Chapter IV – Structural Design
Table of Contents

Section 1: General Guidelines

Section 2: Bridge Division Operating Policies and Procedures

Section 3: Standard Specifications for Highway Bridges

Section 4: LRFD Bridge Design Specifications

Section 5: Railroad Structures

Section 6: Appendices

IV-01.01 Guidelines

This chapter contains NDDOT Bridge Division policies and procedures for the design, evaluation, and rehabilitation of structures on the North Dakota State Highway System. Use of this chapter does not relieve the design engineer of responsibility for the design of a bridge or structural component. Although Bridge Division policy is presented here for numerous situations, content of the chapter is not intended to be exhaustive. Therefore, use of this chapter must be tempered with sound engineering judgment. The basis for these guidelines is found in the current editions of the following reference publications:

1. AASHTO LRFD Bridge Design Specifications

2. Standard Specifications for Highway Bridges (AASHTO)

3. Standard Specifications for Road and Bridge Construction (NDDOT)

Those policies unique to the Standard Specifications for Highway Bridges are located in Section 3 and those policies unique to the LRFD Bridge Design Specifications are located in Section 4. Guidelines in Section 2 pertain to both specifications.

Prior to the beginning of a design, all applicable files in filenet and the plans of the existing structure shall be researched. The environmental documents must also be reviewed. Other items that shall be considered are the need for agreements or permits. Once all of these have been studied, a Preliminary Engineering meeting shall be scheduled with the Bridge Engineer. Persons in attendance shall include the Bridge Engineer, Assistant Bridge Engineer, section leader, designer of the project, and others as appropriate. See Appendix IV-06 D for Preliminary Engineering meeting discussion items. Bridge information determined from the Preliminary Engineering meeting shall be shared with the Design Division so that right of way limits can be established.

The primary design method for structures shall be Load & Resistance Factor Design (LRFD). Details on the LRFD design method can be found in reference publication 1 above. An HL-93 loading shall be used for the live load. In rehabilitating existing structures, the design method shall be Load Factor Design (LFD) with Working Stress Design (WSD) being used for designing piling. An HS 25 loading shall be used for the live load when practical in rehabilitating structures; otherwise an HS 20 loading shall be used.

The National Bridge Inspection Program requires that every bridge in the state be rated for Inventory and Operating loadings. The ratings shall be according to the AASHTO “Manual for Maintenance Inspection of Bridges” using an HS truck and WSD method except for NHS routes, new structures, and rehabilitated bridges, which shall be rated with the LRFR or LFD method. The Structural Management Section (Bridge) will rate the bridges.

IV-02.01 General Policies and Procedures

IV-02.01.01 Alternate Designs

The use of alternate designs can be discussed at the Preliminary Engineering meeting and a decision made whether or not alternate designs will be prepared.

IV-02.01.02 Plan Sheet Authorship

Initials of both the designer and detailer shall be placed on each plan sheet. The plan sheets shall be sealed by a registered Professional Engineer in accordance with Section I-12 of this manual.

IV-02.01.03 Structural Plan Sheet Sequence

The bridge and box culvert plan sheets are located in Section 170 of the plans. The general sequence of the bridge plan sheets is as follows (this is a guide only):

Bridge Layout
Notes
Screeds, Quantities and Miscellaneous
Piling Layout and Bearing Elevations
Abutment Underdrain and Excavation
Abutments
Piers
Beams/Girders
Slab Layout
Diaphragms
Endwalls
Slab Section
Reinforcing Bar List
Approach Slabs

The general sequence of the box culvert plan sheets is as follows (this is a guide only):

Box Culvert Layout
Notes
Excavation and Foundation Fill
Barrel Section and Wing
Floor
Wall and Parapet
Roof
Reinforcing Bar List
Bent Bar and Bar Cutting

IV-02.01.04 Standard Set of Plans

Examples of typical bridge and box culvert plan sets can be found on the Plan Preparation Guide at:

http://www.ugpti.org/dotsc/prepguide/

IV-02.01.05 Structure Numbers

All structures are identified by their structure number. The structure numbers for structures on the State Highway System are determined by the highway the structure is on, and the location of the structure on that highway as indicated by the Reference Point of the center of the structure. For example, Structure Number 2-146.366 means that the structure is located on US Highway 2 at Reference Point 146.366. The Planning/Asset Management Division is responsible for the establishment and maintenance of the reference point system. All reference point designations are extended to three decimal places. The structure number is permanent and does not change due to replacement, regrading, or other factors which might cause minor changes in the structure location reference point. All plan sheets shall contain the structure number, as well as all design records and any other records pertaining to each individual structure. New structure numbers are assigned by the Structural Management section leader.

IV-02.01.06 Structural Code Number

The structural code number, as shown below, shall be put on the upper right corner of the structural layout sheet. The structural code number is used when structural items are paid for separate from roadway items. The structural code number is required when there is major work such as a new bridge or bridge widening. The structural code number is not required when there is minor work such as approach slabs or rail retrofit.

1st DIGIT:	The first digit (code "X") indicates the bridge classification.
X _ _ _	Bridges (structures over 20 feet long)
2nd DIGIT:	The second digit indicates the main function of the structure.
X0 _ _	Highway over waterway
X1 _ _	Highway over railroad
X2 _ _	Highway over highway
X3 _ _	Highway over waterway and railroad
X4 _ _	Highway over waterway and highway
X5 _ _	Highway over railroad and highway
X6 _ _	Highway under railroad
X7 _ _	Highway under highway
X8 _ _	Highway under railroad and highway
X9 _ _	Other combinations, including highway over waterway, railroad, and highway; also 3- and 4-level grade separations and miscellaneous

Note: In determining whether to code X2 _ _ (Highway over highway), or X7 _ _ (Highway under highway), use the highway hierarchy system with the higher class highway going first. For example, in Bismarck where I-94 crosses over Washington Street, it would be coded X2 _ _; where I-94 goes under State Street (US 83), it would be coded X7 _ _.

3rd DIGIT: The third digit identifies the material of the principal supporting span members of the span identified by the fourth digit.

X _ 0 _	Timber
X _ 1 _	Masonry
X _ 2 _	Concrete, not prestressed
X _ 3 _	Steel
X _ 4 _	Steel and concrete
X _ 5 _	Timber and steel
X _ 6 _	Timber and concrete
X _ 7 _	Composite steel and concrete
X _ 8 _	Concrete, prestressed
X _ 9 _	Aluminum

4th DIGIT: The fourth digit identifies the type of span (identifies main span type if the bridge has two or more span types).

X _ _ 0	Slab
X _ _ 1	Girder
X _ _ 2	Truss (except cantilever)
X _ _ 3	Rigid beams
X _ _ 4	Arch
X _ _ 5	Cantilever truss
X _ _ 6	Moveable
X _ _ 7	Suspension
X _ _ 8	Box culvert (bridge length)
X909	Highway tunnels
Y009	Pedestrian overpasses or underpasses

IV-02.01.07 Design Live Load

The HL-93 or HS design live load shall be shown on the structural layout plan sheet.

IV-02.01.08 Elevations

Bridge elevations shall be shown on the structural layout sheet. The elevation of the roadway centerline shall be specified at begin bridge, end bridge, and over piers. The elevation of the bottom of all footings shall also be shown, as well as at other locations on the structure that the designer determines appropriate.

IV-02.01.09 Plan Notes, Special Provisions, Supplemental Specifications

Plan notes shall be developed for all projects. If a special provision is needed, it shall be requested from Technical Services in the Environmental and Transportation Services Division (ETS) in accordance with the instructions in Section I-10 of this manual. For coordination of plan notes, special provisions, standard specifications, and supplemental specifications, see Section 105.05 of the North Dakota Standard Specifications for Road and Bridge Construction.

IV-02.01.10 Shop Drawings

North Dakota Standard Specifications require shop drawings for prestressed concrete beams, precast reinforced concrete box culverts, and structural steel. Other structural items shall be called for in the plans.

IV-02.01.11 Structure Quantities

Structure quantities shall be given to the nearest whole number, with the exception of concrete, which shall be shown to the nearest tenth of a Cubic Yard, approach slabs, which shall be shown to the nearest tenth of a Square Yard, and prestressed concrete beams, which shall be shown to the nearest tenth of a Lineal Foot. The unit of payment for the various structural items shall be as stated in the North Dakota Standard Specifications for Road and Bridge Construction. Generally, Class 1 and Class 2 excavation shall be paid for by Lump Sum quantity. On projects where the excavation quantities are larger than normal, Class 1 and Class 2 excavation shall be paid for by Cubic Yards. Channel excavation shall be paid for by the Cubic Yard, but a plan note shall indicate that payment will be made for plan quantity only. Also, the deck concrete payment will be made for plan quantity only, as indicated by the North Dakota Standard Specifications.

IV-02.01.12 Rebar Designations

A standard rebar designation system shall be used for all bridge plans. The first digit of the bar mark indicates the bar size. The second character is a unique alpha character designating the shape of a straight or bent bar. An “X” preceding this alpha character signifies an epoxy coated bar. The last three digits designate the location of the bar. The 100 series is reserved for abutments, 200 series is reserved for piers, 500 series is reserved for the superstructure, and the 900 series is reserved for approach slabs. For example, a 5XA500 bar mark represents a #5 bar, epoxy coated straight bar (“A”), located in the superstructure.

IV-02.02 Geometrics

IV-02.02.01 Bridge Widths

The minimum clear roadway width shall be as specified in the minimum bridge width tables in Section I-06 of this manual. Certain urban roads may require different shoulder widths.

IV-02.02.02 Grade Recommendations

For drainage purposes, the minimum longitudinal grade shall be 0.2%. If the approach grade is flat, start a 0.2% grade at the ends of the approach slabs with a 50' vertical curve centered over the bridge. A maximum longitudinal grade of 3% is recommended. All bridges shall have a cross slope (transverse grade) of ¼" per foot. Other geometric requirements may supersede these recommendations.

IV-02.02.03 Bridge Skew Angle

It is preferred that the bridge skew angle not exceed 45°.

IV-02.02.04 Clearances

The minimum vertical clearances on grade separations shall be 16'-6". The minimum horizontal clearance on grade separations shall be the roadway clear zone from the edge of the driving lane unless piers are protected (i.e. 34'-0" for interstate highways). See Appendix IV-06 G for Interchange Clearance diagram. Exceptions shall meet requirements of AASHTO's "Roadside Design Guide."

IV-02.02.05 Horizontal Curves

In working with the geometry of horizontal curves, the arc definition shall be used.

IV-02.02.06 Navigational Clearance

When designing a structure where navigational clearance is an issue, the policies and procedures in the Code of Federal Regulations, 23CFR650H, shall be followed.

IV-02.02.07 Pedestrian & Shared-Use Facilities

When designing pedestrian and shared-use facilities for a structural project, the facilities shall be designed per AASHTO requirements, unless otherwise specified by Department Policy.
All highway projects provide an opportunity to enhance the safety and convenience of pedestrian and bicycle traffic. This can be accomplished by providing a pedestrian sidewalk or a shared-use path. An overpass, underpass, or a facility on a highway bridge may be necessary to provide continuity to a sidewalk or path.

