOT must also follow regulations as promulgated by OSHA. OSHA regulations that have an especially significant impact on bridge inspections include those pertaining to:

- heights,
- use of respirators,
- confined spaces,
- lead exposure,
- water, and
- railroads.

Additionally, the safety regulations of the Railroad Company, other governmental agencies or bridge owner shall be followed. During Access-Required Bridge Inspections, it is sometimes necessary to leave the work platform to complete a thorough review of the structure. When the bridge inspector determines that this is necessary, the following guidelines will apply:

1. Notification. The bridge inspector will inform the inspection vehicle operator of his/her intention to leave the work platform.

2. Safety Line. When leaving the work platform, the bridge inspector will use a safety line. This line will be attached to the superstructure or appropriate portion of the bridge that will ensure the safety of the individual leaving the work platform. Climbing activities conducted over live traffic shall require the simultaneous use of two safety lines.

These guidelines apply to NDOT personnel involved in conducting bridge inspections. When individuals from other government agencies, consultants, contractors, etc., are participating in the inspection process, it will be the responsibility of the Special Equipment Operator to ensure that these individuals are not allowed to leave the work platform without conforming to these guidelines.

28.2.11 Inspection Reporting Procedures

§650.313 presents the NBIS bridge inspection reporting procedures. The following sections present specific NDOT reporting procedures for the Nevada Bridge Inspection Program.

28.2.11.1 Report Preparation

The Bridge Inspection Report incorporates the results of both the NBI and PONTIS element level inspections and serves as the permanent inspection record. These Reports portray the condition of the bridge as it relates to public safety. They are also used for future rehabilitation and replacement decisions. Therefore, it is imperative that the Reports present accurate and thorough information. Reports should include photos, sketches, addenda, etc., as necessary to adequately and thoroughly document the condition of the structure but also be as concise as possible. Do not include information that is not necessary to communicate the nature of the structure's condition. When conditions are very good or excellent, as noted by the appropriate condition rating number and PONTIS element data, it is not necessary to provide narrative comments.

28.2.11.2 Review and Processing

Use the following minimum procedure for processing Bridge Inspection Reports:

1. Field Notes. Field notes shall be reviewed at the inspection site for completeness and accuracy.

2. Data Entry. The bridge inspection data from each Report shall be entered into the appropriate computer files by the TL or AI.

3. Draft. Each Report shall be printed and reviewed both by the TL and AI for completeness and accuracy. Errors or omissions noted shall be rectified. Reports deemed accurate and complete will then be initialed by both the TL and AI.

4. QC Review. Initialed Reports shall be circulated for approval. Reports generated by NDOT staff shall be reviewed and signed by the PSE or designated representative, who shall be qualified as a TL. Consultant Reports shall be reviewed by the consultant Project Manager or NDOT-approved, designated alternate, who must be a Registered Professional Civil or Structural Engineer in Nevada.

5. Corrections. If originals are returned to the TL for corrections, they shall be revised to the satisfaction of the QC Reviewer.

6. Final Report. The revised Bridge Inspection Report shall then be submitted to the QC Reviewer for final approval. The consultant review process is finalized by the signing and sealing of the Report by the Project Manager or NDOT-approved, designated alternate. Completed reports shall be forwarded by the consultant to the PSE for final NDOT review and acceptance.

7. Submittal. The original copy of the accepted Bridge Inspection Report shall be forwarded by the PSE to the NDOT Bridge Inventory Management Squad for distribution and filing.

8. Distribution and Filing. Report originals shall be filed in the NDOT Bridge Inventory files with a copy distributed to the owner. For State-owned bridges, the copy is provided to the Assistant District Engineer or designated individual.

28.2.11.3 Submittal Time Requirements

The following identifies the time requirements for submitting the Bridge Inspection Report:

- Reports must be submitted to the PSE for acceptance review within 45 days of the date of inspection.

- Any Bridge Inspection Report returned by the PSE for correction must be returned to the PSE within 75 days of the date of inspection.

- §650.315(b) of the NBIS stipulates that States have 90 days following the date of inspection for State-owned bridges (180 days for non-State-owned bridges) to update the State Bridge Inventory. This is required following a field inspection, or at any other time there is any change in the reporting status of bridges. These time requirements shall have no influence over the 45- and 75-day requirements stipulated above.

28.2.12 Bridge Inventory Procedures

28.2.12.1 Definitions

The following definitions apply:

1. National Bridge Inventory (NBI). The aggregation of structure inventory and appraisal data collected to fulfill the requirements of the National Bridge Inspection Standards, which requires that each State prepare and maintain an inventory of all bridges subject to the NBIS.

2. National Bridge Inventory (NBI) Record. Data that has been coded according to the FHWA *Recording and Coding Guide* for each structure carrying highway traffic or each inventory route which goes under a structure.

3. Structure Inventory and Appraisal (SI&A) Sheet. The graphic representations of the data recorded and stored for each NBI record in accordance with the *Recording and Coding Guide*.

28.2.12.2 NBI Data Reporting

The Bridge Inventory Management Squad is responsible for maintaining the Nevada Bridge Inventory of all public bridges in Nevada not owned by any Federal agency. The Squad also prepares and processes the Structure Inventory and Appraisal (SI&A) data for all bridges in the Nevada Bridge Inventory. The implementation of these functions must comply with §650.315(a) of the NBIS.

The Nevada Bridge Inventory includes an element-level database, electronic directory of Supplemental information files (approved format such as .pdf, .doc, etc.), and the National Bridge Inventory (NBI) file. As mandated by FHWA, the NBI file has been programmed to submit the data in the format described in the FHWA *Recording and Coding Guide* for all bridges in the State. The submission to FHWA is typically due by March 31 of each year. These data are used by FHWA to assign a Sufficiency Rating to each bridge according to the Sufficiency Rating (SR) Formula in the FHWA *Coding Guide*. The ACSE – I/I is the primary point of contact regarding data submittal to FHWA.

28.2.12.3 Meaning of Sufficiency Rating

The "Sufficiency Rating" is used by FHWA as a numerical indicator of a bridge's sufficiency to remain in service. The Sufficiency Rating is based upon a 0 to 100 scale (100 being best), and is calculated using a formula which incorporates four factors:

- Structural Adequacy and Safety (55%),
- Serviceability (30%),
- Essentiality for Public Use (15%), and
- Special Reductions (up to 13%).

Bridges categorized as Structurally Deficient or Functionally Obsolete, with a Sufficiency Rating of less than 50.0, qualify for replacement using Federal Highway Bridge Program (HBP) funds; those bridges with a Sufficiency Rating of 80.0 or less are eligible for rehabilitation. See Section 22.1 for more discussion on the HBP.

A bridge is categorized as Structural Deficient if the bridge:

- is in relatively poor condition due to deterioration;
- has insufficient load-carrying capacity (whether due to the bridge being of older design or due to deterioration); or
- the structure frequently floods, causing significant traffic delays.

Deficient bridges require significant maintenance attention, rehabilitation or replacement. However, the classification of a bridge as Structurally Deficient does NOT typically mean that it is in danger of collapse.

A bridge is categorized as Functionally Obsolete if the bridge:

- is narrow,
- has inadequate underclearances,
- has insufficient load-carrying capacity,

- is poorly aligned with the adjacent roadway, and/or
- can no longer adequately service today's traffic.

Functionally obsolete bridges do not provide the lane widths, shoulder widths, vertical clearances, etc., adequate to serve traffic demand, or the bridge may not be able to handle occasional roadway flooding without causing traffic delays.

Bridges that qualify as both Structurally Deficient and Functionally Obsolete are categorized and reported solely as Structurally Deficient. Further, bridges built or reconstructed within the last 10 years do not qualify as Structurally Deficient or Functionally Obsolete, based on the FHWA "10-year rule."

28.2.12.4 Structure Number Assignment

The Bridge Inventory Management Squad is responsible for assigning Structure Numbers to all structures and for recording these numbers in NDOT's Structure Index. Therefore, the Structures Division has adopted a procedure to ensure that:

- Structure Numbers are properly recorded.
- Each structure location is assigned a unique number that is maintained for all subsequent replacement structures at that location.
- All appropriate files are created or modified for each structure in the contract documents.
- The files are updated in a timely manner consistent with NBIS requirements.

28.2.12.5 Contract Document Review

The following describes the procedures for reviewing the contract documents to identify Structure Numbers:

1. <u>Structure Identification</u>. Review the "Structure List" in each set of contract plans to determine if any "bridge" or "culvert" structures are within the limits of and are affected by the contract. "Bridge" structures shall be as defined in the most recent edition of the FHWA *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges.* "Culvert" structures shall include multi-pipe, RCB, etc., structures with total spans of 10 ft to 20 ft along the roadway centerline.

2. <u>Absence of Structures</u>. Return all copies of contract documents with no "bridge" and/or "culvert" structures to Central Records.

3. <u>Structure Number Verification</u>. Verify that all "bridge" and/or "culvert" structures have been assigned appropriate structure numbers.

4. <u>Structure Number Assignment</u>. If structures exist on the plans and no numbers have been assigned to them, it must be determined if these structures replace structures previously assigned numbers or are new structure locations. New structure locations should be assigned numbers in the current sequence, and replacement structures should be assigned the previous number with the appropriate prefix and/or suffix modifications.

5. Recording. Pen-in structure numbers on the "Structure List" and any other pertinent locations throughout the contract documents. File a copy of these plans in the Division's Contract Plans file.

6. Nevada Map Atlas. Plot all "bridge" structures in the "Nevada Map Atlas" for "bridges" and all "culvert" structures in the "Nevada Map Atlas" for "culverts." The Staff III Associate Engineer – Bridge Inventory Management Squad maintains these map atlases.

28.2.12.6 Bridge Inventory Updates

When the Bridge Inspection and PONTIS Inspection Reports are received, the Bridge Inventory Management Squad performs the following:

- The Squad checks the NBI data within the context of its responsibilities in managing the Nevada Bridge Inventory (e.g., error checks, reasonableness checks). For example, the Squad always checks the geometric range items.
- The Squad decides what NBI data, if any, that has been recommended for change should be updated in PONTIS by the NDOT or consultant TL.
- The Squad files the hard copy of the Bridge Inspection Reports in the appropriate Bridge Inspection folders segregated by Structure Number.
- The Squad prepares the SI&A data for submission to FHWA.

28.2.13 Quality Assurance Program (NDOT)

28.2.13.1 General

§650.305 of the NBIS defines Quality Assurance (QA) as:

> *The use of sampling and other measures to assure the adequacy of quality control procedures in order to verify or measure the quality level of the entire bridge inspection and load rating program.*

§650.313(g) requires that a State DOT have QA procedures that:

> *Include periodic field review of inspection teams, periodic bridge inspection refresher training for program managers and team leaders, and independent review of inspection reports and computations.*

Each year, NDOT conducts a QA review of approximately 5% of the bridges inspected in each District. The purpose of the review is to evaluate the field inspection and report processing performance of both NDOT and consultant inspection NBI staff. This is intended to be a constructive process with the objective of improving the quality, accuracy, utility, etc., of the inspections. The QA Review Team typically consists of:

- the Principal Structures Engineer (PSE) , and
- a representative of the District Bridge Maintenance Crew or an alternate from the District or Structures Division as determined by the PSE (field review only).