To establish uniformity and consistency in the design and construction of pedestrian sidewalks and shared-use paths on a highway bridge or acting as a stand alone bridge, the following standards shall apply.

1. The maximum clear width for pedestrian sidewalks and shared-use paths shall be no greater than 10'-0".

2. Railings or barriers on a pedestrian sidewalk shall be a minimum of 42" on the outer edge.

3. Railings or barriers on a shared-use path shall be a minimum of 54" on the outer edge.

4. Bridge railing and the attachment to the deck overhang must satisfy crash testing requirements.

5. “traffic underneath” refers to motor vehicles on highways and train traffic.

6. “low speed” is defined as speeds not exceeding 45 mph.

The Department has developed four scenarios that establish the barrier and railing requirements when pedestrian sidewalks or shared-use paths are built as individual structures or as part of a highway bridge. Examples of these scenarios are located in Appendix IV-06 C, descriptions of which are found below.

1. a. Highway bridge that has a pedestrian sidewalk or shared-use path and traffic underneath (Appendix IV-06 C, Figure 1A)

   b. Low Speed highway bridge that has a pedestrian sidewalk or shared-use path and traffic underneath (Appendix IV-06 C, Figure 1B)

2. a. Pedestrian or shared-use bridge that has traffic underneath (Appendix IV-06 C, Figure 2A)

   b. Pedestrian bridge that has no traffic underneath (Appendix IV-06 C, Figure 2B)

   c. Shared-use bridge that has no traffic underneath (Appendix IV-06 C, Figure 2C)

3. a. Highway bridge with a pedestrian sidewalk and no traffic underneath (Appendix IV-06 C, Figure 3A)

   b. Highway bridge with a shared-use path and no traffic underneath (Appendix IV-06 C, Figure 3B)

4. a. Low Speed highway bridge with a pedestrian sidewalk and no traffic underneath (Appendix IV-06 C, Figure 4A)

   b. Low Speed highway bridge with a shared-use path and no traffic underneath (Appendix IV-06 C, Figure 4B)

IV-02.03 Foundations

IV-02.03.01 Bearing Piles

Plans shall show the required pile bearing capacity for each pile size. The required pile bearing shown shall be the maximum allowed by AASHTO specifications for that size pile unless the Engineer determines that a special case exists and a lesser capacity is selected. Minimum pile penetration shall be shown on the plans if the Geotechnical Engineer determines that underlying soil layers need to be driven through to avoid a situation that may lead to a pile failure.

IV-02.03.02 Pile Types

The type of pile to be used is generally specified in the "Foundation Report and Recommendation." On structure widening projects, the use of the same type of piling as was used in the existing structure is recommended.

IV-02.03.03 Design of Friction Piling

Friction piling can be designed in accordance with the instructions in Appendix IV-06 A.

IV-02.03.04 Test Piles

A test pile generally is not required, however they may be considered on larger bridges where significant savings may be realized.

IV-02.03.05 Pile Load Tests

Pile load tests are required when using allowable pile stress that is greater than the AASHTO design stress.

IV-02.03.06 Settlement/Downdrag

The "Foundation Report and Recommendation" often specifies that an embankment be placed to allow settlement to occur before starting construction of a substructure. In some cases, a surcharge embankment (additional height of fill above profile grade) may also be recommended as a means of accelerating the rate of consolidation.

Depending on the soil profile and length of the settlement waiting period, long term settlement of the soil may introduce downdrag in the piling. Downdrag is the downward load induced in the pile by the settling soil as it grips the pile due to negative skin friction. The amount of downdrag load to consider for design will be as specified in the "Foundation Report and Recommendation."

Live loads negates the effects of downdrag. Therefore, when determining load combinations, do not combine live load (or other transient loads) with downdrag. Consider a load combination that includes dead load plus live load and also a load combination that includes dead load plus downdrag, but do not consider live load and downdrag within the same load combination.

If piling is required to be driven in pre-bored holes through the embankment to natural ground, the predrilled hole shall have a minimum diameter of 24" for HP14 pile, 21" for HP12 pile, and 18" for HP10 pile. The holes shall be backfilled with bentonite or other approved material.

IV-02.03.07 Pile Spacing

To facilitate pile-driving operations, the minimum center-to-center pile spacing is 2'-6" with a 3'-0" minimum preferred. It may be necessary to increase the plan dimensions of a footing or pile cap when using battered piles. The standard embedment into a pier or abutment footing for a driven pile is 1'-0" and shall be dimensioned in the plans.

IV-02.03.08 Footings

Any footings or foundations with a thickness of 5'-0" or greater shall be treated as mass concrete. This may require the Contractor to modify the concrete mix and/or to instrument the concrete member and take action to ensure that the temperature differential between the inside and outside of the member is small enough to minimize the potential for cracking.

IV-02.03.09 Minimum Soil Cover

The minimum cover (soil, earth, or slope paving) on top of a footing is 12". For a pier footing which extends under a roadway, the minimum cover is 2'-0".

IV-02.03.10 Bottom of Footing

To minimize the potential for frost movements impacting the structure, the bottom of footings shall be placed at least 4'-6" below grade. When feasible, the bottom of footings (or seals if they are used) shall be placed below the estimated scour elevation.

IV-02.03.11 Scour

For bridges over a river or stream, spread footings are not allowed due to the potential for scour. When designing footings in areas of potential scour, assume no beneficial ground support for the piling from the flowline to the predicted total scour elevation during the extreme event load case.

IV-02.03.12 Pile Design for Scour

The estimated depth of scour and volume of soil removed along the length of pile shall be considered in pile design. The design procedures section of Chapter 3, HEC-18 shall be referred to for further guidance.

IV-02.03.13 Footing Plan Dimensions/Formwork

Footing plan dimensions shall be laid out in a manner that will allow support of the formwork used to construct the substructure elements above it. This is accomplished by extending the

footing at least 6" beyond the vertical face of the wall or stem.

IV-02.03.14 Footing Thickness/Shear

The footing thickness shall be sized such that shear reinforcement is not required.

IV-02.03.15 Spread Footings

Spread footings shall be used only when piling cannot be feasibly driven. AASHTO design procedures and safety factors shall be used for the design of spread footings.

IV-02.03.16 Footing Supported on Piling

The length of pile embedment into the footing shall be dimensioned in the plans. Battered piles shall be identified with a symbol that differs from vertically driven piles.

IV-02.03.17 Seal Design

A conventional cast-in-place seal is a mass of unreinforced concrete poured under water inside the sheet piling of a cofferdam. It is designed to withstand the hydrostatic pressure produced at the bottom of the seal when the water above is removed. Dewatering the cofferdam allows cutting of piles, placement of reinforcing steel, and pouring of the footing in a dry environment.

Design of the seal consists of determining a concrete thickness that will counterbalance the hydrostatic pressure with an adequate factor of safety. Lateral forces from stream flow pressure are resisted by penetration of the sheet piling below the streambed elevation and by the bracing inside the cofferdam. The cofferdam design is the responsibility of the Contractor.

IV-02.03.18 Design Philosophy

The loads used for the design of pile foundations shall be based on the following assumptions:

- All portions of piers are not cracked.
- Footing will not move laterally or rotate, but will remain infinitely rigid.

IV-02.04 Abutments

IV-02.04.01 Integral Abutments

Integral abutments are preferred when the geometrics allow for a jointless bridge deck. The following are design constraints for integral type abutments:

Maximum height	12'-0"
Minimum thickness	2'-0" (beams < 62" deep)
Minimum thickness	2'-6" (beams > 62" deep)
Minimum embedment in embankment	3'-0" (includes depth of riprap)
Maximum embedment in embankment	5'-0" (includes depth of riprap)
Minimum freeboard	2'-0"
Minimum low beam bottom to abutment bottom	5'-0"
Maximum low beam bottom to abutment bottom	5'-6"

For design example, see Appendix IV-06 E.

IV-02.04.02 Non-integral Abutments

The use of non-integral type abutments shall be as dictated by design and geometric requirements.

IV-02.04.03 Pile Spacing

The approximate maximum spacing for steel piles in an integral type abutment shall be 10'-0". The approximate maximum spacing for timber piles in an integral abutment shall be 7'-0". The minimum spacing for all piles shall be 2'-6" with a 3'-0" minimum preferred.

IV-02.04.04 Pile Embedment

Piles shall be embedded a minimum of 3'-0" into the wall of integral type abutments. Maximum embedment shall be governed by punching shear.

IV-02.04.05 Pile Orientation

Steel H-piling in integral type abutments shall be oriented so that the weak axis (web) is perpendicular to the face of the abutment.

IV-02.04.06 Straight Wings for Integral Abutments

For straight wings parallel to the abutment endwall, the slope of wings shall range from 4:1 to 2:1. Wing lengths shall range from a minimum of 6'-0" to a maximum of 15'-0". Wings for integral abutments shall have flat bottoms. Typically wings are tapered to one-half of the abutments thickness.

IV-02.04.07 U-Shape Wings for Integral Abutments

For U-shaped wings parallel to the bridge, wing lengths shall be a maximum of 20'-0" measured along the inside face. The thickness of the wings shall be 2'-0".

IV-02.04.08 Bridge Length Using Integral Abutments

Integral abutments shall be used on bridges less than 400' long with 0° skews. The maximum length of an integral abutment bridge with a skew is 400' multiplied by the cosine of the skew angle. Integral abutments shall not be used if the skew is over 30°. Parapet abutments shall be used in all other cases.

IV-02.04.09 Backfill, Drainage, and Waterproofing

Seepage trenches are generally used behind all abutments. See Plan Preparation Guide. If seepage trenches are not provided, seepage holes shall be provided through the walls. These holes shall be located close to the bottom of the backfill material. Vertical drainage and/or perforated pipe may be used as site conditions warrant. Two-ply fabric waterproofing shall be used at all construction joints.

IV-02.04.10 Joint between Approach Slab and Abutment

The approach slab shall be tied to the abutment or endwall for integral bridges. There shall be no tining of the concrete within 6" of the joint.

IV-02.04.11 Minimum Reinforcement

For integral abutments, a minimum reinforcement of No. 5 bars at 12" spacing (both directions), or that required by analysis, whichever is greater shall be used. For other abutment types, use that required by analysis of the AASHTO minimum, or the CALTRANS formula below, whichever is greater.

$$P_{min} = 1.7\left(\frac{h}{d}\right)^2 \frac{\sqrt{f'_c}}{f_y}$$

Where P = minimum reinforcement ratio
h = total depth of section
d = effective depth of section
f'_c = concrete compressive strength
f_y = yield strength of reinforcing steel

IV-02.04.12 Scour Design

Countermeasures such as riprap, guide banks (spur dikes), or other features shall be used to control potential scour. These items shall be discussed in the hydraulic report and/or TS&L comments.

IV-02.04.13 Waterproofing

Joint waterproofing shall be provided for construction joints below ground.

IV-02.05 Piers

IV-02.05.01 Type

For integral abutment bridges the piers shall be on a single row of pile. For stream crossings, a wall pier shall generally be used to minimize collection of debris. For separations, a wall with columns and a cap shall be used. The piers shall be the designer's preference considering economics, geometrics, and aesthetics.

For non-integral bridges, piers that use multiple rows of pile shall be used.

IV-02.05.02 Minimum Wall Thickness

Wall type piers shall have a minimum wall thickness of 24" for wall heights less than 25'. Wall type piers shall have a minimum wall thickness of 30" for wall heights between 25' and 32'. For wall type piers greater than 32' in height, the minimum wall thickness shall be 36".

IV-02.05.03 Caps

A pier cap width and length shall be chosen that is sufficient to support bearings and provide adequate edge distances. Cap widths shall be no less than 24". For fixed piers using prestressed beams with protruding steel, the beam end shall be a minimum of 6" from the edge of the pier cap and there shall be a minimum of 12" between beam ends. For I-beams over 48" in depth, the beam end shall be a minimum of 1'-0" from the edge of the pier cap. Integral piers shall have a shear key on top of the cap. Vertical steel dowel bars are not necessary.

Pier caps shall be sloped in a straight line for box beams. The cap shall be sloped or stepped between flat bearing areas for prestressed I-beams and steel girders.

IV-02.05.04 Columns

Columns can be either round or rectangular. They shall be proportioned to the rest of the structure to present an aesthetically pleasing bridge. The minimum column diameter or side of rectangular column is 2'-6". Consideration shall be given to standard forms when determining the size of circular columns.

IV-02.05.05 Height

The maximum height of piers without footings shall be 35'. The maximum height of piers with footings is the designer's decision.

IV-02.05.06 Cofferdams

The Contractor shall be responsible for the design of any cofferdams used on the project.

IV-02.05.07 Pile Embedment

For wall piers with a single row of piling, pile embedment shall be at least height of wall/3, but not less than 4'-0". For multi-column piers with a single row of piling, pile embedment shall

extend to within 2'-0" from top of pile cap, unless punching shear requires a greater clear distance. For piers with pile caps, pile embedment shall not be less than 4'-0".

IV-02.05.08 Scour

The elevation of the bottom of pier footing, pile cap, etc., shall be based on the calculated scour depth as shown in the hydraulic report and discussed in Hydraulic Engineering Circular 18.

IV-02.05.09 Scour Design

Countermeasures such as riprap, guide banks (spur dikes), or other features shall be used to control potential scour. These items shall be discussed in the hydraulic report or TS&L comments.

IV-02.05.10 Minimum Pier Embedment

The minimum pier embedment values below are based on frost heave considerations and includes the depth of riprap except for spread footings.

For spread footings on stream crossings	Min. = 5'-0"
For spread footings, other than stream crossings:	
West and central North Dakota	Min. = 5'-0"
Eastern North Dakota	Min. = 6'-0"
Northeastern North Dakota	Min. = 7'-0"
For pile footings on stream crossings	Min. = 5'-0"
For pile footings on other than stream crossings	Min. = 4'-6"
For piers without footings	Min. = 3'-0"

Note: Footing elevations based on calculated scour depths may govern.

IV-02.05.11 Crash Walls

Piers for railroad overhead structures shall have protective crash walls unless considered heavy construction. Design crash walls using AREMA specifications.

IV-02.05.12 Reinforcing Steel

For wall piers on a single row of piling, a minimum reinforcement of No. 5 bars at 12" spacing in both directions and in both faces shall be used.

IV-02.05.13 Pier Edge Treatment

Wall type piers shall have 1½" bevels on vertical edges. All other edges shall have the ¾" bevel required in the North Dakota Standard Specifications. All wall piers on stream crossing structures shall have ice nose protection on the upstream end.

IV-02.05.14 Pile Spacing

Minimum distance between the outside rows of piling shall be height of pier/7, but not less than 3'-0", as required by FHWA policy.

IV-02.06 Superstructures

IV-02.06.01 Design Parameters

Continuous composite (negative and positive) design shall be used on multi-span steel and prestressed concrete bridges. Continuous design shall be used on slab and T-beam bridges. Slabs on prestressed concrete beam bridges shall be considered composite for live load. Prestressed beams up to 48" in depth can be designed as composite and continuous using 90% of the simple moment due to live load. Deeper beams shall be designed as composite and simple, but detailed as continuous.

IV-02.06.02 Design Temperature Range

To determine the design temperature range for structures, AASHTO specifications for cold climate shall be used. For concrete structures, a mean temperature of 40°F shall be used.

IV-02.06.03 Maximum Beam/Girder Dimensions

The maximum length and height of beams/girders are controlled by transportation requirements. Railroad limits for hauling are approximately 165' for length and 15' for beam/girder depth. Truck limits are approximately 120' in length. Shipments with a width of 14' require a pilot car.

IV-02.06.04 Economical Span Arrangements

For prestressed concrete beams, use spans utilizing the same beam size when practical. For steel girders, use approximate span ratio of 1:1.3:1 and 1:1.25:1.25:1 when practical. Haunches are more costly to fabricate, so the economics shall be considered when designing haunched girders.

IV-02.06.05 Maximum Live Load Deflections

According to AASHTO policy, live load deflections shall be limited to 1/800 for structures with only vehicles and 1/1000 for structures which also provide for pedestrians.

IV-02.06.06 Diaphragms

Intermediate diaphragms shall not be used on prestressed box beams, except when the span is over a roadway or a railroad track and the beams are subject to impact from overheight loads. For prestressed I-beams, no intermediate diaphragms are required for spans of 45' or less, unless over a roadway or railroad track. One intermediate diaphragm at the midpoint in beams from 45' to 90' long shall be provided. Intermediate diaphragms at 1/3 points in spans over 90' long shall be provided.

Intermediate concrete diaphragms shall be placed at least 72 hours before the slab. Prestressed box beam diaphragms may be placed before the slab; however, they shall have a 72 hour cure before slab placement commences. Steel diaphragms are not allowed for prestressed beam superstructures. Pier diaphragms and endwalls shall be placed at the same time as the deck concrete for prestressed beam superstructures.

On steel I-beam bridges, with integral abutments, the abutment backwall shall be placed up to the top flange at least 72 hours before the slab. For steel girders 48" deep or more, steel X-frame or K-frame diaphragms shall be used. For girders less than 48" deep, steel bent plate diaphragms shall be used.

IV-02.06.07 Cantilever Design

The maximum deck cantilever length measured from the beam/girder centerline shall be 0.4 times the beam/girder spacing. When the cantilever length exceeds the I-beam depth, or 1.5 times the box beam depth, a precautionary note on possible beam twisting during construction shall be added to the plans.

IV-02.06.08 Beam/Girder Lines

The maximum beam/girder spacing shall be 10'-0". All structures shall have a minimum of four beam/girder lines. A minimum of five beam/girder lines are conducive to deck overlays and deck replacements constructed half at a time. The number of beam lines shall be kept constant throughout all spans in prestressed concrete bridges.

IV-02.06.09 Stay-in-Place Forms

For deck inspection reasons, stay-in-place forms shall be used only when conventional forming cannot be used.

IV-02.06.10 Class of Concrete and Cement Type

Superstructure concrete shall be Class AAE-3 with f'_c = 4,000 psi. Type I or IA cement shall be used.

IV-02.06.11 Concrete Treatment and Surface Finish

Plans shall include a bid item for penetrating water repellant on the driving surface of the bridge. Type D surface finish shall be used for barrier and curb surfaces that are visible to the motorist. Other surfaces shall be finished according to the North Dakota Standard Specifications.

IV-02.06.12 Concrete Cure

Superstructure concrete shall be cured using methods specified by the North Dakota Standard Specifications.

IV-02.06.13 Design for Future Wearing Surface

Structures shall be designed for a 15 psf future wearing surface.

IV-02.06.14 Cover over Reinforcement

Concrete cover over reinforcement shall be as specified by AASHTO, except that the cover over the top layer of deck reinforcement shall be 2½". When performing design calculations, it shall be assumed that ½" of the 2½" cover on the top reinforcement is worn off.

IV-02.06.15 Minimum Deck Thickness

Minimum thickness of the deck slab for any structure shall be 8".

IV-02.06.16 Reinforcing Steel Splices

Reinforcing steel splices shall be as specified by AASHTO and when possible splice locations shall be staggered.

IV-02.06.17 Reinforcement Bar Coupler

Connectors need to provide a capacity that is 125% of the nominal bar capacity.

IV-02.06.18 Epoxy Coated Reinforcement

All superstructure reinforcing bars except those that run transversely through end beams in endwalls and diaphragms shall be epoxy coated.

IV-02.06.19 Expansion Joints

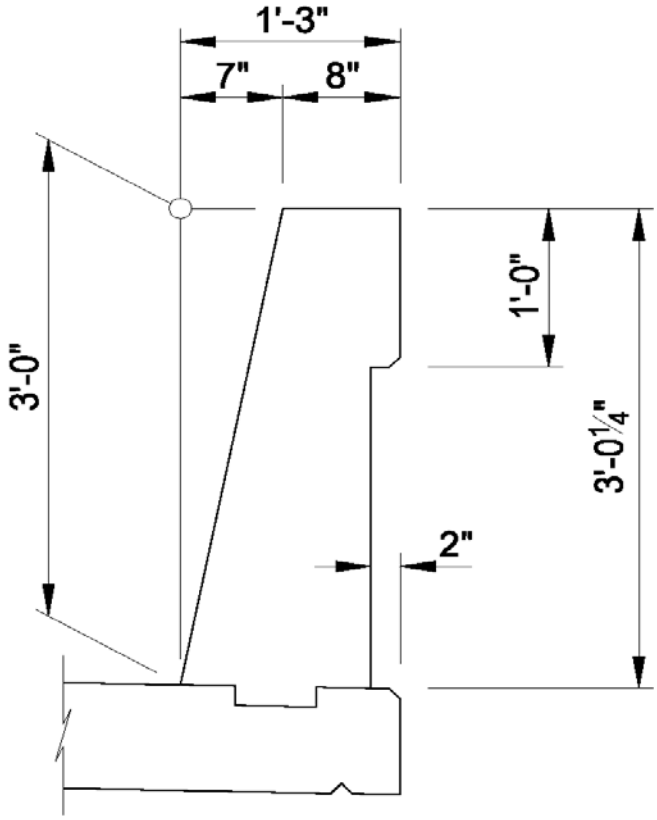
When expansion joints are necessary, strip seals for joints with up to 5" of movement shall be used. For joints with greater than 5" of movement, finger type joints with troughs shall be used. For expansion joints in barrier walls with more than 5" of movement, sliding steel plates shall be used.

IV-02.06.20 Minimum Deck Concrete Placement

North Dakota Standard Specifications require a minimum placement rate of 25 cubic yards per hour. If a greater rate of placement is needed, it shall be noted in the plans. The rate of placement may be based upon a 10-hour work day. Full width deck pours shall be utilized when there is no stage construction (decks 100 feet wide have been placed successfully).