28.2.13.2 Field Review

The following summarizes the field review element of NDOT's QA Program:

1. Schedule. The QA field inspections are typically conducted in three, one-week trips; one in each of the three Districts.

2. Structure-Type Selection. The QA Review Team selects a random sample of those bridges in the District inspected during the preceding months. This sample will consist of between 15 and 25 structures with a distribution of structure types similar to that covered during the original inspections. However, in no case will more than two culvert-type structures be selected. The sample must also represent approximately equal numbers of structures inspected by each Inspection Team Leader. Furthermore, the sample must not include any structure previously included as part of a QA Review in the previous two years.

3. Review Procedure. For each structure, the QA Review Team will conduct a 100% independent inspection (i.e., without any knowledge of the original Bridge Inspection Report findings). The QA Review Team will assemble its own element-based condition and NBI ratings.

4. Results. The QA field inspection results will be compared to the results of the original inspections to identify any significant differences. Condition rating differences will be considered significant when they exceed a difference of ± 1. PONTIS element changes will be considered significant when Condition State distributions or total quantity values vary by more than 15%.

28.2.13.3 Office Review

The PSE will review the original, approved Bridge Inspection Report. The individual sections of the Report will be reviewed in detail for consistency, accuracy and thoroughness.

28.2.13.4 Summary

The PSE will prepare a Summary Report of the QA Review. The Report will itemize the findings for each structure included in the QA Review, will provide a discussion on each significant finding and will document recommendations for improving inspection performance.

28.2.14 Quality Assurance (FHWA)

FHWA conducts an annual review of the Nevada Bridge Inspection Program as part of its nationwide Quality Assurance oversight of the NBIS. FHWA coordinates its QA review directly with the ACSE – I/I. The annual review is conducted by the Nevada Division Office Bridge Engineer and may include additional FHWA representatives. The following briefly describes the FHWA review:

1. Office Review. FHWA will conduct one or two on-site meetings with NDOT. Typical items that could be reviewed are:

- the integrity of the NBI data;
- the percent complete for load rating of existing bridges; and
- specialty items (e.g., Fracture Critical Member Inspections, Underwater Inspections).

2. Field Review. FHWA spends approximately one week in the field with the PSE. FHWA will review selected bridges concurrent with their scheduled Routine Inspections. Inspections completed by both NDOT and consultant forces are subject to FHWA review.

3. Summary Report. FHWA will prepare a Summary Report of its findings and recommendations and hold a close-out meeting with NDOT.

28.2.15 Administrative Procedures

28.2.15.1 Dangerous Duty Pay (DDP)

Nevada Administrative Code 284.208 provides for an additional 10% compensation for each hour that an employee performs duties at a height of more than 16 ft, or engages in underwater diving. DDP time shall be applicable to inspection work performed on eligible bridges. A list of eligible bridges is maintained by the PSE.

28.2.15.2 Bridge Owner Education Program

NDOT is implementing a Program to educate non-State bridge owners in Nevada on their bridges.

28.3 LOAD RATING AND RELATED OPERATIONAL ISSUES

28.3.1 Load Rating

28.3.1.1 Definitions

§650.305 of the NBIS defines load rating as:

> *The determination of the live load carrying capacity of a bridge using bridge plans and supplemented by information gathered from a field inspection.*

In addition, the following definitions apply:

1. Inventory Rating. The load level that can safely use an existing structure for an indefinite period of time.

2. Operating Rating. The maximum permissible load level to which the structure may be subjected for the load configuration used in the rating.

28.3.1.2 Responsibilities

The Load Rating/Over-Dimensional/Over-Weight Permitting Squad within the Inventory/Inspection Section is responsible for determining the load-carrying capacities of all bridges in the State of Nevada open to the public. The NDOT procedures and methodology as presented in Section 28.3.1 meet all NBIS requirements.

Regarding weight-restriction (load) posting, the Load Rating/Over-Dimensional/Over-Weight Permitting Squad determines the need for load posting of all State-owned bridges. The Chief Structures Engineer has the authority to order that such a bridge be posted. For non-State bridges, the Squad recommends to the applicable owner the appropriate load posting for bridges under its jurisdiction.

28.3.1.3 Procedures

The bridge designer shall place all computer models, computations, assumptions and correspondence on a Compact Disc for storage in the Structural Design Notebook (see Chapter 7) and for storage on the NDOT server in the 011Bridge/011LoadRatings/BridgeLoadRatings folder for potential use in rating the bridge. Include a base computer model of the bridge in a separate location of the Compact Disc for future analysis needs. Computer models shall be developed on approved software programs.

When a bridge inspection reveals a quantifiable change in the bridge condition (e.g., increased metal section loss), the bridge must be re-rated to determine the new load-carrying capacities. These new load ratings are then entered into the NBI and PONTIS databases. If the load-carrying capacity falls below certain limits, the bridge must be posted.

28.3.1.4 General Rating Methodology

For all new or replacement bridges designed in-house or by NDOT consultants, the bridge designer shall calculate the Inventory and Operating Ratings during the design phase of a project. Load rating shall be based on the Load Factor Rating Method. All bridges shall be load rated for HS-20 loading (truck or lane, whichever produces the lowest rating). The number of wheel lines per girder (or other live load application), that is the live load distribution factor, shall be in accordance with the AASHTO *Standard Specifications for Highway Bridges* requirements for design.

All bridges shall be load rated for the Operating Rating for the California permit vehicles P13, P11, P9, P7 and P5. Axle loads of the P trucks shall be at 18 ft on center and the tandem axles shall be modeled as a single 48-kip load at the center of the tandem. Each vehicle shall be applied separately, and the number of wheel lines per girder (or other live load application) shall be the same as for HS-20 truck loading.

28.3.1.5 Rating by Structural Component

Girder bridges shall be rated by line girder analysis where applicable. Concrete box girders, except those of complex geometry, shall be rated on a whole-bridge basis with the total number of wheel lines equal to the sum of the wheel lines for the individual girders.

Concrete bridge decks supported by girders shall not be rated unless there is evidence of distress or deterioration to below a NBI 6 condition rating. Other types of bridge decks shall be rated. When concrete bridge decks are rated, bridge decks exposed to traffic shall be rated with the top ½ in of deck considered a sacrificial wearing surface and not included in the deck section properties.

Superstructure girders, floor beams, trusses and arches (including earth filled spandrel arches) shall be rated.

Concrete pier caps shall not be rated unless there is evidence of distress, deterioration to below a NBI 6 condition rating, or evidence that they would control the rating.

The substructure, including pier columns, abutments, footings and wing walls, shall not be rated unless a special rating is requested. Such special rating is justified only where there is evidence of distress, deterioration to below a NBI 6 condition rating, or scour or other undermining.

28.3.1.6 Material Properties

Material properties shall be as shown in the plans unless the latest Bridge Inspection Report states deterioration of a component and a NBI condition rating less than 6. With reported deterioration, material properties of the deteriorated component may be reduced based on engineering judgment derived from a visual inspection of the bridge or may be increased or decreased based on materials testing. Materials testing shall be performed only by written authorization of NDOT. Material testing shall not be done where the inventory rating factor for HS-20 loading is one (1.00) or more when based on materials properties without testing.

NDOT Standard Specifications after 1961 specify f'_c = 3000 psi for Class A, AA, B, BA, D and DA concrete and f'_c = 2500 psi for Class C and CA. Structures where the concrete strength f'_c is not specified in the plans shall be rated using the default concrete strengths in the AASHTO *Manual for Condition Evaluation of Bridges* for the date of construction, except that the above *NDOT Standard Specification* strengths shall be used where they apply. Concrete strengths for strength calculations may be reduced for deterioration per the above paragraph.

28.3.1.7 Dimensions

Dimensions shall be as shown in the plans unless the latest Bridge Inspection Report states deterioration of a component with a NBI condition rating less than 6 or if measured dimensions as part of a visual inspection deviate significantly from the plan dimensions. With reported deterioration, structural dimensions of a deteriorated component may be reduced based on engineering judgment derived from a visual inspection of the bridge to discount deteriorated material. A comprehensive field measurement of dimensions shall be performed only by written authorization from NDOT.

The section properties of composite girders shall be based on the full depth of the composite deck slab unless deterioration is noted and the NBI condition rating for the deck is less than 6.

28.3.1.8 Software

Software shall be used to perform the ratings as specified below where their use is possible. Data preparation, input files and output files shall be arranged such that the programs may be rerun by NDOT for truck-specific ratings and to update the ratings with a minimum of effort.

Culverts shall be rated using the computer program BRASS-CULVERT.

Girder bridges, other than those of post-tensioned concrete or curved steel girders, shall be rated by the computer program BRASS-GIRDER. If the P trucks are included in the BRASS-GIRDER truck library, the P5, P9 and P13 trucks shall be named exactly P5, P9 and P13 (capital P).

Girder bridges of combinations of pre-tensioned and post-tensioned concrete that do not have rigidly connected supports shall be rated by the computer program BRASS-GIRDER as possible and by other means if necessary. Such bridges with rigidly connected supports shall be rated by other means. Allowable concrete tension stress for inventory rating shall be $6\sqrt{f'_c}$ (psi units), except that the top of bridge decks located north of longitude 38°N, or in other areas where de-icing salts are used, shall be limited to $3\sqrt{f'_c}$ (psi units).

Girder bridges of post-tensioned concrete shall be rated by whole-bridge or line-girder analysis when applicable. Such bridges shall be rated by computing moments and shears by analysis using the computer program BD2 or WinBDS and computing strengths and rating factors with the Excel spreadsheet PTRater provided by NDOT. Bridges sharply curved, extremely flared, hourglass shaped or highly skewed with strong piers may require advanced analysis as a non-typical bridge. Allowable concrete tension stress for inventory rating shall be $6\sqrt{f'_c}$ (psi units), except that the top of bridge decks located north of longitude 38°N, or in other areas where de-icing salts are used, shall be limited to $3\sqrt{f'_c}$ (psi units).

Girder bridges of curved steel girders shall be rated by the computer program MDX.

Arch bridges, and other non-typical bridges, shall be analyzed by the computer program SAP2000 with manual calculations and spreadsheets for the rating as required. An alternative general frame analysis program (2D or 3D) or specialized software may be used only with approval from NDOT.

28.3.1.9 Skewed Bridges

The reactions and shears in an exterior girder of a skewed bridge are higher at the obtuse corners and lower at the acute corners compared to an interior girder. This becomes more pronounced as the skew increases. Section 13.2.2.3 discusses the analysis of skewed bridges.

28.3.1.10 Rating Methodology Details

Member properties for structural analysis shall be in accordance with the AASHTO *Manual for Condition Evaluation of Bridges.* Properties for concrete members shall be based on gross concrete section without adjustment for reinforcement or cracking, except that torsion properties of concrete members shall be adjusted when the torsion exceeds the cracking torsion of the member.

Bridges with rigidly connected (integral) supports shall be analyzed as rigid frames. Diaphragm abutments free to translate and rotate at the bottom of the diaphragm shall be considered a simple support when $H < 0.1S$, where H = the extension of the diaphragm below the superstructure and S = the length of superstructure span that the diaphragm terminates. Skewed pier walls rigidly connected to the superstructure shall be modeled with appropriate section properties. The following is recommended but not required practice.