IV-02.06.21 Bridge Railings

For new bridges or deck replacements, use the 36" single slope barrier (see sketch below). The two-tube retrofit may be used on existing bridges where applicable. Only approved transitions and guardrail attachments shall be used. On two-tube retrofit plans the following note shall be added, “The anchor bolts shall be embedded into concrete with a chemical adhesive system that can develop a tensile strength of at least 18,250 lbs.” Mechanical anchorages will not be allowed.



BARRIER RAILING

IV-02.06.22 Canopies for Construction

Dimensions and specification requirements are not in the North Dakota Standard Specifications and shall be provided in the plans.

IV-02.06.23 Deck Overlays

The design of deck overlays shall be as specified in the North Dakota Standard Specifications for low slump concrete. Typically, milling machines (mechanical equipment) are used for removal. Quantities for bridge deck overlay:

Class 1 (SY) = 100% of deck area
Class 2 (SY) = max of 20% of deck area or delamination survey
Class 3 (SY) = 5% of deck area or 25% of Class 2
Class 2-A (LF) = Class 2 (SY) x 1.8

Consideration shall be given to using the Class 4 overlay bid item.

IV-02.06.24 Longitudinal Deck Joints

When widening a deck, the longitudinal joint shall be over a beam/girder where possible. In this case the joint shall be sawed and sealed with silicone sealant. If the joint is not located over a beam/girder, a keyway shall be provided in the joint to transfer the shear.

IV-02.06.25 Joint Spacing in Barriers

Joints in barriers shall not be used unless required by an expansion joint in the deck. Use “V” grooves equally spaced (approximately every 10') between adjacent substructure units or as shown in plans.

IV-02.06.26 Deck Drainage

Deck drainage shall be designed according to FHWA Hydraulic Circular 21, “Design of Bridge Deck Drainage”. The design frequency for all bridges shall be 10-year. The design spread shall be ½ of the driving lane. Typically, deck drains are 6" diameter PVC pipe through the deck. If the deck geometry is such that the water running through the deck drain would hit the beam/girder, the drain shall be galvanized steel tubing (4" x 6" minimum) extending to the bottom of the beam/girder and braced as required.

IV-02.06.27 Roadway Width for Estimating

The out-to-out dimension of the bridge deck, including sidewalks, shall be used for estimating purposes.

IV-02.06.28 Deck Replacement

Replacements for decks that must be totally removed because of condition shall be designed for a minimum of HS20 live load.

IV-02.06.29 Concrete Deck Removal

Deck concrete shall not be allowed to fall into streams nor be used as riprap.

IV-02.06.30 Low Modulus Silicone Sealant

Silicone sealant (Type 5) shall be used to fill joints as designated in the plans. The joint between inside face of barrier and deck shall be sealed. The joint between the bridge barrier and approach slab barrier shall also be sealed.

IV-02.06.31 Cold Joints

Cold joints in bridge decks shall normally be saw cut and sealed with silicone sealant (Type 5).

IV-02.06.32 Water Repellent and Silicone Sealant (Type 5)

Whenever a silicone type joint sealer is specified for construction joints in decks which are to be treated with a penetrating water repellent, the following construction sequence shall be specified by a plan note:

1. The joint(s) shall be grooved with a saw.

2. The water repellent shall be applied to the entire deck, including the grooved joint(s).

3. The water repellent shall be allowed to cure in accordance with the manufacturer's recommendations.

4. The joint sealer shall be placed in the grooved joint(s) and allowed to cure.

IV-02.06.33 Deck Reinforcing Steel Spacing

The minimum transverse deck reinforcing steel spacing shall be 6". If smaller spacing of reinforcing steel is required, increase bar size up to No. 6 bars. If still more reinforcing steel is required, increase deck thickness in ½" increments.

IV-02.06.34 Barrier Skewed Ends

For bridge skews of 15° and greater, square off the ends of the barrier at the bridge ends with the barrier not extending past the bridge/approach slab joint. Provide a "Barrier End Detail" on the Slab Layout sheet.

IV-02.07 Structural Steel

IV-02.07.01 General Design Philosophy

Structural steel superstructures are shallower and lighter than concrete superstructures. In addition to long span and specialty structures, steel superstructures shall be considered where foundations are expensive or where a change in superstructure height has significant cost implications on the approaches.

Girders shall be designed to be composite with the concrete deck throughout the entire girder length. Shear connectors, in the form of shear studs shall be provided in both positive and negative moment areas and over field splices.

During design, it may be assumed that the dead load of the steel girder is 15% larger than that computed using only the flanges and web. This is a reasonable estimate for the weight of stiffeners, diaphragms or cross frames and connections.

For large structures a web depth study shall be performed to arrive at the optimal girder depth.

IV-02.07.02 Materials

Steel bridges are fabricated and constructed with steel elements that are produced at two different types of steel mills; shape mills and plate mills. In addition to different products, the grades of steel available from each type of mill differ slightly.

Plate mills produce flat sections that are used to fabricate plate girders, connections, gusset plates, etc. Plate steel is also produced in a number of different material specifications. Larger plate mills have a width limitation of 150". The maximum available plate length varies by mill and varies by cross-sectional dimensions of the plate.

Typically, only fillet welds and full penetration welds are permitted. Weld designs shall be based on E70 filler material.

With the exception of pile splices and shear connectors, field welding is not used or permitted.

Additional references are the ANSI/AASHTO/AWS “Bridge Welding Code - D1.5”, and “Welding of Steel Bridges”, Vol. I, Chapter 15, Highway Structures Design Handbook.

Steel plates used in the fabrication of bearings shall be Grade 36, 50, or 50W.

IV-02.07.03 General Dimensions and Details

Designers shall provide simple details that are easily fabricated and do not sacrifice the integrity of the bridge. Details that trap water or produce an environment that is conducive to corrosion shall be avoided. In addition, details with inadequate clearances are difficult to fabricate and

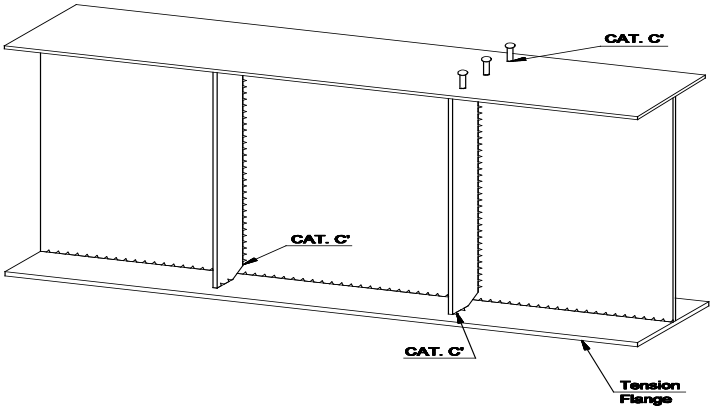
erect.

Structural steel plans and details shall clearly describe the material to be used for each structural steel component. Even for projects where structural steel is paid for on a lump sum basis, informational quantities shall be provided in the plan set to quantify the amounts of different steels incorporated into the project.

Welding is used in many locations during the fabrication of plate girders. It is used to connect:

- web plates to flange plates
- stiffeners and connection plates to web plates
- stiffeners and connection plates to compression flanges

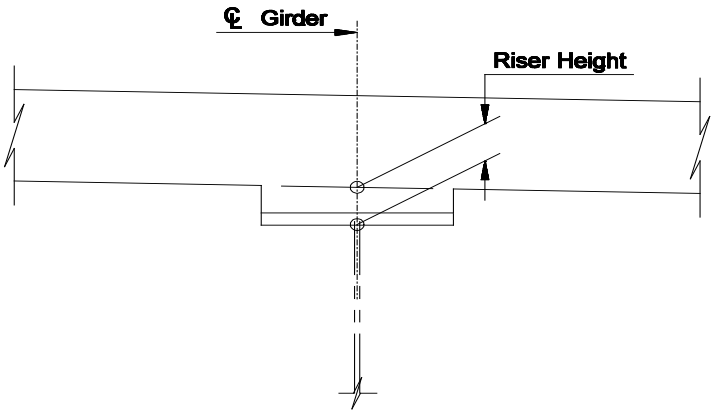
The following figure identifies the locations of these welds and the appropriate fatigue category to be used for checking live load stress ranges.



Field splices shall be located at or near points of dead load contraflexure.

The LRFD Specifications do not explicitly give a maximum diaphragm spacing as was previously given in the Standard Specifications. The spacing of diaphragms used for bracing is used to determine allowable compressive stresses. Maximum diaphragm spacing is 25'-0". If the span length is not divisible by 25'-0", use 25'-0" spacing in the positive bending area and the smaller area near the pier.
The segment length to top flange width ratio shall be limited to a ratio of 85 for stability during shipping and erection.

Risers are used with steel superstructures to provide a construction tolerance for the profile of the deck. The riser shall have vertical edges that are flush with the edges of the top flange. The riser is defined as the distance between the bottom of the deck and the top of the web. The minimum height or thickness for the concrete portion of the riser is 1 inch. The minimum shall be provided at the edge of the flange taking into account the cross slope of the deck. At field splices check that the top plates do not penetrate the bottom of the deck by more than ½".

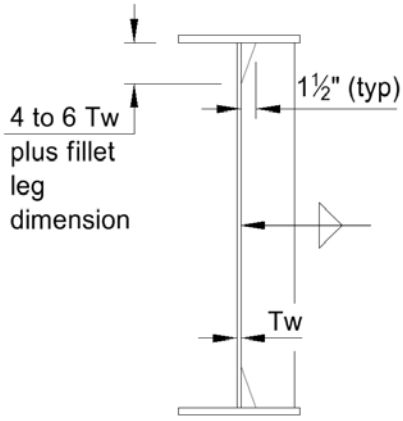


Structural steel quantities are computed by finding the weight of steel beams or girders, diaphragms, cross frames, and all other plates (e.g., sole and gusset plates).

When sizing stiffeners and connection plates, a limited number of thicknesses shall be used. Connection plates must be a minimum of ½" x 7½" to permit two lines of bolts.

IV-02.07.04 Stiffeners, Transverse and Bearing

Single or paired transverse stiffeners may be used. For paired stiffeners, the same details as shown for a single stiffener shall be used. When longitudinal stiffeners are required, all transverse stiffeners shall be placed on one side of the web, and the longitudinal stiffener placed on the opposite side.



Fillet weld (both sides) to both flanges when used as a cross-frame or diaphragm connection plate. Otherwise, a tight fit to tension flanges and fillet weld to compression flanges shall be used. As an option, when the stiffener is not used as a connector plate, the stiffener may be cut short from the tension flange by a distance equal to the vertical leg of the cope. For bearing stiffeners at piers and abutments, fillet weld both sides of the stiffener to the top flange, mill to bear and fillet weld to the bottom flange. The same cope dimensions as used for transverse stiffeners shall be used.

IV-02.07.05 Stain Prevention

A ¾" bead of silicone caulking shall be placed around the steel girders, excluding the top flange, prior to the girders being set on the substructures. The caulking shall be located 6 feet from the abutments and 6 feet from the piers on the higher elevation side.

The girders shall be painted 10 feet from each open joint and 5 feet from integral abutments.

IV-02.07.06 Rolled Steel Beams

Typically, no cover plates shall be used on rolled beams for new structures.

IV-02.07.07 Shear Connectors

$^{7}/_{8}$" diameter stud shear connectors that extend above the bottom mat of deck reinforcement shall be used in positive and negative moment areas spaced according to AASHTO specifications. Stud shear connectors shall be installed in the field using automatically timed stud welding equipment.

IV-02.07.08 Surface Treatment for Bolted Joints

Bolted joints shall be designed for a Class A surface for slip resistance (Slip Coefficient 0.33). Faying surfaces shall be primed the same as the rest of the beam.

IV-02.07.09 Grades of Structural Steel

Structural steel grades shall be as specified by AASHTO specification M 270, Grade 36, 50, or 50W.

IV-02.07.10 Steel Erection

For shop assembly (reference AASHTO Construction Specifications), progressive girder assembly shall be specified in the plans except for bridges with complex geometry. For complex structures, full girder or complete structure shop assembly shall be specified. Requirements shall be determined at the Preliminary Engineering meeting. Field erection shall be as specified in the North Dakota Standard Specifications.

IV-02.07.11 Camber

For most steel bridges camber will be fabricated into the girder to offset the deflections due to applied dead loads. The cambered member is fabricated with a profile opposite of that caused by dead load deflection.

For rolled beams introducing camber can be a relatively expensive operation. The beam shall be placed “natural camber up”.

Plate girders shall always be cambered. This is accomplished without mechanical means or heat straightening techniques. Vertical cambers are introduced by cutting the web plates with the

desired profile. Horizontal curvature is introduced by cutting flange plates with the proper horizontal shape. During fabrication, the web and flanges are attached to each other to produce a member with the proper geometric characteristics.

Girders shall be cambered for anticipated dead load deflections and vertical curve.

Camber information shall be included in the plans and presented in fractions of an inch ($^{1}/_{16}$" precision). A table and schematic detail shall be used to convey the information. Within the schematic detail, the horizontal reference line, chord lines connecting field piece ends, and the camber curve shall be labeled. Tabularized information at field splices, support points, and at intermediate points along the length of field pieces shall be provided. Each field piece shall be defined by at least ten points (20 points for longer pieces) that are uniformly spaced at intervals between 5' and 20'.

IV-02.07.12 Welded Steel Girders

Plate sizes shall be as controlled by product availability in the AISC Manual of Steel Construction and availability from suppliers. The use of hybrid girders is not recommended. Optional field splices can be provided in the plans if deemed necessary or practical. Tension members shall meet the longitudinal Charpy V-notch Zone 2 test. Top and bottom flanges may be different sizes at the piers.

Plate thicknesses in $^{1}/_{16}$" increments for thicknesses up to 1" shall be selected. For thicknesses between 1" and 3", $^{1}/_{8}$" increments shall be used. For thicknesses between 3" and 4", ¼" increments shall be used.

In general, additional web thickness increases shear capacity. An increase in web height or flange area increases moment capacity and reduces live load deflections.

In general, these guidelines shall be followed in plate size selection for plate girders:

Flanges
No flange plate shall be less than ¾" thick, nor less than 12" in width.

The width of the flange shall not change at a welded butt splice. The change in flange area at butt weld splices shall not exceed 100%. In general it is economical to provide a butt splice if 1000 lbs or more of steel can be saved.

Top and bottom flanges shall be kept a constant width within each field piece. If changing the top flange width at a field splice, the flange width shall not be tapered.

Web
For web plates the minimum thickness is ½". The ½" web reduces the potential for warping during fabrication.

For continuous structures the web shall be sized to be $^{1}/_{16}$" thinner than a web which requires no stiffeners for shear.

Longitudinal stiffeners shall only be considered for girders over 84" deep. The stiffeners shall be terminated at a low stress point with a fatigue resistant detail. Generally, longitudinal stiffeners shall be continuous through transverse and bearing stiffeners.

IV-02.07.13 Curved Girders

AASHTO Design Specifications do not cover the design of curved bridges. Curved bridges shall be designed in accordance with the 2003 AASHTO "Guide Specification for Horizontally Curved Steel Bridges."

IV-02.07.14 Bolted Connections and Splices

Most structural connections or splices shall be detailed with $^{7}/_{8}$" diameter M 164 bolts. The typical bolt pattern is a 3" grid with 1½" edge distances.

For applications where strength is not the primary design consideration, ASTM A 307 bolts may be used.

The change in flange area at bolted splices shall not exceed 100%. The splice plates shall be of the same steel as the elements being connected. The minimum thickness of splice plates is $^{5}/_{16}$".

Bolted field splices shall be designed as slip-critical connections. Assume that the threads are not included in the shear plane and that a Class A surface coating or condition is available for slip resistance (Slip Coefficient 0.33). Faying surfaces shall be primed the same as the rest of the girder.

IV-02.07.15 High Performance Steel Girders

Girders using High Performance Steel (HPS, F_y = 70 ksi) can be an economical alternate to girders using 50 ksi steel.

IV-02.08 Prestressed Concrete

IV-02.08.01 Shapes

Modified “I” sections or box sections are normally used. Deeper sections are more economical and “I” sections are more economical than box sections. Depth of member generally depends upon clearance requirements. End blocks shall be used for all beams.

IV-02.08.02 Stress Steel

Typically 0.6" diameter 270 ksi low relaxation strands as specified in AASHTO M 203 are used. ½" strands may be used.

IV-02.08.03 Design Compressive Stresses for Concrete

A minimum of 4,000 psi at stress transfer shall be used. The compressive strength at 28 days shall be a minimum of 5,000 psi. The design maximum strength shall be approximately 7,000 psi.

IV-02.08.04 Tension

Allowable tension shall be as specified by AASHTO specifications.

IV-02.08.05 Concrete Cover over Reinforcement

A minimum of 1" of concrete cover shall be used for stirrups and tie bars. A minimum of 1½" of concrete cover shall be used for prestressing steel and main reinforcement.

IV-02.08.06 Reinforcing Steel

Reinforcing steel shall be designed according to AASHTO specifications. Grade 60 reinforcement shall be used. Grade 40 rebar may be used for the reinforcing that requires bending after beam fabrication.

IV-02.08.07 Continuous Girders

Prestressed beams up to 48" in depth shall be designed as continuous, composite, using 90% of the simple moment due to live load. Beams greater in depth shall be designed simple, composite, but shall be detailed as continuous.

IV-02.08.08 Post-Tensioned Concrete

Post-tensioned concrete is not recommended.

IV-02.08.09 Debonded Strands

Partially and fully debonded strands are allowed.

IV-02.08.10 Risers

A positive riser shall be designed on I-beams and spread box beams. A ½" minimum riser at centerline of beam shall be provided.

IV-02.08.11 Prestressing Steel

The modulus of elasticity for prestressing steels (E_p) is 29,000 ksi (Forterra mill certifications) for strands and reinforcing bars.

IV-02.08.12 Prestressed Concrete Unit Weight

The concrete modulus of elasticity computations and dead load calculations shall be based on a unit weight of 0.155 kcf.

IV-02.08.13 Load Distribution to Substructures

For prestressed beam bridges, the deck dead load shall be distributed to the abutments assuming the spans are simple. The deck dead load shall be distributed to the piers assuming the spans are continuous, even though most construction specifications are to pour the pier diaphragms with the deck concrete.

IV-02.09 Cast-In-Place and Precast Reinforced Concrete Box Culverts

IV-02.09.01 Size Capabilities

The RCB computer program parameters range from a single 4' x 4' RCB to a quadruple 16' x 16' RCB. Sizes in between are available in 1' increments.

IV-02.09.02 Size and Length

Size of the RCB shall be determined by hydraulic requirements. The length of an RCB shall be determined by cover/fill and clear zone requirements. Extend the RCB so that the end (outside edge of parapet) is where the embankment cross section intersects a point 1'-3" above the bottom of the roof of the box culvert.

IV-02.09.03 Skewed Culverts

For a skew of less than 15°, a square end shall be used. Skewed culverts shall be designed for skew angles in 5° increments. The RCB plotting program plots right and back. If the skew is opposite, the details shall be modified.

IV-02.09.04 Class of Concrete and Type of Cement

RCB concrete shall be Class AE-3. Cement shall be Type I or IA, unless sulfates are present and other types are recommended by the Materials Division or determined at the TS&L inspection.

IV-02.09.05 Reinforcement

The RCB design program uses Grade 60 reinforcement. It also determines minimum reinforcement and reinforcing steel splices.

The reinforcement used in precast concrete box culverts can be either conventional bar reinforcement or welded wire reinforcement. Welded wire reinforcement has a yield strength slightly larger than conventional bar reinforcement (65 ksi versus 60 ksi).

IV-02.09.06 Concrete Cover over Reinforcement

Concrete cover over reinforcement shall be as specified by AASHTO specifications.

IV-02.09.07 Concrete Finish

If used as an underpass, surface finish “D” shall be specified for exposed portions of the RCB. Graffiti protection may be used when requested by the district or political subdivision.

IV-02.09.08 Loadings

Box culverts shall normally be designed for HL-93 live loading. For earth loading, 5'-0" fill height increments shall be used. The entire box shall be designed for the same amount of fill.

IV-02.09.09 Culvert Extensions

If the amount of fill increases by more than 2.5', existing RCBs shall be checked for allowable live loads before extending. Decisions to extend shall be made by the Design Engineer and the Bridge Engineer. Extensions shall be designed by the RCB design program. Remove 1'-6" of the existing box culvert roof and walls in order to tie the extension to the existing reinforcing steel. The extension may be longer than required in order to avoid removing the wingwall footing.

IV-02.09.10 Hydraulic Data

The hydraulic data for the proposed culvert shall be placed on the RCB structural drawings, as required by FHWA.

Hydraulic Data items:

Drainage Area
Stream Gradient
Design Frequency
Design Discharge
Design Headwater Stage
Design Tailwater Stage
Velocity Through Box Culvert
100-Year Frequency Discharge
100-Year Frequency Headwater
Overtopping Stage
Overtopping Discharge

IV-02.09.11 Drawing Number/Structure Number

The drawing number/structure number (highway and mile/reference point) shall be added to the detail drawings.

IV-02.09.12 Precast RCB

Unless noted in the PCR and/or TS&L report, alternates shall be prepared for box culverts. Cast-in-place box culverts will be used for skews greater than 15°.

Standard designs for precast concrete box culverts are available with openings varying from 6' to 14' wide by 4' by 14' high. The designs utilize concrete strengths between 5 and 6 ksi and are suitable for fill heights ranging from 2'-0" on up.

The design moments shall be shown on the layout sheet.

The wings shall be the same length and height as the cast-in-place alternate. The apron shall be the same width as the cast-in-place alternate.