For 2D analysis by the whole-bridge or line-girder method, skewed pier walls rigidly connected to the superstructure may be modeled as a wall with:

- width = true wall length
- thickness = true thickness/cos(skew angle)
- skew angle = angle of support line from a perpendicular to the bridge centerline

The following applies:

1. Foundation Fixity. Columns or pier walls with structural hinges detailed at the bottom shall be considered pinned for rotation in the direction provided by the hinge at the hinge location. Columns or pier walls fixed to the foundation of a spread footing or pile cap shall be considered rigidly supported at a distance L below the top of the footing as appropriate for the analysis. The following is recommended but not required practice.

   Piers rigidly connected to the foundation may be considered fixed at a depth L below the top of the footing where:

$$L = \frac{L_f}{\left[4\left(L_f / L_c\right) + 6\left(I_f / I_c\right)\right]}$$

   where:

   L_c = column clear length
   L_f = footing length from CL column to edge of footing
   I_f = footing moment of inertia = $BH^3/12$
   (where B = footing width and H = footing depth)
   I_c = column moment of inertia

Column shafts shall be assumed fixed at L_s below the ground surface where:

$$L_s = 1.8\left(E_c / n_h\right)^{1/5}$$

L_s units are as obtained when consistent units are used and n_h is from the table below with medium soil as the default unless plans indicate another soil type.

	n_h (lb/in^3) (for static loading)		
	Loose	Medium	Dense
Above water table	30	80	200
Below water table	20	60	120

2. Sidewalk Loads. Bridges with sidewalks shall be rated based on the sidewalk carrying pedestrian live loads and stray wheel loads per AASHTO *Standard Specifications*. Sidewalk dead load is included and distributed across the entire bridge for box girders and slabs, to the nearest tub girder for tub girder bridges, and equally to the two nearest girders for I-girder and other open section bridges.

3. Barrier Rail, Curb and Median Loads. The loads from barrier rail and curb at the edge of the bridge shall be equally distributed across the bridge for box girders and slabs, to the exterior tub girder for tub girder bridges, and equally to the two outside girders for I-girder and other open section bridges. Median loads shall be distributed to the entire bridge for box girders and slabs, and equally to two girders on either side of the median for tub girder, I-girder and other open section bridges. When loads so distributed overlap, a uniform load across the bridge may be used.

4. Lost Forms and Stay-in-Place Metal Forms. Unless the plans indicate otherwise, box girders shall have a lost deck form weight of 12 psf. Decks with stay-in-place metal forms shall have a weight 12 psf greater than the nominal deck thickness as a load due to form weight and corrugation fill.

28.3.1.11 Deliverables

NDOT will provide the spreadsheet LoadRatingSummarySheets.xls to be completed and returned as a deliverable with the appropriate file name as specified below.

28.3.1.11.1 *Printed Deliverables (for Each Bridge Rated)*

1. One copy of the “Supplemental Maintenance Report” (part of the LoadRatingSummarySheets.xls) completed and with seal and signature of a Nevada licensed civil or structural engineer at the right of the Comments Block.

2. One copy of the “Load Rating Summary Sheet” for the girder or culvert as applicable (part of the LoadRatingSummarySheets.xls) completed. A similar sheet, derived from these, is required as a “Load Rating Summary Sheet” for each load rating when other methods are used for the rating.

3. One copy of manual calculations.

4. For Rating by BRASS-GIRDER: A print of the data echo and rating factor summary from the BRASS-GIRDER output file, to be compiled by cut and paste from the output file. Do not provide a print of the entire output file.

5. For Rating by PTRater: A print of the data and results of the worksheet “Main” in PTRater for each span rated and a sufficient excerpt of WinBDS output to document the input bridge properties and loads. Print WinBDS output in portrait mode using Courier 6 pt or Courier New 6 pt font with ¾-in margins.

6. For Rating by BRASS-CULVERT: A print of the summary of culvert geometry and loads (typically p. 13) and output rating factors (p. 19 for 1 cell; p. 22 for 2 cells, etc.) from the BRASS-CULVERT output to be compiled by cut and paste from the output file. Do not provide a print of the entire output file.

28.3.1.11.2 *Electronic Deliverables*

For the following specification of electronic deliverable, the bridgename is the bridge number without “ – ” (examples are B1558 for Bridge B-1558 or I1228N for I-1228N).

Provide a CD (standard density) for each group of bridges rated with bridge numbers in groups of 10 with the last digit 0 to 9 in sequence being a group (e.g., 500 to 509 is a group but 501 to 510 is not). For each structure rated, create on the appropriate CD a folder named bridgename, containing the following files and subfolders:

1. The spreadsheet LoadRatingSummarySheets.xls with sheets for the Supplemental Maintenance Report and Load Rating Summary Sheet completed and the file named bridgenameLRS.xls. Note that the condition rating is the NBI rating (0 to 9) from the last inspection; leave blank for initial rating of unconstructed bridges.

2. Custom spreadsheets as .xls Excel spreadsheet files; manual calculations scanned into electronic format and provided as .pdf or .jpg or .tif files; and text files as .txt files as used for the rating.

3. For Rating by BRASS-GIRDER: The BRASS-GIRDER input file named bridgename.dat and the BRASS-GIRDER output file named bridgename.out placed in a subfolder named BrassGirder.

4. For Rating by PTRater: The BD2 or WinBDS input file named bridgename.bds and the output file named bridgename.out in a subfolder named BD2. Additionally for each span rated, the PTRater file named PTRate“bridgename”sN.xls, where N is the span number (e.g., PTRateB1558Ns2.xls for span 2 of bridge B-1558N).

5. For rating of culverts by BRASS-CULVERT, the BRASS-CULVERT input file with file extension .cus with the name bridgename.cus and the BRASS-CULVERT output file with file extension .out named bridgename.out in a subfolder named BrassCulvert.

6. For rating of curved steel bridges, input and output files from the MDX program with the name bridgename.xxx for the input file and bridgename.out for the output file where .xxx is the native file extension of the rating program for input files. Place in a subfolder named MDX.

7. For arches and other non-standard bridges, input and output files for the computer programs used with files named in an organized manner to facilitate review of the input

data, review of the output results, and reanalysis of the bridge using the programs. Use a subfolder for each program used.

28.3.2 <u>Bridge Posting</u>

28.3.2.1 Weight Restriction

For bridges on the NHS, a bridge will be posted if its operating rating is below HS-20. For all other bridges, posting will be required if the operating rating is below H-15. In both cases, the posted load for signing shall be the inventory rating calculated for the bridge under evaluation. See Section 28.3.1.1 for the definitions of operating rating and inventory rating.

28.3.2.2 Size Restriction

It may be necessary to post a bridge because of vertical clearance restrictions or bridge width restrictions. The Traffic Engineering Section is responsible for selecting and installing any signs for bridge posting due to vehicular size restrictions, which will be based on the *Manual on Uniform Traffic Control Devices*. This includes any advance warning signs to notify approaching vehicles of the restrictions.

28.3.3 <u>Over-Dimensional Permits (Superloads)</u>

28.3.3.1 General

Very heavy and large transporter vehicles are allowed to travel over the State's highways by an over-dimensional permit. These permits are issued by the NDOT Load Rating/Over-Dimensional/Over-Weight Permitting Squad. Nevada allows double-wide vehicles to operate with these permits carrying double the load allowed for an 8-ft wide vehicle.

Nevada uses the same single-trip permit methodology as the States of California and Arizona. This methodology requires the integration of bridge design, bridge rating and truck weight regulation. Bridges are load rated for the Caltrans P5, P7, P9, P11 and P13 permit vehicles as permit loads, and a database of these ratings is maintained by Structures Division. A transporter truck is classified by its axle weights and axle spacing as a loading intensity and number of axles. The highest loading intensity allowed is called "Purple Loading." Bridges on a proposed route are checked for adequacy based on the load rating for a P truck with the same number of axles as the transporter. Additional load is allowed for vehicles with extra width and more than two wheel lines per axle. A single-wide transporter at Purple Loading produces stresses in a bridge up to those produced by a P-truck with the same number of axles. Similarly, a double-wide transporter with Purple Loading is equivalent to up to two P trucks side by side, each with the same number of axles as the transporter. Bridges listed as having P13 permit truck design are expected to carry a double-wide transporter equivalent to two P13 trucks side by side.

This methodology allows using load ratings on file for the P-trucks to quickly and accurately determine bridge adequacy for transporters. The Structures Division usually checks the adequacy of the bridges on the route of transporters over 250,000 lbs GVW in addition to the checks performed by the Load Rating/Over-Dimensional/Over-Weight Permitting Squad. Details of the single-trip permit methodology are provided in the following Sections.

28.3.3.2 Permit Limits

An over-dimensional permit is for a non-divisible load only, load intensity limit, GVW as the route will allow (bridges must be adequate), no length limit and width as the route will allow. The following applies:

1. NRS Tire Load Limits.

   Max tire load = 675 lbs/in x Tire Width for any steering axle
   = 550 lbs/in x Tire Width for any fixed axle

2. Truck Loading Classification and Maximum Load Intensity. Truck is classified as Purple, Green or Orange Loading as follows:

   a. Purple Loading.

      Single Axle: (axle-weight)/Bonus Factor ≤ 28,000 lbs

      Group of Axles: Sum of [(axle-weight)/Bonus Factor] ≤ 1.5 x 700(L + 40) lbs, where the group of axles considered are ≤ 18 ft center-to-center, first to last, and L = center-to-center distance of first to last axle in group considered, and tandem axles with spacing < 3.5 ft is considered a single axle, and Bonus Factors are tabulated for number of wheel lines and width.

   b. Green Loading.

      Max. Single-Axle Weight = 0.86 x Purple Load
      Max. Group-of-Axles Weight = 0.86 x Purple Load

   c. Orange Loading (seldom used).

      Max. Single-Axle Weight = 0.66 x Purple Load
      Max. Group-of-Axles Weight = 0.66 x Purple Load

   The maximum load intensity allowed is Purple Loading.

   Bonus Factors for axle width and number of wheel lines are as follows (all dimensions are in feet):

Out-to-Out Width of Truck at Tires (W, ft)	2 Tires/Axle	4 or 6 Tires/Axle	8 Tires/Axle	Dollys with 8 Tires in 2 to 4 Tires Dollys/Axle
W < 7	1.00	1.00	—	—
7 ≤ W < 10	1.00	1.00	1.15	—
10 ≤ W ≤ 14	1.00	1.10	1.25	—
W ≥ 13*	—	—	—	Lesser of (2) or (2 × intW/20) or (2 × X_{eff}/7)

* *For 13 ≤ W ≤ 15, minimum Dolly clearance = 2 ft. For W > 15, minimum Dolly clearance = 3 ft.*

X_{eff} = Minimum of (Dolly width) or (W – 2 × Dolly Width + 3)

Note: Dollys or special suspension required for axle width greater than 14 ft.

Special Bonus Factors for large axle spacing: Up to two tridem axles, with axle widths less than 10 ft, have a variable Bonus Factor = 60,000 lbs/(total axle weight of tridem in lbs), but not more than 1.15, provided that the following is met:

- Spacing from center axle of tridem to any axle not in tridem ≥ 18 ft.
- Spacing from any axle in tridem to any axle in another bonused tridem ≥ 24 ft.

Note: The Bonus Factor is reduced from 1.15 to limit the allowed axle weight of the tridem group to 60,000 lbs.