IV-02.09.13 Precast RCB-Multibarrel

Multibarrel structures may be cast in one section, or may be single barrel or multibarrel units placed side by side. The space between units shall be a maximum of 6" and a minimum of 3". This space shall be backfilled with controlled density backfill. Exposed areas of controlled density backfill shall be capped with 2" of full strength concrete to provide weather protection for the low density fill. Barrel sections shall be tied together post-tensioning a continuous cable to 20 kips.

IV-02.09.14 Controlled Density Backfill

The controlled density backfill shall be a blend of cement, water, pozzolanic materials, and fillers. The material shall be fluid at the time of placement in order to flow into and fill voids in the backfill area. The material shall be capable of supporting normal loads after 6 hours and shall have a compressive strength in the range of 75 to 125 psi at 28 days. The Contractor shall provide mix designs and compression strength test results of the material to the Engineer for approval 5 days prior to placement.

IV-02.09.15 Pedestrian & Shared-Use

Use an opening of 12' wide by 8' high for pedestrian and shared-used box culverts.

IV-02.10 Miscellaneous

IV-02.10.01 Grade of Reinforcement

Grade 60 reinforcement shall be used.

IV-02.10.02 Rebar Lengths

A maximum length of 60 feet for bars shall be used.

IV-02.10.03 Reinforcing Steel Splices

Design splices of reinforcement according to AASHTO. Generally, lap splices are used, but mechanical splices can be used where practical and economical.

IV-02.10.04 Concrete Cover over Reinforcement

Concrete cover over reinforcing steel shall be according to AASHTO. An exception to this is that concrete cover over reinforcement shall be 4" where the concrete is cast against and is permanently exposed to earth.

IV-02.10.05 Concrete Unit Weight

The concrete modulus of elasticity computations and dead load calculations shall be based on a unit weight of 0.150 kcf.

IV-02.10.06 Substructure Class of Concrete and Type of Cement

Class AE-3 concrete shall be used in substructures. Unless specified in the plans, the type of cement used in substructure concrete shall be as stated in the North Dakota Standard Specifications for Road and Bridge Construction, which is Type I or IA. The need for sulfate resistant cement shall be determined in the hydraulic report or at the TS&L inspection, and if required shall be noted in the plans.

IV-02.10.07 Concrete Admixtures

Proposed concrete admixtures shall be approved by the Materials and Research Division. The use of admixtures shall be in accordance with the North Dakota Standard Specifications for Road and Bridge Construction.

IV-02.10.08 Substructure Concrete Surface Finish

Substructure surfaces shall receive the rubbed finish (surface finish “C”) unless otherwise noted in the plans. For areas that are highly visible to the public, consideration shall be given to the use of surface finish “D”. Graffiti protection material shall also be considered in these locations.

IV-02.10.09 Welding

Field welding, other than welding for piling and shear studs, is rarely permitted. Welder and welding procedure shall be certified through the Materials and Research Division. Large fabricated bearings, or other similar assemblies, require stress relief (use plan note). All structural welding shall comply with AASHTO/AWS specifications.

IV-02.10.10 Approach Slabs

The standard length of approach slab is 20' and shall be designed for an HL-93 live load. A span length of 15' shall be assumed for design purposes. Standard reinforcing for approach slabs 14" thick is #7's @ 6" for the bottom and top mats. No. 5's at 15 equal spaces shall be used for transverse distribution reinforcing top and bottom.

Approach slabs shall have a barrier or curb to direct water to the end of the approach slab. The approach slab shall be tied to the bridge with reinforcing steel that extends 3'-0" into the bridge deck and approach slab, placed 7" below the top of the approach slab. For skewed bridges with an asphalt approach roadway, the approach slabs shall be skewed 0° at the roadway/slab joint. When concrete pavement abuts the approach slab, Design Division shall be checked with for dowel or tie bar placement. Excavation and backfill quantities shall be computed and shown as bid items, or may be included in approach slab bid item. The need for pile supported approach slabs will be determined at the Preliminary Engineering Meeting.

IV-02.10.11 Drainage at Bridge Ends

Water shall be allowed to flow off the approach slab. Fiberglass roving, or similar anti-erosion material, shall be provided by the Design Division on the roadway foreslope. Whenever possible, existing drains shall be removed and the pipe plugged. This means that a 20'-0" approach slab shall be satisfactory for all installations except where existing drains cannot be eliminated by the Design Division. Approach slabs without the crash tested transition shall be modified to provide a similar drainage pattern.

IV-02.10.12 Slope Protection

The need for slope protection shall be established at the Preliminary Engineering meeting. The type of slope protection to be used shall also be determined at this meeting. NDDOT policy is to use concrete slope protection in urban areas and aggregate slope protection in rural areas. When used on skewed structures, the edges of the slope protection shall be parallel to the roadway.

IV-02.10.13 Utilities

Attaching utilities to bridge structures shall be avoided whenever possible. However, when it is not possible to locate the utilities elsewhere, see Appendix IV-06 B for guidance.

IV-02.10.14 Retaining Walls

Retaining walls shall be designed according to AASHTO specifications. Typically the Materials and Research Division designs reinforced earth retaining walls and similar structures. Cast-in-

place walls are designed by the Bridge Division. Patented walls may be considered for special installations.

IV-02.10.15 Super Span, Con-Span Culverts, Etc.

This type of structure may be considered on a limited basis.

IV-02.10.16 SPP and SPPA Cutoff Walls

Headwalls and cutoff walls shall normally be used on all structural plate steel pipe.

IV-02.10.17 Preformed Expansion Joint Filler

When preformed expansion joint filler is used at beam bearing seats, provide ½" joint filler under the entire beam area that would be in contact with the concrete bearing seat.

IV-03.01 Foundations

IV-03.01.01 Allowable Pile Loadings

The maximum allowable pile loads for various types of pile are as follows:

Steel H-pile:

Commonly used sizes:	HP10 x 42 - - - - 78 tons	(56 tons, 36 ksi)
(F_y = 50 ksi)	HP12 x 53 - - - - 97 tons	(70 tons, 36 ksi)
	HP14 x 73 - - - - 134 tons	(97 tons, 36 ksi)
	HP14 x 102 - - - 188 tons	(135 tons, 36 ksi)

These allowables are based on $0.25F_y$ (12,500 psi) and can be increased to $0.33F_y$ if the additional requirements of AASHTO are met.

Timber Pile:

Butt Diameter	Capacity
10"	20 tons
12"	24 tons
14"	28 tons
16"	32 tons

These capacities are based on the table on page 45 of the Eleventh Edition (1973) of the AASHTO's "Standard Specifications for Highway Bridges."

Steel Encased Concrete (SEC) Pile:

AASHTO loads for SEC piling are based on $0.25F_y A_s + 0.40f'_c A_c$ where F_y = 35,000 psi and f'_c = 3000 psi. The area of steel (A_s) is computed assuming $^1/_{16}$" of steel is corroded off of the outside diameter of the pipe. When driving the steel shell of SEC pile, care shall be taken not to exceed the allowable driving stress on the steel.

The allowable driving stresses on pile shall be according to the AASHTO specifications.

Sometimes the soil conditions cannot support the maximum pile loads. The designer shall consult with the Geotechnical Engineer to determine the allowable pile loads.

IV-03.01.02 Pile Driving Formula

Include the following pile driving formula and note in the bridge plan sheets.

For double acting or single acting diesel hammers, calculate the safe bearing value of piles by the following formula:

$$P = \frac{3.5E}{S + 0.2} \times \frac{W + 0.2M}{W + M}$$

Where:

P = Safe bearing value, in pounds.
W = Weight of striking parts (ram), in pounds.
M = Weight of parts being driven, in pounds. Includes pile weight, anvil (if any), driving cap, etc.
E = Energy per blow, in foot-pounds.
S = Average penetration of pile in inches per blow for last ten blows.

For single acting hammers, calculate E by multiplying observed stroke (ft) and W (lbs).

IV-03.02 Abutments

IV-03.02.01 Earth Pressure on Abutment Walls

Use 1,000 pounds per square foot on integral abutment walls, 650 psf on integral wing walls. For integral abutments, use $B_E = 1.0$ for walls and wings. Use 40 psf equivalent fluid pressure on other walls, or a value calculated by Rankine or Coulomb theory of lateral earth pressure.

IV-03.02.02 Scour Design

AASHTO 4.4.5.2 may apply.

IV-03.02.03 Design Loads

Abutments shall be designed for AASHTO HS25 live load. Also, use military loading if it controls on mainline Interstate bridges. If the foundation report indicates significant settlement, negative skin friction shall be addressed.

IV-03.03 Piers

IV-03.03.01 Pile Embedment

Design according to AASHTO 4.5.15.

IV-03.03.02 Ice Pressure [3.18.2]

Stream crossing piers shall be designed for an ice pressure of 200 psi, assuming 4'-0" ice thickness on the Missouri River and 3'-0" ice thickness on other streams.

IV-03.03.03 Design Loads

Piers shall be designed for AASHTO HS25 live load. Also, use military load if it controls on mainline Interstate bridges.

IV-04.01 Introduction

IV-04.01.01 Material Contained in Section IV-04

The LRFD Design Section contains material arranged around the following subsection headings. To simplify locating material, subsection numbers correspond to those used in the LRFD specifications:

1. Introduction
2. Reserved (General Design and Location Features)
3. Loads and Load Factors
4. Structural Analysis and Evaluation
5. Concrete Structures
6. Steel Structures
7. Reserved (Aluminum Structures)
8. Reserved (Wood Structures)
9. Decks and Deck Systems
10. Foundations
11. Abutments, Piers, and Walls
12. Buried Structures
13. Railings
14. Joints and Bearings

IV-04.01.02 Limit States to Consider in Design

Bridge designs shall typically consider Strength, Service, Extreme Event, and Fatigue and Fracture limit states. The limit state checks will vary with the component under consideration. Not all elements will require consideration of all limit states. For example, the fatigue limit state need not be considered for fully prestressed pretensioned elements.

IV-04.02 General Design and Location Features

This subsection intentionally left blank for future use.

IV-04.03 Loads and Load Factors

IV-04.03.01 Load Factors and Combinations [3.4.1]

The standard load combinations for LRFD design are presented in LRFD Table 3.4.1-1. NDDOT regularly uses all load combinations except Strength II and Extreme Event I. These are used only for specialized structures or situations.

Several of the loads have variable load factors (e.g., γ_p, γ_{TG}, γ_{SE}). The load factors for permanent loads (γ_p) typically have two values, a maximum value and a minimum value. When analyzing a structure it will often be necessary to use both values. The objective is to envelope the maximum load effects on various elements for design.

A box culvert structure illustrates the use of both values. When determining the moment in the top slab of the culvert the maximum load factor is used with vertical earth loads, while the minimum load factor is used on the lateral or horizontal earth loads. The situation reverses when determining the moments in the wall of the culvert. A minimum load factor is used on the vertical earth loads and a maximum value is used on the horizontal earth loads.