28.3.3.3 Bridge Adequacy

Bridges are rated for single-trip permit trucks based on the operating ratings for the Caltrans P5, P7, P9, P11 and P13 trucks as computed with the multi-lane live load distribution factor. The bridge rating is expressed as a five-letter color code. The letters from left to right are the rating for 5-axle, 7-axle, 9-axle, 11-axle and 13-axle permit trucks. The letters of the color code are:

Operating Rating	Code	Associated Loading Capacity
≥ 1.00	P	Purple
≥ 0.86	G	Green
≥ 0.66	O	Orange
< 0.66	R	Restricted

A typical color code would be PPGGO, meaning:

- P (Purple) capacity for 5-axle trucks
- P (Purple) capacity for 7-axle trucks
- G (Green) capacity for 9-axle trucks
- G (Green) capacity for 11-axle trucks
- O (Orange) capacity for 13-axle trucks

The bridge color code rating is for double-wide transporter trucks. Bridges designed for two P13 trucks side-by-side as a permit load (or overload in the AASHTO *Standard Specifications*) have a PPPPP rating by design. The P13 permit truck design using the multi-lane live load distribution factor is a slightly conservative, but standard method, used to produce the desired PPPPP rating.

The following applies:

- A bridge is adequate to carry a truck of Purple Loading with n axles without restrictions, if the permit rating for the bridge is “P” for a truck with n axles.

- A bridge is adequate to carry a truck of Green Loading with n axles without restrictions, if the permit rating for the bridge is “G” for a truck with n axles.

- A bridge is adequate to carry a truck of Orange Loading with n axles without restrictions, if the permit rating for the bridge is “O” for a truck with n axles.

- A bridge is not adequate to carry a truck of any Loading with n axles without special analysis, if the permit rating for the bridge is “R” for a truck with n axles.

28.3.3.4 Bridge Capacity Increases

Bridge capacities may be increased by restrictions on vehicular speed and location as follows:

1. The rating for a bridge is considered one classification higher than that from the load rating if the vehicle is restricted as follows: Either a 5-mph crossing speed, or the vehicle must cross in the center of a bridge with 10 ft clearance to the bridge railing on each side.

2. The rating for a bridge is considered two classifications higher than that from the load rating if the vehicle is restricted as follows: Both a 5-mph crossing speed, and the vehicle must cross in the center of a bridge with 10 ft clearance to the bridge railing on each side.

Bridge capacities may be increased for single-wide vehicles as follows: The rating for a bridge is one classification higher than that from the load rating for single-wide trucks with bonus factors no more than less than 1.10. *Note: This is approximately the no-bonus exception of the Caltrans procedures.*

Appendix 28A

NATIONAL BRIDGE INSPECTION STANDARDS

For convenience, Appendix 28A reproduces 23 CFR Part 650 Subpart C.

NATIONAL BRIDGE INSPECTION STANDARDS (23CFR, Part 650, Subpart C)

Title 23: Highways
PART 650 —BRIDGES, STRUCTURES, AND HYDRAULICS

Subpart C – National Bridge Inspection Standards

Source: 69 FR 74436, Dec. 14, 2004, unless otherwise noted.

§ 650.301 Purpose.

This subpart sets the national standards for the proper safety inspection and evaluation of all highway bridges in accordance with 23 U.S.C. 151.

§ 650.303 Applicability.

The National Bridge Inspection Standards (NBIS) in this subpart apply to all structures defined as highway bridges located on all public roads.

§650.305 Definitions.

Terms used in this subpart are defined as follows:

American Association of State Highway and Transportation Officials (AASHTO) Manual. "Manual for Condition Evaluation of Bridges," second edition, published by the American Association of State Highway and Transportation Officials (incorporated by reference, see §650.317).

Bridge. A structure including supports erected over a depression or an obstruction, such as water, highway, or railway, and having a track or passageway for carrying traffic or other moving loads, and having an opening measured along the center of the roadway of more than 20 feet between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes; it may also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening.

Bridge inspection experience. Active participation in bridge inspections in accordance with the NBIS, in either a field inspection, supervisory, or management role. A combination of bridge design, bridge maintenance, bridge construction and bridge inspection experience, with the predominant amount in bridge inspection, is acceptable.

Bridge inspection refresher training. The National Highway Institute "Bridge Inspection Refresher Training Course"[1] or other State, local, or federally developed instruction aimed to improve quality of inspections, introduce new techniques, and maintain the consistency of the inspection program.

Bridge Inspector's Reference Manual (BIRM). A comprehensive FHWA manual on programs, procedures and techniques for inspecting and evaluating a variety of in-service highway bridges. This manual may be purchased from the U.S. Government Printing Office, Washington, DC 20402 and from National Technical Information Service, Springfield, Virginia 22161, and is available at the following URL: *http://www.fhwa.dot.gov/bridge/bripub.htm.*

Complex bridge. Movable, suspension, cable stayed, and other bridges with unusual characteristics.

Comprehensive bridge inspection training. Training that covers all aspects of bridge inspection and enables inspectors to relate conditions observed on a bridge to established criteria (see the Bridge Inspector's Reference Manual for the recommended material to be covered in a comprehensive training course).

Critical finding. A structural or safety related deficiency that requires immediate follow-up inspection or action.

Damage inspection. This is an unscheduled inspection to assess structural damage resulting from environmental factors or human actions.

Fracture critical member (FCM). A steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse.

Fracture critical member inspection. A hands-on inspection of a fracture critical member or member components that may include visual and other nondestructive evaluation.

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[1] The National Highway Institute training may be found at the following URL: *http://www.nhi.fhwa.dot.gov./*

Hands-on. Inspection within arms length of the component. Inspection uses visual techniques that may be supplemented by nondestructive testing.

Highway. The term “highway” is defined in 23 U.S.C. 101(a)(11).

In-depth inspection. A close-up, inspection of one or more members above or below the water level to identify any deficiencies not readily detectable using routine inspection procedures; hands-on inspection may be necessary at some locations.

Initial inspection. The first inspection of a bridge as it becomes a part of the bridge file to provide all Structure Inventory and Appraisal (SI&A) data and other relevant data and to determine baseline structural conditions.

Legal load. The maximum legal load for each vehicle configuration permitted by law for the State in which the bridge is located.

Load rating. The determination of the live load carrying capacity of a bridge using bridge plans and supplemented by information gathered from a field inspection.

National Institute for Certification in Engineering Technologies (NICET). The NICET provides nationally applicable voluntary certification programs covering several broad engineering technology fields and a number of specialized subfields. For information on the NICET program certification contact: National Institute for Certification in Engineering Technologies, 1420 King Street, Alexandria, VA 22314-2794.

Operating rating. The maximum permissible live load to which the structure may be subjected for the load configuration used in the rating.

Professional engineer (PE). An individual, who has fulfilled education and experience requirements and passed rigorous exams that, under State licensure laws, permits them to offer engineering services directly to the public. Engineering licensure laws vary from State to State, but, in general, to become a PE an individual must be a graduate of an engineering program accredited by the Accreditation Board for Engineering and Technology, pass the Fundamentals of Engineering exam, gain four years of experience working under a PE, and pass the Principles of Practice of Engineering exam.

Program manager. The individual in charge of the program, that has been assigned or delegated the duties and responsibilities for bridge inspection, reporting, and inventory. The program manager provides overall leadership and is available to inspection team leaders to provide guidance.

Public road. The term “public road” is defined in 23 U.S.C. 101(a)(27).

Quality assurance (QA). The use of sampling and other measures to assure the adequacy of quality control procedures in order to verify or measure the quality level of the entire bridge inspection and load rating program.

Quality control (QC). Procedures that are intended to maintain the quality of a bridge inspections and load rating at or above a specified level.

Routine inspection. Regularly scheduled inspection consisting of observation and/or measurements needed to determine the physical and functional condition of the bridge, to identify any changes from initial or previously recorded conditions, and to ensure that the structure continues to satisfy present service requirements.

Routine permit load. A live load, which has a gross weight, axle weight or distance between axles not conforming with State statutes for legally configured vehicles, authorized for unlimited trips over an extended period of time to move alongside other heavy vehicles on a regular basis.

Scour. Erosion of streambed or bank material due to flowing water; often considered as being localized around piers and abutments of bridges.

Scour critical bridge. A bridge with a foundation element that has been determined to be unstable for the observed or evaluated scour condition.

Special inspection. An inspection scheduled at the discretion of the bridge owner, used to monitor a particular known or suspected deficiency.

Team leader. Individual in charge of an inspection team responsible for planning, preparing, and performing field inspection of the bridge.

Underwater diver bridge inspection training. Training that covers all aspects of underwater bridge inspection and enables inspectors to relate the conditions of underwater bridge elements to established criteria (see the Bridge Inspector’s Reference Manual section on underwater inspection for the recommended material to be covered in an underwater diver bridge inspection training course).

Underwater inspection. Inspection of the underwater portion of a bridge substructure and the surrounding channel, which cannot be inspected visually at low water by wading or probing, generally requiring diving or other appropriate techniques.

§ 650.307 Bridge inspection organization.

(a) Each State transportation department must inspect, or cause to be inspected, all highway bridges located on public roads that are fully or partially located within the State's boundaries, except for bridges that are owned by Federal agencies.

(b) Federal agencies must inspect, or cause to be inspected, all highway bridges located on public roads that are fully or partially located within the respective agency responsibility or jurisdiction.

(c) Each State transportation department or Federal agency must include a bridge inspection organization that is responsible for the following:

(1) Statewide or Federal agency wide bridge inspection policies and procedures, quality assurance and quality control, and preparation and maintenance of a bridge inventory.

(2) Bridge inspections, reports, load ratings and other requirements of these standards.

(d) Functions identified in paragraphs (c)(1) and (2) of this section may be delegated, but such delegation does not relieve the State transportation department or Federal agency of any of its responsibilities under this subpart.

(e) The State transportation department or Federal agency bridge inspection organization must have a program manager with the qualifications defined in §650.309(a), who has been delegated responsibility for paragraphs (c)(1) and (2) of this section.

§ 650.309 Qualifications of personnel.

(a) A program manager must, at a minimum:

(1) Be a registered professional engineer, or have ten years bridge inspection experience; and

(2) Successfully complete a Federal Highway Administration (FHWA) approved comprehensive bridge inspection training course.

(b) There are five ways to qualify as a team leader. A team leader must, at a minimum:

(1) Have the qualifications specified in paragraph (a) of this section; or

(2) Have five years bridge inspection experience and have successfully completed an FHWA approved comprehensive bridge inspection training course; or

(3) Be certified as a Level III or IV Bridge Safety Inspector under the National Society of Professional Engineer's program for National Certification in Engineering Technologies (NICET) and have successfully completed an FHWA approved comprehensive bridge inspection training course, or

(4) Have all of the following:

(i) A bachelor's degree in engineering from a college or university accredited by or determined as substantially equivalent by the Accreditation Board for Engineering and Technology;

(ii) Successfully passed the National Council of Examiners for Engineering and Surveying Fundamentals of Engineering examination;

(iii) Two years of bridge inspection experience; and

(iv) Successfully completed an FHWA approved comprehensive bridge inspection training course, or

(5) Have all of the following:

(i) An associate's degree in engineering or engineering technology from a college or university accredited by or determined as substantially equivalent by the Accreditation Board for Engineering and Technology;

(ii) Four years of bridge inspection experience; and

(iii) Successfully completed an FHWA approved comprehensive bridge inspection training course.

(c) The individual charged with the overall responsibility for load rating bridges must be a registered professional engineer.

(d) An underwater bridge inspection diver must complete an FHWA approved comprehensive bridge inspection training course or other FHWA approved underwater diver bridge inspection training course.

§ 650.311 Inspection frequency.

(a) *Routine inspections.* (1) Inspect each bridge at regular intervals not to exceed twenty-four months.

(2) Certain bridges require inspection at less than twenty-four-month intervals. Establish criteria to determine the level and frequency to which these bridges are inspected considering such factors as age, traffic characteristics, and known deficiencies.

(3) Certain bridges may be inspected at greater than twenty-four month intervals, not to exceed forty-eight-months, with written FHWA approval. This may be appropriate when past inspection findings and analysis justifies the increased inspection interval.

(b) *Underwater inspections.* (1) Inspect underwater structural elements at regular intervals not to exceed sixty months.

(2) Certain underwater structural elements require inspection at less than sixty-month intervals. Establish criteria to determine the level and frequency to which these members are inspected considering such factors as construction material, environment, age, scour characteristics, condition rating from past inspections and known deficiencies.

(3) Certain underwater structural elements may be inspected at greater than sixty-month intervals, not to exceed seventy-two months, with written FHWA approval. This may be appropriate when past inspection findings and analysis justifies the increased inspection interval.

(c) *Fracture critical member (FCM) inspections.* (1) Inspect FCMs at intervals not to exceed twenty-four months.

(2) Certain FCMs require inspection at less than twenty-four-month intervals. Establish criteria to determine the level and frequency to which these members are inspected considering such factors as age, traffic characteristics, and known deficiencies.

(d) Damage, in-depth, and special inspections. Establish criteria to determine the level and frequency of these inspections.

§ 650.313 Inspection procedures.

(a) Inspect each bridge in accordance with the inspection procedures in the AASHTO Manual (incorporated by reference, *see* §650.317).

(b) Provide at least one team leader, who meets the minimum qualifications stated in §650.309, at the bridge at all times during each initial, routine, in-depth, fracture critical member and underwater inspection.

(c) Rate each bridge as to its safe load-carrying capacity in accordance with the AASHTO Manual (incorporated by reference, *see* §650.317). Post or restrict the bridge in accordance with the AASHTO Manual or in accordance with State law, when the maximum unrestricted legal loads or State routine permit loads exceed that allowed under the operating rating or equivalent rating factor.

(d) Prepare bridge files as described in the AASHTO Manual (incorporated by reference, *see* §650.317). Maintain reports on the results of bridge inspections together with notations of any action taken to address the findings of such inspections. Maintain relevant maintenance and inspection data to allow assessment of current bridge condition. Record the findings and results of bridge inspections on standard State or Federal agency forms.

(e) Identify bridges with FCMs, bridges requiring underwater inspection, and bridges that are scour critical.

(1) Bridges with fracture critical members. In the inspection records, identify the location of FCMs and describe the FCM inspection frequency and procedures. Inspect FCMs according to these procedures.

(2) Bridges requiring underwater inspections. Identify the location of underwater elements and include a description of the underwater elements, the inspection frequency and the procedures in the inspection records for each bridge requiring underwater inspection. Inspect those elements requiring underwater inspections according to these procedures.

(3) Bridges that are scour critical. Prepare a plan of action to monitor known and potential deficiencies and to address critical findings. Monitor bridges that are scour critical in accordance with the plan.

(f) *Complex bridges.* Identify specialized inspection procedures, and additional inspector training and experience required to inspect complex bridges. Inspect complex bridges according to those procedures.

(g) *Quality control and quality assurance.* Assure systematic quality control (QC) and quality assurance (QA) procedures are used to maintain a high degree of accuracy and consistency in the inspection program. Include periodic field review of inspection teams, periodic bridge inspection refresher training for program managers and team leaders, and independent review of inspection reports and computations.

(h) *Follow-up on critical findings.* Establish a statewide or Federal agency wide procedure to assure that critical findings are addressed in a timely manner. Periodically notify the FHWA of the actions taken to resolve or monitor critical findings.

§ 650.315 Inventory.

(a) Each State or Federal agency must prepare and maintain an inventory of all bridges subject to the NBIS. Certain Structure Inventory and Appraisal (SI&A) data must be collected and retained by the State or Federal agency for collection by the FHWA as requested. A tabulation of this data is contained in the SI&A sheet distributed by the FHWA as part of the “Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges,” (December 1995) together with subsequent interim changes or the most recent version. Report the data using FHWA established procedures as outlined in the “Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges.”

(b) For routine, in-depth, fracture critical member, underwater, damage and special inspections enter the SI&A data into the State or Federal agency inventory within 90 days of the date of inspection for State or Federal agency bridges and within 180 days of the date of inspection for all other bridges.

(c) For existing bridge modifications that alter previously recorded data and for new bridges, enter the SI&A data into the State or Federal agency inventory within 90 days after the completion of the work for State or Federal agency bridges and within 180 days after the completion of the work for all other bridges.

(d) For changes in load restriction or closure status, enter the SI&A data into the State or Federal agency inventory within 90 days after the change in status of the structure for State or Federal agency bridges and within 180 days after the change in status of the structure for all other bridges.

§ 650.317 Reference manuals.

(a) The materials listed in this subpart are incorporated by reference in the corresponding sections noted. These incorporations by reference were approved by the Director of the Federal Register in accordance with 5 U.S.C. 552(a) and 1 CFR part 51. These materials are incorporated as they exist on the date of the approval, and notice of any change in these documents will be published in the Federal Register. The materials are available for purchase at the address listed below, and are available for inspection at the National Archives and Records Administration (NARA). These materials may also be reviewed at the Department of Transportation Library, 400 Seventh Street, SW., Washington, DC, in Room 2200. For information on the availability of these materials at NARA call (202) 741–6030, or go to the following URL:*http://www.archives.gov/federal_register/code_of_federal_regulations/ibr_locations.html.* In the event there is a conflict between the standards in this subpart and any of these materials, the standards in this subpart will apply.

(b) The following materials are available for purchase from the American Association of State Highway and Transportation Officials, Suite 249, 444 N. Capitol Street, NW., Washington, DC 20001. The materials may also be ordered via the AASHTO bookstore located at the following URL: *http://www.aashto.org/aashto/home.nsf/FrontPage.*

(1) The Manual for Condition Evaluation of Bridges, 1994, second edition, as amended by the 1995, 1996, 1998, and 2000 interim revisions, AASHTO, incorporation by reference approved for §§650.305 and 650.313.

(2) 2001 Interim Revision to the Manual for Condition Evaluation of Bridges, AASHTO, incorporation by reference approved for §§650.305 and 650.313.

(3) 2003 Interim Revision to the Manual for Condition Evaluation of Bridges, AASHTO, incorporation by reference approved for §§650.305 and 650.313.

Chapter 29

BRIDGE MANAGEMENT

NDOT STRUCTURES MANUAL

September 2008

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Chapter 29
BRIDGE MANAGEMENT

29.1 PONTIS

29.1.1 General

PONTIS® is an AASHTO bridge management software package that relies upon collected condition data and cost data for bridge elements (e.g., girders, piers, railings). This data is analyzed to identify least-cost (optimal), long-term preservation and improvement policies for a network of bridges.

PONTIS can play a vital role in NDOT's asset-management process by allocating resources to preserve the existing infrastructure investment, ensure safety and maintain mobility. PONTIS stores inventory and inspection information on bridges, culverts and other structures in a relational database that supports modeling, analysis and reporting tools to facilitate project, budget and program development. PONTIS can assist in the formulation of network-wide preservation and improvement policies for use in evaluating the needs of each structure in the network, and it can make project recommendations for the NDOT program of capital projects. It also can analyze the impact of various project alternatives on the performance of individual structures or a network of structures.

29.1.2 NDOT Status

NDOT has been a subscriber to PONTIS since 1996. NDOT currently uses PONTIS to warehouse the State's NBI data and to collect and store all element-level bridge inspection data. To implement a formal bridge management system, NDOT must take the following steps:

- Refine the list of bridge elements, condition state definitions and environmental assignment policies.
- Incorporate additional data, such as Legal and Design standard data, Improvement cost data, User cost data and Preservation cost data.
- Develop preservation policies.
- Develop programming scenarios (e.g., Budget sets, Agency policy rule sets, Scenario cost and time thresholds).
- Model the development and validation of PONTIS.
- Implement project planning and development.

29.1.3 The Bridge Management Process

The PONTIS bridge management process begins with the building of a relational database that includes importing NBI data and adding element-level inspection information. The process continues with the development of a preservation policy for each element and environmental combination in the database. PONTIS performs a network analysis to identify needs and benefits and to support the allocation of resources for the development of specific preservation and improvement program recommendations. From these recommendations, engineers and managers can use the Project Planning module to identify an initial program of work. The PONTIS bridge management process also includes tools for refining results, incorporating agency business practices, and allowing project tracking through the database.

29.1.4 Elements

The concept of bridge elements is the foundation of the PONTIS Bridge Management System. PONTIS uses element-level inspection data as the basis for bridge preservation analyses. The main components of a typical bridge (e.g., deck, superstructure, substructure) are subdivided into numerous elements to add more detail and precision. A superstructure might contain several elements such as concrete girders, concrete bridge deck, bearings, etc. Elements are also classified by material types (e.g., concrete, steel, timber). Through element-level inspections, NDOT quantifies the condition of structures. Each bridge element is assigned an element number and a description. The unit quantity for an element is placed in a Condition State. There are up to five available Condition States, 1 to 5 for an element. Condition State 1 is the best possible. PONTIS uses this information to compute the costs and benefits of bridge preservation. All of NDOT's bridges can be defined from a set of commonly recognized, or CoRe, elements as defined by AASHTO. See the AASHTO *Guide for Commonly Recognized (CoRe) Structural Elements*. Element level data is provided in accordance with procedures outlined in the "NDOT PONTIS Coding Guide" (see Appendix 29A).

29.1.5 Bridge Inspection

Bridge inventory and inspection data are managed using the PONTIS Inspection module. This consists of a set of relational tables and graphical user interfaces for creating new structures, deleting structures, reviewing existing data, entering new inspection data with comments, or checking data out of or into the system. This is a highly customizable data structure that includes the required National Bridge Inventory data, detailed element-level condition inspection data and custom agency data. The user may use the following programs within the Inspection module for a bridge, a group of bridges or all bridges in the database:

1. Translator. This functionality is not used by NDOT. NDOT conducts both NBI and PONTIS element level inspections, thus inputting codes for NBI Items 58 through 62. Translator is embedded in the PONTIS software and can convert the AASHTO CoRe Structural Elements information to the NBI condition codes, Items 58, 59, 60 and 62. FHWA will accept the results of Translator into the National Bridge Inventory.

2. Sufficiency Rating. This program calculates the Sufficiency Rating, Structure Evaluation Rating, Deck Geometry Rating, Vertical and Horizontal Rating and Structurally Deficient or Functionally Obsolete status.

3. Validation. This program uses the FHWA Edit Update Program to perform data validation checks of the most recent NBI data. Validation results display the Bridge ID, FHWA Error ID, Error Severity and Validation Message.