When assembling load combinations, no more than one load factor for any load component shall be used. For example, when checking uplift, a load factor of 0.90 or 1.25 shall be used for the dead load on all spans. Designers shall not try to use 0.9 on the span adjacent to the uplift point and 1.25 on the next span.

Designers must ensure the structure has been checked for adequacy in carrying all appropriate load combinations at any possible construction stage. For example, a high abutment shall be checked for any permissible construction case in addition to the final condition. The abutment may be completely constructed prior to placement of the beams (a case which maximizes the horizontal earth pressure load with a minimum of vertical load) or the abutment could be constructed such that the superstructure is completed prior to backfilling. This latter case would maximize vertical load without horizontal earth pressure load. Designers shall investigate both cases.

Load Combinations
The load factors and the combination of different load components presented in LRFD Table 3.4.1-1 have been calibrated to produce structures with more uniform reliability than that offered with Standard Specification designs. In general, NDDOT recognizes all of the load combinations presented in the LRFD Specifications. The Strength II and Extreme Event I load combinations will rarely control. Also, Service IV load combination will rarely apply.

Strength I: Basic load combination used to determine the flexural and shear demands without wind.
Strength II: Basic load combination used to determine the flexural and shear demands of a structure subject to a permit vehicle or a special design vehicle specified by the owner. (NDDOT does not use a special vehicle for design.)

Strength III: Load combination used to determine flexural and shear demands that include wind. (Winds over 55 mph)

Strength IV: Load combination relating to very high dead load to live load force effect ratios.

Strength V: Load combination corresponding to normal vehicular use of the bridge concurrent with a wind of 55 mph.

Extreme Event I: Load combination including earthquake effects. Earthquake analysis is typically not performed.

Extreme Event II: Load combination corresponding to ice loads, collision loads, and certain hydraulic events with a reduced vehicular live load. This combination is used for barrier and deck overhang designs.

Service I: Load combination used for the design of many elements. It is used for service load stress checks (prestressed concrete), deflection checks, crack control checks in reinforced concrete, etc.

Service II: Load combination used to check yielding and connections in steel structures.

Service III: Load combination used to check nominal tension in prestressed concrete structures.

Service IV: Load combination relating only to tension in prestressed concrete columns with the objective of crack control.

Fatigue: Load combination used for the design of structures subject to repetitive live load. This pertains primarily to steel structures and steel reinforcement in concrete structures.

IV-04.03.02 Load Modifiers [1.3.3, 1.3.4, 1.3.5]

For most structures, each of the load modifiers will be 1.00. For a limited number of bridges, load modifiers with values different from 1.00 need to be used. Table 3.2.1 summarizes NDDOT's policy for load modifiers.

Table 3.2.1 – Standard NDDOT Load Modifiers

Modifier	Value	Condition
Ductility (ηD)	1.00	Steel structures, timber bridges, ductile concrete structures
	1.05	Non ductile concrete structures
Redundancy (ηR)	1.00	Redundant
	1.05	Non Redundant
* Importance (ηI)	0.90	Temporary Bridges
	0.95	ADT < 500
	1.00	500 ≤ ADT ≤ 40,000
	1.05	Major river crossing or ADT > 40,000 or Mainline interstate bridge

* Use Importance Factor for design of the superstructure only.

Note that load modifiers apply only to the Strength limit state. For all other limit states, use a value of 1.00 for all load modifiers.

IV-04.03.03 Live Loads [3.6]

HL-93 is the designation for the calibrated design live load provided in the LRFD Specifications. It shall be considered the normal design load for NDDOT highway structures.

Where appropriate, additional live loads shall be considered. Additional live loads might include: snow removal equipment on sidewalks and bridge inspection or snooper loads on bridges with large overhangs. If construction equipment or maintenance equipment can or will operate adjacent to retaining walls and abutments, a live load surcharge shall be incorporated into the design.

IV-04.03.04 HL-93 Live Load, LL [3.6.1.2]

The design truck, fatigue truck, design tandem, truck train and lane loads described in the LRFD Specifications shall be used. The double tandem loading described in the commentary to LRFD Article 3.6.1.3.1 will not be used.

IV-04.03.05 Multiple Presence Factor, MPF [3.6.1.1.2]

When a structure is being evaluated for load cases involving more than two lanes of traffic a reduction factor or multiplier can be used. This factor recognizes the reduced probability that all lanes will be fully loaded at the same time. It should be noted that the LRFD Specifications requires a 1.2 factor to be used for the design of structures carrying a single lane of traffic.

IV-04.03.06 Dynamic Load Allowance, IM [3.6.1.6]

What was known as impact in the Standard Specifications is called dynamic load allowance in the LRFD Specifications. The base dynamic load allowance factors are presented in LRFD Table 3.6.2.1-1. Designers shall note that the base value is reduced for buried components and that dynamic load allowance need not be applied to wood components.

IV-04.03.07 Pedestrian Live Load, PL [3.6.1.6]

For conventional highway bridges with sidewalks, a truck live load across the sidewalk, instead of pedestrian live loads, shall be applied. Bridges carrying only pedestrian or bicycle traffic shall be designed with a live load intensity of 0.085 ksf.

IV-04.03.08 Braking Force, BR [3.6.4]

The braking force shall be applied to all design lanes carrying traffic headed in the same direction. All design lanes shall be loaded simultaneously for bridges likely to be one-directional in the future. Braking forces are to be applied at a height six feet above the roadway surface and in a longitudinal direction.

The dynamic load allowance factor is not applied to braking forces. However, multiple presence

factors shall be used. With elastomeric bearings, the force shall be applied at the bearing.

IV-04.03.09 Centrifugal Force, CE [3.6.3]

Similar to braking forces, multiple presence factors shall be applied to the centrifugal force and the dynamic load allowance shall not be applied. Centrifugal forces are to be applied at a height six feet above the roadway surface.

IV-04.03.10 Live Load Application to Buried Structures

For buried structures, a design truck or tandem, without the lane load, is applied to the roadway and distributed through the fill. If the fill is 1' or less, the live load is applied as a concentrated load to the top of the structure. For fills over 1', the tire footprint spreads out through the soil fill.

IV-04.03.11 Live Load Surcharge, LS [3.11.6]

Retaining walls and abutments typically need to be designed for load combinations with live load surcharge. The equivalent soil heights to be used for different heights of abutments and retaining walls are provided in LRFD Tables 3.11.6.4-1 and 3.11.6.4-2.

IV-04.03.12 Wind Loads, WS [3.8.1.2, 3.8.2]

The design wind speed is 100 mph. For most structures the total height will be below 30 feet and base wind pressures can be used for design. The vertical overturning wind load described in LRFD Article 3.8.2 shall be considered in design.

IV-04.03.13 Wind on Live Load, WL [3.8.1.3]

The force effects of wind on live load shall be considered for the Strength V and the Service I load combinations. The force components (parallel and normal) for different wind skew angles are presented in LRFD Table 3.8.1.3-1. The wind on live load forces are applied at a height 6 feet above the top of the deck.

IV-04.03.14 Earthquake Effects, EQ [3.10]

All of North Dakota is in Seismic Performance Zone 1 with acceleration coefficients varying between 1 and 2.5%. With very small acceleration coefficients, earthquake forces will rarely govern the design of NDDOT structures. However, the LRFD Specifications require that Performance Zone 1 structures satisfy detailing requirements. These requirements pertain to the length of bearing seats supporting superstructure elements and the capacity of a force path for superstructure dead loads to be transferred to substructure elements.

In accordance with LRFD Article 3.10.9.2, the connection between superstructure and substructure shall be capable of transferring 20% of the vertical reaction due to permanent load and tributary live load. Fixed bearing anchorages shall be capable of resisting a horizontal force equal to 20% of the total vertical dead load.

IV-04.03.15 Ice Loads, IC [3.9]

The design ice load shall be 4.0' of ice on the Missouri River and 3.0' of ice on other streams with a crushing strength of 16.0 ksf.

IV-04.03.16 Earth Pressure, EV, EH or ES [3.5.1, 3.5.2] [3.11.5, 3.11.6]

For applications with level backfill, simplified equivalent fluid methods can be used. For level backfill applications the walls shall be designed for an active earth pressure of 0.040 kcf equivalent fluid weight.

The horizontal resultant for lateral earth pressures shall be assumed to act at a height of H/3 from bottom of the wall.

IV-04.03.17 Temperature, Shrinkage, Creep & Settlement, TU, SH, CR & SE [3.12]

Temperature, shrinkage, creep, and settlement produce several structural effects. They generate internal forces, redistribute internal forces, and produce movements.

The design thermal movement in determining longitudinal thermal force effects, designing joint openings, and designing bearings, shall be calculated using LRFD Procedure A. For a cold climate, the temperature ranges are -30°F to 120°F for steel and are 0°F to 80°F for concrete as shown in Table 3.12.2.1-1.

NDDOT's design relative humidity is 70% for concrete shrinkage computations.

IV-04.03.18 Pile Downdrag, DD

For situations where long friction piles or end-bearing piles penetrate through a soft, top layer of material, there may be settlement of the soft layer. The settlement of this layer will add load to the pile through friction. The foundation report will provide the amount of downdrag to be considered.

IV-04.03.19 Friction Forces, FR [3.13]

Friction forces are used in the design of several structural components. For example, substructure units supporting bearings with sliding surfaces shall be designed to resist the friction force required to mobilize the bearing.

IV-04.03.20 Sliding Bearings

LRFD Table 14.7.2.5-1 provides design coefficients of friction for PTFE sliding surfaces. Design for -30ºF and interpolate.

IV-04.03.21 Extreme Event

The probability of extreme event loads occurring simultaneously is extremely small and therefore, is not to be applied concurrently. In some cases, extreme event loads are mutually exclusive. A vessel collision load cannot occur when the waterway is iced over.

IV-04.03.22 Uplift [Table 3.4.1.-2]

For curved bridges with skews or continuous bridges with spans that vary significantly, there is a possibility of uplift at the end supports. For situations where a sidespan is less than 70% of the adjacent continuous span, uplift shall be considered. Uplift may occur during construction if deck placement is not sequenced properly or during service due to the application of live load if the spans are not balanced. If uplift occurs, the performance of the bearings and expansion joints may be compromised. When evaluating a structure for uplift the load factors for permanent load shall be reviewed. Minimum and maximum factors shall be combined for different elements to generate the most conservative or largest uplift force effect.

IV-04.04 Structural Analysis and Evaluation

IV-04.04.01 Load Distribution [4.6.2]

The LRFD Specifications encourage the use of either refined or approximate methods of analysis. An approximate method of analysis can be utilized to determine the lateral live load distribution to individual girders for typical highway bridges. Lateral live load distribution factors are dependent on multiple characteristics of each bridge. There are specific ranges of applicability for the use of approximate methods of analysis. Extending the application of such approximate methods beyond the limits requires sound and reasonable judgement. Otherwise refined analytical methods shall be used.

IV-04.04.02 Dead Load Distribution

For beam bridges the dead load of the deck is distributed to the beams based on their respective tributary widths. Superimposed dead loads (future wearing surface, railings, barriers, and medians) are to be distributed equally to all beam lines.