PONTIS uses the most recent data from the Inspection module to determine network-level and bridge-level preservation and improvement needs.

Chapter 28 discusses the Nevada Bridge Inspection Program.

29.1.6 Preservation and Improvements

PONTIS makes a distinction between preservation actions and functional improvements. Preservation actions seek to maintain or restore the physical condition of structure elements. Improvement actions are intended to improve the structure to satisfy the current and future functional demands. Preservation actions include maintenance, repair, rehabilitation or replacement of elements or groups of elements. The preservation model identifies the optimal preservation actions based on the objective of minimizing costs. Standard types of functional improvement actions include bridge widening, raising the bridge, strengthening and replacement. Programming of improvement actions is based on policy standards (e.g., lane and shoulder widths, vertical and horizontal clearances, unit costs and benefits supplied by the user). PONTIS is designed to support the testing of different combinations of functional improvement policies and cost-and-benefit assumptions. PONTIS analyzes preservation and improvement alternatives separately and incorporates them into a network analysis.

29.1.7 Network Level Analysis

A PONTIS simulation models a recommended bridge program for a network of bridges based on a set of scenario parameters. Scenario parameters include budgets, policies, costs and types of work. The goal of a simulation is to generate project alternatives for each bridge by maximizing benefits within a constrained budget. The competing alternatives are preservation, preservation plus improvement, replacement and user-defined projects. These are ranked and recommended for programming according to an incremental benefit-cost analysis. The results of a network-level analysis serve as a good starting point for the project development and programming process.

29.1.8 Project-Level Analysis

In the Project Planning module, the results of the network analyses and inspector work candidates are used to generate projects. Projects consist of specific work items on one or more bridges. Programs are developed from groups of projects for a specified period of time and types of work. The module includes a tool for running a single bridge simulation to evaluate the impact of a proposed set of actions before committing to a project. Projects are assigned to programs (e.g., Statewide Maintenance, Interstate Maintenance, Highway Bridge Program). Once a program has been developed, the network analysis can be rerun to further refine the results.

29.1.9 Reviewing and Refining Results

Three methods are available for reviewing the results of a PONTIS analysis. The Results module provides a report builder for needs, programmed work and performance measures for any network scenario. The Reports button provides access to additional reports, and the Work Candidates Panel in the Project Planning module provides details on scenario and inspector work candidate recommendations. By modifying scenario parameters for budgets, improvement policies and improvement costs, PONTIS refines the results to reflect agency business practices. Further refinements are made by defining Simulation Rules for scoping, look ahead for user programmed work and major rehabilitation. Preservation policies can be refined by creating and using the agency policy rules or by adjusting preservation model elicitations. Users may perform what-if analyses by running multiple scenarios to compare results for different combinations of parameters.

Appendix 29A

PONTIS CODING GUIDE

29A.1 Introduction

Appendix 29A presents a NDOT supplement to the AASHTO *Guide for Commonly Recognized (CoRe) Structural Elements* (the *Guide*) to assist the bridge inspector with the evaluation and coding of PONTIS “CoRe” (Commonly Recognized) and “Smart Flag” element Condition State Ratings. The PONTIS Condition State Ratings specified for each CoRe elements differ from the Numerical Condition Ratings (NCRs) used to encode bridge component data for the Bridge Inspection Report and for Items #58-62 of the Structure Inventory and Appraisal (SI&A) Report. PONTIS Element ratings are PONTIS-specific and follow specific Condition State definitions, which are provided for each CoRe and Smart Flag element. The *Guide* presents Condition State descriptions for CoRe elements and Smart Flags not specified herein. When encoding this PONTIS data, these definitions must be strictly followed.

For PONTIS to accurately model deterioration rates for a specific bridge, PONTIS bridge element data must be obtained during successive inspections gathered and encoded in a similar fashion. All inspectors must, therefore, follow the same conventions involving data acquisition and data entry. Specific conventions to be replicated involve unit of measurement assignment, girder tabulations and condition state assignment.

29A.2 Unit of Measurement Assignment Conventions

These conventions address the specific units of measurement that will be assigned to certain deficiencies as they occur in bridge elements that are rated per linear foot.

29A.2.1 Cracking

Regarding horizontal or diagonal cracking, the number of linear meters of the element affected by this cracking will be recorded, rounding up to the nearest meter. Regarding vertical cracking in an element, the inspector shall count the number of significant cracks (structural cracks, not including paint cracking) in the element, and assign one-half linear meter to each crack, provided that cracks are at a separation distance of greater than one-half meter. For cracks less than one-half meter apart, simply record the overall affected length of cracking, rounding up as appropriate. For map or alligator cracking, record the linear meters of the affected area, rounding up to the nearest meter.

Cracking in opposing sides of an element (e.g., cracking in opposite sides of a pier cap or pier wall) requires careful data recording, because PONTIS records only the physical length of most elements and not the cumulative length of all faces. Thus, cracking that occurs in opposite faces of a member, but occupies the same linear dimension along the member’s length, will only be recorded as that affected length times a factor of one. If cracking occurs in opposite faces of a member and in different locations along the member’s length, then the sum of both affected lengths should be recorded.

29A.2.2 Spalling/Scaling/Delamination

Spalling, scaling, delamination and other minor to moderate deterioration that occurs in an element will be assigned a minimum measurement of one-half meter of length for each area of damage, provided that such areas are more than one-half meter apart. For areas of deterioration measuring greater than one-half meter in length and for areas less than one-half meter apart, record the overall length of the affected area, rounding up to the nearest meter. Areas of deterioration occurring on opposite sides of a member shall be treated similarly to the cracking scenario discussed previously.

29A.2.3 Severe Damage/Deterioration

Bridge members that exhibit severe damage or deterioration, whether due to vehicular impact, chemical attack, flooding, settlement or any other cause, require considerable engineering judgment and common sense by the inspector. In evaluating a severely damaged AASHTO I-girder, for example, the inspector must either subdivide the length of the girder into various lengths meeting the criteria for Condition States 1, 2, 3 and 4, or determine that the entire girder requires replacement; in this case, its entire length should be placed into Condition State 4.

In most cases, for members that have sustained damage sufficiently severe to require replacement and members that have failed, their entire lengths (or per each unit) shall be rated in the lowest condition state rating available. The subdivision of lengths into various condition states will typically be relegated to those members exhibiting repairable damage. If there is uncertainty when rating severe damage, seek the advice of experienced individuals.

29A.2.4 Affected Length of Arch/Truss Member Deficiencies

All measurements for deterioration in arch or truss members shall be the measurement of the affected length along the horizontal projection of the arch or truss. See Figure 29A-2 for a sample illustration.

29A.3 <u>Girder Tabulation Conventions</u>

Another important convention is the tabulation on the number of individual girders present on a given bridge. In general, girder tabulation will follow the format presented in Figure 29A-1.

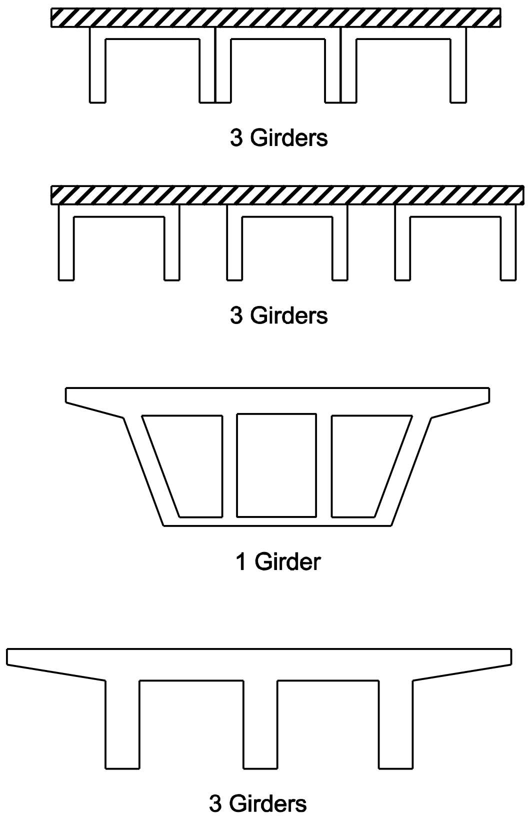
Where two or more different types of girders exist in the same bridge, the total number of girders for each separate type shall be tabulated and rated independently, following the format presented in Figure 29A-2. For additional information, consult the specific PONTIS CORE elements for open and box girders from the element listings in Sections 29A.5.3 and 29A.5.4.

29A.4 <u>Condition State Assignment Conventions</u>

The assignment of various condition states to a bridge element often requires considerable engineering judgment and an adherence to the exact condition state definitions (as listed in the *Guide*) for that element.

The following sections provide NDOT's conventions for the coding of condition states for the various PONTIS CoRe and Smart Flag bridge elements.

Note: In general, the "number of girders" in the PONTIS inventory for a bridge can be determined from the number of girders that are visible to the engineer during a field investigation. See the following examples.



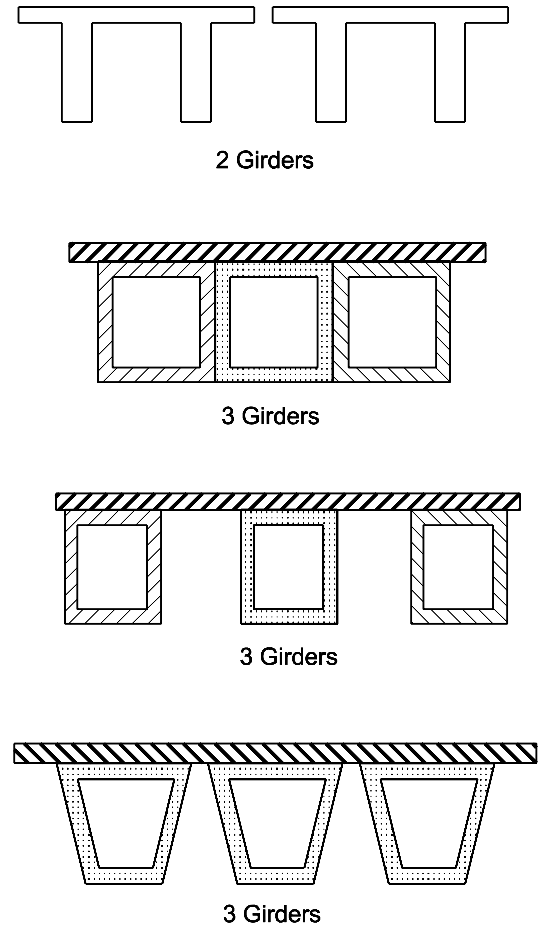
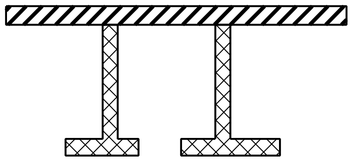
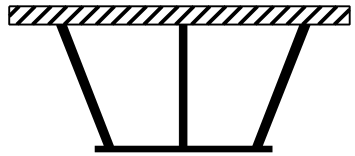


Figure 29A-1— CONVENTIONS FOR PONTIS

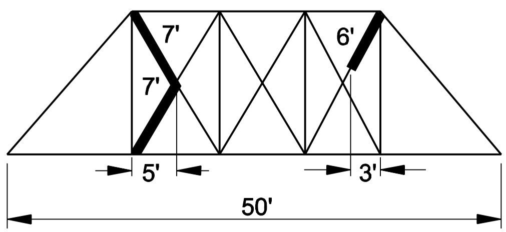


2 Girders



1 Girder

Truss and Arch Measurements



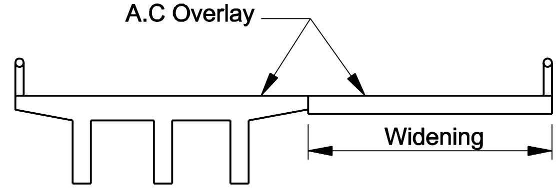
Deteriorated Parts of Truss

All measurements of a truss are along the horizontal projection, including the deterioration measurements. The total length of the above truss is 50' while the recorded length of the deteriorated part of the truss is equal to 8' (5' + 3').

The convention used for a spandrel arch will be similar to that used for a truss.

The unit for coding the Deck/Slab PONTIS bridge elements is "EACH". Thus, when the bridge has one Deck/Slab type, the quantity is 1 (It does not matter how many spans there are). However, when the same bridge has two different types of Deck/Slab types (Example: Elements 13 and 39), record the quantity equal to 1 for each element.

Example:



Typical Section

Elem. No.	Description	Quantity	Unit
13	Concrete Deck (AC)	1	Ea
39	Concrete Slab (AC)	1	Ea

In addition to the other substructure, superstructure and miscellaneous elements.

Figure 29A-2 — CONVENTIONS FOR PONTIS

29A.5 Core Bridge Elements

29A.5.1 Deck, Element Nos. 12-32, Rating Units: Ea

This element shall be used to provide PONTIS ratings for the bridge deck and/or deck surface covering, if present. Rating criteria specific to different deck material types is:

1. Concrete Deck (Element Nos. 12-27). For bridges using a concrete deck, the condition states reference only deficiencies in the deck surface. Specifically, PONTIS references only repaired areas; potholes and impending potholes for decks with asphalt overlays; adding spalls and delaminated areas as viable deficiencies in bare decks; and decks either covered with thin or rigid overlays or protected with epoxy-coated bars or cathodic protection. These deficiencies will be rated based upon their combined area of distress as a percentage of deck area, as outlined in the condition state descriptions.

   The Smart Flag “Deck Cracking” (Element No. 358) will be used to rate cracking in the deck. Condition state descriptions for this element, as illustrated in both the PONTIS program and PONTIS *User’s Manual*, are self-explanatory. When deficiencies more significant than cracking begin to appear in the deck, the use of this Smart Flag should be discontinued.

   The Smart Flag “Soffit” (Element No. 359) may be used to rate deck distresses visible in the underside of a concrete deck as a result of internal corrosion. Further, “Soffit” may be used to rate deck distresses evident when the surface of the deck is covered with an overlay and, thus, is not visible.

   A problem arises when the deck top of a structure is covered with an overlay, and the entire deck underside is also hidden from view via stay-in-place formwork. In this case, one can only rate the condition of the deck surfacing using standard condition state descriptions.

2. Steel Deck — Open/Concrete Filled Grid (Element Nos. 28-29). These elements provide PONTIS ratings for the condition of the entire steel deck and all connection devices (e.g., welds, rivets). Condition state conditions for these elements are self-explanatory.

3. Deck — Corrugated/Orthotropic/etc. (Element No. 30). This element is used to provide PONTIS condition ratings for concrete-filled corrugated metal decks or orthotropic decks. Orthotropic decks are often found in Nevada on railroad bridge structures. Condition state definitions for this element are self-explanatory.

4. Wood Deck — Bare or Overlaid (Element Nos. 31-32). These elements are used to provide PONTIS ratings for the condition of all surfaces of timber bridge decks, whether bare or overlaid with an asphalt surfacing. Condition state definitions are self-explanatory.

5. Combination Deck/Slab Types. For bridges constructed using two or more different types of deck or slab construction, each deck/slab type should be documented and rated independently, using the appropriate condition state definitions.

29A.5.2 Slabs, Element Nos. 38-55, Rating Units: Ea

These elements will be used to provide PONTIS ratings for the condition of deck slabs constructed of either concrete or timber. Evaluation criteria for these materials is essentially identical to that previously presented for concrete and timber bridge decks. Smart Flags “Deck Cracking” (Element No. 358) and “Soffit” (Element No. 359) may be used for concrete slabs as previously described.

Combination slab/deck types should be documented and rated independently, as previously described under “Combination Deck/Slab Types.”

29A.5.3 Closed Web/Box Girder, Element Nos. 101-105, Rating Units: Meter

This element will be used to provide PONTIS ratings for the condition of box girders constructed of steel, conventionally reinforced concrete or prestressed concrete.

The total number of linear meters of box girder to be entered into PONTIS as “Total Quantity” shall be the total number of girders multiplied by total linear meters of each. The number of girders shall be determined by following the girder measurement conventions as shown in Figures 29A-1 and 29A-2. In general, precast box girders placed side-by-side shall be counted as one per individual box; a cast-in-place box girder, which may contain several internal cells, shall be counted as only one girder. When precast girders exist alongside a cast-in-place box girder (as in many widened bridges), the total linear meters of each box girder type shall be calculated, then the subtotals merged, if of common element type (i.e., both prestressed).

The total number of linear meters shall be subdivided into different condition states following the specific condition state definitions for that element. Condition state definitions for closed web/box girder elements are self-explanatory.

29A.5.4 Open Girder, Element Nos. 106-111 (Excluding 108), Rating Units: Meter

These elements shall be used to provide PONTIS ratings for the condition of open girders constructed of steel, conventionally reinforced concrete, prestressed concrete or timber.

The total number of linear meters of open girder to be entered into PONTIS as “Total Quantity” shall be the number of girders multiplied by the number of linear meters of each. The total number of linear meters shall be subdivided into different condition states following the specific condition state definitions for that element. Condition state definitions for open girder elements are self-explanatory.

Unlike the SBIS inspection reporting methodology, multiple open-girder types co-existing on the same bridge will all be rated as open girders, and not girders and stringers. Different open girder types shall simply be subdivided into their proper open girder element numbers and be rated therein.

29A.5.5 Railroad Car Girder, Element No. 108, Rating Units: Meter

This element, specifically adopted by NDOT, will be used to provide PONTIS ratings for the condition of railroad flatcar “girders,” which act as superstructures for many rural Nevada bridges. These girders, most often constructed of painted steel, will be rated using the painted steel condition state descriptions provided for Element No. 107 “Painted Steel Open Girder.”

This element will be rated in linear meters (no. girders x no. linear meters each = "Total Quantity"), with an entire, singular flatcar considered as one girder. Where multiple, side-by-side flatcars make up a bridge deck, each flatcar shall be considered a separate girder.

29A.5.6 Stringer, Element Nos. 112-117, Rating Units: Meter

These elements shall be used to provide PONTIS ratings for the condition of stringers constructed of steel, prestressed concrete, conventionally reinforced concrete or timber. For PONTIS purposes, stringer elements will include only those members (typically longitudinal in orientation) that support the deck in true stringer-floor beam systems. Superstructure members of longitudinal orientation that do not juncture with floor beams will usually be classified as "Open Girders."

Stringers will be rated in linear meters, with "Total Quantity" calculated by multiplying the total number of stringers of given type by the total linear meters of each. Stringers of different type will be subdivided into different element numbers and rated therein, using the appropriate condition state definitions. Condition state definitions for stringers are self-explanatory.

29A.5.7 Steel Truss Members, Element Nos. 120-130, Rating Units: Meter

These elements shall be used to provide PONTIS ratings for the condition of steel truss members. All members will be rated in linear meters, measured along the horizontal projection of the span, as shown in Figure 29A-2.

Steel truss members will be rated using the condition state definitions of the appropriate element number. All definitions are self-explanatory.

29A.5.8 Timber Truss/Arch, Element No. 135, Rating Units: Meter

This element will be used to provide PONTIS ratings for the condition of all members of truss and arch structures that are constructed of timber. All members will be rated in linear meters, measured along the horizontal projection of the span, as with steel truss members. Condition state definitions for this element are self-explanatory.

29A.5.9 Arch, Element Nos. 140-145, Rating Units: Meter

These elements will be used to provide PONTIS ratings for the condition of arches constructed of steel, prestressed concrete, conventionally reinforced concrete or masonry. All arch members will be rated in linear meters, measured along the horizontal projection of the span, as with truss members. Condition state definitions for arch members are self-explanatory.

29A.5.10 Cable (Not Embedded in Concrete), Element Nos. 146-147, Rating Units: Ea

These elements will be used to provide PONTIS ratings for the condition of coated and uncoated metal cables (e.g., main and suspender cables of suspension bridges, hangers of tied arches, cables of cable-stayed bridges). Cables will be rated as "each" (or each cable of a system) and will use a unique set of condition state definitions that evaluates the cable itself, including cable banding and anchorages. Element No. 147 also evaluates the condition of the protective cable coating.

29A.5.11 Floor Beam, Element Nos. 151-156, Rating Units: Meter

These elements will be used to provide PONTIS ratings for the condition of floor beams constructed of steel, conventionally reinforced concrete, prestressed concrete and timber. These elements will be used to rate stand-alone floor beams and floor beams used in stringer/floor beam systems.

Floor beams will be rated in linear meters with “Total Quantity” calculated by multiplying the number of floor beams times the linear meters of each. Condition state definitions for use in the breakdown of total linear meters into various condition states are standard for the various material types and are self-explanatory.

29A.5.12 Pin-and-Hanger Assembly, Element Nos. 160-161, Rating Units: Ea

These elements will be used to provide PONTIS ratings for the condition of steel pins and/or pin-and-hanger assemblies used in bridge superstructures. Examples include pin connections in trusses and pin-and-hanger assemblies in girders. Pins used in bearing assemblies must not be rated with these elements but, rather, as bearings. Pins and pin-and-hanger assemblies must be rated per each using the appropriate condition state descriptions, which are self-explanatory.

29A.5.13 Column or Pile Extension, Element Nos. 201-206, Rating Units: Ea

These elements will be used to provide PONTIS ratings for the condition of piles or columns constructed of steel, conventionally reinforced concrete, prestressed concrete or timber.

When rating piles or columns only, the portions of each member that are clearly visible above the groundline will be rated. The underground portions of each are assumed to be in good condition (Condition State 1) if no visible distress exists in the member that can be directly attributed to an underground deficiency in that member.

Columns and piles will be rated per each, in accordance with the appropriate condition state definitions, which are self-explanatory. For structures using more than one type of pile or column, group each type of member into its proper element number and rate therein.

29A.5.14 Pier Wall, Element Nos. 210-211, Rating Units: Meter

These elements will be used to provide PONTIS ratings for the condition of pier walls or shafts constructed of either reinforced concrete or any “other” material (e.g., masonry, concrete-filled steel sheet piling).

A distinction is necessary between what constitutes a pier wall vs a column. Much of this decision lies with the judgment of the inspector, because no clear-cut formal distinctions have been made in the bridge inspection training literature. However, columns are rated per each, but pier walls are rated per linear meter. Therefore, it may benefit the inspector to rate as pier walls those members that can easily be subdivided and rated on a per-linear-meter basis.

Members with clearly definable long and short faces, whether flared or straight, shall usually be termed pier walls for PONTIS.

Pier walls shall be rated following the condition state definitions for either “Reinforced Concrete” or “Other.” Both sets of definitions are self-explanatory.

29A.5.15 Abutment, Element Nos. 215-217, Rating Units: Meter

These elements will be used to provide PONTIS ratings for the condition of bridge abutments constructed of either reinforced concrete, timber or any “other” suitable material (e.g., masonry, concrete-filled steel sheet piling).

For PONTIS, abutments will be rated in linear meters along the length of the stem wall (and/or backwall) portions <u>only</u>. PONTIS does not incorporate the wingwalls or retaining walls constructed adjacent to each abutment. For widened bridges, simply measure the new overall stem wall face widths, from outer edge of abutment to outer edge of abutment.

The condition state definitions for abutments are standard for the various material types and are self-explanatory.

29A.5.16 Submerged Pile Cap/Footing, Element No. 220, Rating Units: Ea

This element will be used to provide PONTIS ratings for the condition of reinforced concrete footings, whether or not supported by foundation (bearing) piling. Those footings that use foundation piling act as submerged pile caps, thus, contributing to the element name. All footings will be lumped into this element.

Footings will be rated per each, following the condition state definitions provided. Footings that are buried at the time of inspection will be placed into Condition State 1, unless there exists some above-ground condition indicative of underlying footing distress. In this case, the footing rating will be left to the discretion of the inspector. At this point, the inspector should also decide if further investigation is warranted.

29A.5.17 Submerged Pile, Element Nos. 225-228, Rating Units: Ea

These elements will be used to provide PONTIS ratings for the condition of submerged foundation piling, whether constructed of steel, prestressed concrete, conventionally reinforced concrete or timber. Foundation (bearing) piles will be rated in this element whether submerged or partially visible at the time of inspection.

Foundation piles that are buried at the time of inspection shall be placed into Condition State 1, unless there exists some above-ground condition indicative of underlying distress to the piling. In this case, the submerged pile rating will be at the discretion of the inspector. At this point, the inspector should also decide if further investigative action is warranted. Submerged piles will be rated per each using the appropriate condition state definitions, which are self-explanatory.

29A.5.18 Cap, Element Nos. 230-235, Rating Units: Meter

These elements will be used to provide PONTIS ratings for the condition of pier, bent or abutment caps constructed of steel, conventionally reinforced concrete, prestressed concrete or timber.

Caps will be rated in linear meters of length (number of caps multiplied by meters of length each = "Total Quantity") using the appropriate condition state definitions, which are self-explanatory. Bridges constructed using caps of differing materials should have those caps subdivided into proper element numbers by material type.

29A.5.19 Culvert, Element Nos. 240-243, Rating Units: Meter

These elements will be used to provide PONTIS ratings for the condition of box, arch and pipe culvert structures, constructed of metal, concrete, timber or any other suitable material. The Steel Culvert Element (No. 240) includes all metal types including steel, aluminum and galvanized materials. Additionally, the Concrete Culvert Element (No. 241) includes both conventionally reinforced and prestressed concrete materials, and the Other Culvert Element (No. 243) will include masonry and combinations of other materials.

Culverts will be rated on a linear meter basis, with the "Total Quantity" measurement encompassing the total length of all barrels combined (number of barrels multiplied by length of each). Deficiencies common to more than one barrel (e.g., through-cracking in barrel sidewalls) will be rated as occurring in each barrel affected.

Condition state definitions for use in rating culvert elements are self-explanatory.

29A.5.20 Expansion Joints, Element Nos. 300-304, Rating Units: Meter

These elements will be used to provide PONTIS ratings for Strip Seal Expansion Joints (Element No. 300), Pourable Joint Seals (Element No. 301), Compression Joint Seals (Element No. 302), Assembly (Modular) Joint Seals (Element No. 303) and Open Expansion Joints (Element No. 304). The intended use of Element Nos. 300, 301, 302 and 304 are self-explanatory, as are their respective condition state definitions.

In contrast, the intended usage of Element No. 303 "Assembly (Modular) Joint Seal" is somewhat vague. This element will be used to rate mechanical expansion joints, including sliding plate joints, fingerplate joints and modular elastomeric seals. Condition state definitions for this element are self-explanatory.

Rate expansion joints filled with only fiberboard "bond breaker" material (i.e., no poured sealant) as open expansion joints.

All expansion joints are rated in linear meters, with the total linear meters of all joints of similar type being recorded as the "Total Quantity" of each element.

29A.5.21 Bearings, Element Nos. 310-315, Rating Units: Ea

These elements will be used to provide PONTIS ratings for Elastomeric Bearings (Element No. 310), Moveable Bearings (Element No. 311), Enclosed/Concealed Bearings (Element No. 312), Fixed Bearings (Element No. 313), Pot Bearings (Element No. 314) and Disc Bearings (Element

No. 315). The intended use of Element Nos. 310-311 and 313-315 are self-explanatory, as are their respective condition state definitions.

In contrast, the intended use of Element No. 312 “Enclosed/Concealed Bearing” requires clarification. This element shall be used to rate bearings that are partially or completely hidden from view, thus compromising the physical inspection of the bearing member itself. An example of enclosed/concealed bearings might be a series of elastomeric bearing pads, used to support a prestressed box girder, and recessed far enough back upon the abutment bridge seat to be almost out of view.

The rating of such bearings comprises an evaluation of the bearing itself (as possible) and an evaluation of the bearing support and supported members, including noting excessive vertical and horizontal offsets and examining their structural condition and any movement under live loading.

Bearings are rated per each, with differing bearing types being subdivided and rated according to their proper element condition state definitions.

29A.5.22 Approach Slab, Element Nos. 320-321, Rating Units: Ea

These elements will be used to provide PONTIS ratings for the condition of approach slabs constructed of either conventionally reinforced or prestressed concrete. In Nevada, approach slabs are typically constructed using conventionally reinforced concrete (Element No. 321).

Approach slabs are rated per each (per individual slab) using the condition state definitions, which are self-explanatory.

29A.5.23 Bridge Railing, Element Nos. 330-333, Rating Units: Meter

These elements will be used to provide PONTIS ratings for the condition of bridge railings constructed of metal (all types and shapes), concrete, timber and miscellaneous (other) materials.

Element No. 333 “Miscellaneous – Bridge Railing” will be rated for all rail types and shapes other than metal, concrete or timber, and it includes those rails constructed using combinations of materials. An example of a combination rail is a rail constructed using timber posts and steel railing panels.

Bridge rails constructed using a metallic rail (e.g., tubular aluminum) mounted atop a concrete parapet shall be encoded listing both Element No. 330 (Metal Bridge Railing) and Element No. 331 (Concrete Bridge Railing), with the appropriate length of each type listed under “Total Quantity.”

Bridge railing total quantities will be subdivided into various condition states using the appropriate condition state definitions, which are self-explanatory.

29A.6 Smart Flags

29A.6.1 Steel Fatigue, Element No. 356, Rating Units: Ea

This element will be used to provide a rating for fatigue damage to a particular bridge. This smart flag shall be encoded only for those bridges where fatigue damage is known to exist.

Fatigue damage that has been repaired or arrested shall be placed into Condition State 1. First-time fatigue damage and newly documented additional fatigue damage shall be placed into Condition State 2. Fatigue damage severe enough to warrant analysis of the element to ascertain its serviceability shall be placed into Condition State 3. Rating is per each (per bridge).

29A.6.2 Pack Rust, Element No. 357, Rating Units: Ea

This element will be used to provide a rating for pack rust (impacted crevice corrosion) that already exists in a bridge. This element addresses crevice corrosion in steel connections and built-up members and rates the severity of resulting distress to that connection.

This element follows a set of self-explanatory condition state definitions and is rated per each (per bridge).

29A.6.3 Deck Cracking, Element No. 358, Rating Units: Ea

This element will be used to provide a rating for cracking that occurs in the deck surface of a bridge. This smart flag will typically be rated for those structures that exhibit deck cracking, but do not as yet exhibit the more significant deficiencies listed in the deck and slab core element condition state definitions (e.g., spalling, potholes, delaminated areas, repaired areas). Where these deficiencies exist, the use of the deck cracking smart flag should be discontinued.

A distinction is necessary between Condition States 2 and 3 for this element. Condition State 2 should be used to rate deck cracks that are moderate in either size or density (frequency of occurrence). Condition State 3 should be used when deck cracks are of moderate size and moderate density. Condition States 1 and 4 definitions are self-explanatory. This element is rated per each (per bridge).

29A.6.4 Soffit, Element No. 359, Rating Units: Ea

This element will be used to provide a rating for distresses visible in the underside of a deck or slab, primarily attributable to active corrosion within the member. This element should be rated when possible for bridges with a deck that is covered with an overlay. A soffit may, however, be rated at any time its inclusion into PONTIS is deemed useful to describe deterioration in the deck not reportable by other means.

This element is rated per each (per bridge) and uses a set of condition state definitions that are self-explanatory.

29A.6.5 Settlement, Element No. 360, Rating Units: Ea

This element will be used to provide a rating for substructure settlement (e.g., vertical settlement, rotation) found to exist at a bridge and to provide some measure of the magnitude of that settlement. This element is rated per each (per bridge).

This element uses only three condition states to rate settlement. Condition State 1 should be rated when visible settlement exists in the substructure and it appears to have stabilized. Condition State 2 rates settlement that appears to be continuing and could potentially cause problems if left unarrested. Settlement documented for the first time will usually be placed into Condition State 2, unless of a very minor nature. Condition State 3 should be used to rate severe settlement that threatens the integrity of the bridge to the extent that structural analysis is warranted.

29A.6.6 Scour, Element No. 361, Rating Units: Ea

This element will be used to provide a rating for scour at a bridge and to provide a measure of the magnitude of that scour. This element is rated per each (per bridge).

This element uses three condition states to rate bridge scour. Condition State 1 should be rated to document minor scour, but which is of little concern to the structural integrity of the bridge. Condition State 2 should be rated to document scour that could potentially threaten the structural integrity of the bridge if left unchecked. Pier or abutment scour that is documented for the first time and is found to have exposed portions of the footing of that element should be placed in Condition State 2. Additionally, previously documented scour that is found to be increasing in magnitude should be placed in this condition state. Condition State 3 will be used to document scour that is severe enough to warrant analysis of the bridge.

29A.6.7 Traffic Impact, Element No. 362, Rating Units: Ea

This element will be used to provide a rating for distress to any bridge element caused by traffic impact. This smart flag will primarily address superstructure damage caused by vehicular impact (high load hits). This element is rated per each (per bridge).

This element uses three condition states to rate traffic impact damage. Condition State 1 identifies impact damage that has been repaired. Condition State 2 will be used to document damage that has not been repaired and does not threaten the serviceability of the bridge. Condition State 3 should be rated to document damage that has impaired the strength of the impacted member, warranting analysis to determine the serviceability of the bridge.

29A.6.8 Section Loss, Element No. 363, Rating Units: Ea

This element will be used to provide a PONTIS rating for measurable loss of section experienced by a bridge element. Although most commonly associated with steel, this element could also be applied to section loss in elements composed of timber or even concrete. This element is rated per each (per bridge), and condition state definitions are self-explanatory.

Note that Condition State 1 is reserved for section loss that has been either repaired or cleaned and painted over. Measurable section loss documented for the first time, therefore, will be placed into Condition State 2, at a minimum.