For concrete slab bridges the weight of the barrier loads shall be distributed to the edge strip. For design of the interior strip, the weight of the barriers shall be distributed across the entire width of the slab and combined with other superimposed dead loads.

Conduit loads supported by hangers attached to the deck and sign structure loads shall be distributed equally to all beams.

IV-04.04.03 Live Load Distribution

Equations and tables for live load distribution factors are provided in the LRFD Specifications. For typical beam bridges distribution factors are provided for interior beam flexure (single lane, multiple lanes, and fatigue), and interior beam shear (single lane, multiple lanes, and fatigue). The lever rule and distribution formulas shall be used to determine the amount of live load carried by the exterior beam. LRFD C4.6.2.2.2d provides a formula for computation of an additional distribution factor for bridges that have diaphragms or cross frames. Use of the rigid cross section or pile equation distribution factor is not required for design of exterior beams. Also, the 1.2 multiple presence factor for one lane loaded shall not be applied to exterior beams.

IV-04.04.04 Steel and Prestressed Concrete Beams

Unlike the Standard Specifications, the live load distribution factors (LLDF) for beam bridges are dependent on the stiffness of the components that make up the cross section [LRFD Equation 4.6.2.2.1-1]. Theoretically, the distribution factor changes for each change in cross section. However, this is more refinement than necessary. For simple span structures a single LLDF (computed at midspan) may be used. For continuous structures a single LLDF may be used for each positive moment region and for each negative moment region, with the moment regions defined by the dead load contraflexure points. For bridges with consistent geometry (same number of beam lines in each span, etc.) the largest positive moment LLDF may be used for all positive moment locations. Similarly, the largest negative moment LLDF may be used for all

negative moment regions.

For skewed superstructures, the LLDF for end shear throughout the length of the girder shall be used.

IV-04.04.05 Slab Spans and Timber Decks [4.6.2.3]

Concrete slabs and timber decks shall be designed using a one-foot wide longitudinal strip. The LRFD Specifications provide equations for live load distribution factors (LLDF) that result in equivalent strip widths, E, that are assumed to carry one lane of traffic. The equivalent strip width shall be converted to a live load distribution factor for the unit strip by taking the reciprocal of the width.

$$\mathrm{LLDF} = \frac{1}{\mathrm{E}}$$

IV-04.04.06 Sidewalk Pedestrian Live Load [3.6.1.6]

Unlike the Standard Specifications, no reduction in sidewalk pedestrian live load intensity based on span length and sidewalk width is provided in the LRFD Specifications.

The vehicular live load shall be placed on the sidewalk and in adjacent traffic lanes with no pedestrian live load on the sidewalk.

IV-04.04.07 Pedestrian Bridge Live Load

Bridges designed for only bicycle or pedestrian traffic use a slightly higher pedestrian live load (0.085 ksf).

IV-04.04.08 LRFD Exceptions

The LRFD Specifications are comprehensive. In addition to an updated design methodology, the new specifications also include information that has not been included in the Standard Specifications. For example, the LRFD Specifications contain guidance for vessel impact loads that previously were contained in the “Guide Specification and Commentary for Vessel Collision Design of highway Bridges.” However, not all AASHTO design guidance has been incorporated in the LRFD Specifications. A few special topics have not yet been incorporated into the new specification. Information on these topics is given below.

IV-04.04.09 Railroad Bridges and Bridges or Structures near Railroads

Railroad bridges are to be designed according to the most current “American Railway Engineering and Maintenance-of-Way Association (AREMA) Specifications.”

Designers shall be aware that railroads may have specific criteria for structural design of items carrying their tracks or in the vicinity of their tracks. The design criteria varies from railroad to railroad.

IV-04.05 Concrete Structures

IV-04.05.01 Prestressed Losses

Use the approximate method for estimating time-dependent losses in prestressing steel. Do not consider any elastic gains in prestressing steel in determining losses.

Use an average annual ambient relative humidity of 70% in calculating prestressing losses due to concrete creep and shrinkage as per AASHTO Figure 5.4.2.3.3-1.

IV-04.06 Steel Structures

This subsection intentionally left blank for future use.

IV-04.07 Aluminum Structures

This subsection intentionally left blank for future use.

IV-04.08 Wood Structures

This subsection intentionally left blank for future use.

IV-04.09 Decks and Deck Systems

IV-04.09.01 Minimum Deck Thickness [9.7.1.1]

According to 13.7.3.1.2, for decks supporting concrete barriers, the minimum edge thickness for concrete deck overhangs shall be 8". Therefore, for a constant slab thickness, the minimum deck thickness to be used is 8".

IV-04.09.02 Deck Design Method of Analysis [4.6.2]

The traditional approximate method of analysis shall be used for deck design. The empirical deck design method in 9.7.2 shall not be used. The deck shall be treated as a continuous beam. Moments as provided in Table A4-1 are to be applied at the design section. The use of Table A4-1 must be within the assumptions and limitations listed at the beginning of the appendix.

IV-04.09.03 Crack Control [5.7.3.4]

A Class 1 exposure condition where $\gamma_e = 1.00$ shall be used.

IV-04.09.04 Overhang Design [A13.4]

A13.4.2 specifies that the vehicle collision force to be used in deck overhang design is to be equal to the rail capacity R_w. This ensures that the deck will be stronger than the rail and that the yield line failure mechanism will occur in the parapet. Because of the large difference between rail capacity and collision force, NDDOT requires the deck overhang to carry the lesser of the rail capacity R_w or 1.1 x F_t.

IV-04.09.05 Skewed Decks [9.7.1.3]

The primary transverse reinforcement shall be placed perpendicular to the main supporting components regardless of skew.

IV-04.10 Foundations

IV-04.10.01 Allowable Pile Loadings

The maximum allowable pile loads for steel H-pile is as follows:

<u>Steel H-pile:</u>

Commonly used sizes:	HP10 x 42 - - - - 105 tons
(F_y = 50 ksi)	HP12 x 53 - - - - 130 tons
	HP14 x 73 - - - - 180 tons
	HP14 x 102 - - - 250 tons

These allowables are based on $0.25F_y$ (12,500 psi) multiplied by a 1.3333 (or $^4/_3$) factor.

IV-04.10.02 Pile Driving Formula

Include the following pile driving formula and note in the bridge plan sheets.

For double acting or single acting diesel hammers, calculate the safe bearing value of piles by the following formula:

$$P = \frac{4.5E}{S + 0.2} \times \frac{W + 0.2M}{W + M}$$

Where:

P = Safe bearing value, in pounds.
W = Weight of striking parts (ram), in pounds.
M = Weight of parts being driven, in pounds. Includes pile weight, anvil (if any), driving cap, etc.
E = Energy per blow, in foot-pounds.
S = Average penetration of pile in inches per blow for last ten blows.

For single acting hammers, calculate E by multiplying observed stroke (ft) and W (lbs).

IV-04.11 Abutments, Piers, and Walls

IV-04.11.01 Integral Abutment Design/Analysis

Piling shall be designed for axial loads only. It shall be assumed that one half of the approach slab vertical load is carried by the abutment. The live load shall be distributed over the entire length of abutment. The number of lanes shall be applied that will fit on the superstructure adjusted by the multiple presence factor. Eccentricity of the live load shall be taken into account by applying one lane reaction up to the maximum number of lanes that will fit on the bridge.

The abutment front and back face vertical bars shall be designed for the following load case:

- It shall be assumed that abutment wall acts as a cantilever fixed at the bottom of the superstructure and free at the bottom of the wall.
- Design for a lateral load of 865 psf with the active horizontal earth pressure factor of 1.50 listed in Table 3.4.1-2. If U-shaped abutment is used, the DL moment from the wing shall be distributed over the first 10' of the abutment.
- Wings parallel to abutment do not contribute to design loads for the abutment wall.

The front and back face horizontal bars shall be designed for the passive soil pressure, which results when the bridge expands. The wall shall be considered a continuous beam with piles as supports and the following moment designed for:

$$M_{up} = \gamma_{EH} \times \frac{(865 * L^2)}{10}$$

$$L = \text{Pile Spacing}$$

IV-04.11.02 Parapet Abutment Design/Analysis

For design of piling or footing bearing pressures, as a minimum consider the following load cases:

Construction Case 1 – Strength I (0.90DC + 1.00EV + 1.5EH + 1.75LS)
Abutment has been constructed and backfilled, but the superstructure and approach panel are not in place. Minimum load factors for vertical loads and maximum load factors for horizontal loads shall be used. It shall be assumed that a live load surcharge is acting.

Construction Case 2 – Strength I (1.25DC)
Abutment has been constructed, but not backfilled. The superstructure has been erected, but approach panel is not in place. The maximum load factor for dead load shall be used.

Final Case 1 – Strength I (1.25DC + 1.35EV + 0.90EH + 1.75LL)
Bridge is complete and approach panel is in place. Maximum load factors for vertical loads and minimum load factors for horizontal earth pressure shall be used.

Final Case 2 – Strength I (1.25DC + 1.35EV + 1.50EH + 1.75LL)
Bridge is complete and approach panel is in place. Maximum load factor for all loads shall be used.

Abutments shall be designed for active pressure using an equivalent fluid weight of 0.040 kcf. Neglect passive earth pressure in front of abutments.

LRFD Table 3.11.6.4-1 for determination of live load surcharge equivalent soil heights shall be used. Live load surcharge only shall be applied when there is no approach panel.

It shall be assumed that one half of the approach slab vertical load is carried by the abutment.

The superstructure loads (dead load and live load) shall be distributed over the entire length of abutment. For live load, the number of lanes that will fit on the superstructure adjusted by the multiple presence factor shall be applied. The eccentricity of the live load needs to be taken into account.

The footing thickness shall be designed such that no shear reinforcement is required. Performance of the Service I crack control check per LRFD 5.7.3.4 is not required for abutment footings.

The abutment stem and backwall shall be designed for horizontal earth pressure and live load surcharge loads.

IV-04.11.03 Wingwalls

For integral abutments with straight wings the maximum length is 15'-0" and the design load shall be 550 psf with the active horizontal earth pressure factor of 1.50 listed in Table 3.4.1-2. The horizontal reinforcing used shall be the same in the front and back face.

For integral abutments with wings parallel to the centerline roadway, the maximum length of the wing shall be 20'-0", measured along the inside face and shall be designed for an equivalent fluid pressure of 40 pcf with the active horizontal earth pressure factor of 1.50 listed in Table 3.4.1-2. The wings shall be on the outside of the barriers and the approach slab shall be between the wings. A ½" joint filler shall be placed between the wing and the approach slab.

Reinforcement shall be provided through the construction joint at the intersection of the wing and abutment wall to transfer wingwall loads to the abutment.

IV-04.11.04 Crash Walls [AREMA 2.1.5.1 and C-2.1.5.1]

Refer to latest AREMA publication for crash wall requirements.

IV-04.11.05 Retaining Walls

Retaining wall designs need to consider several parameters. These parameters include:

- Height of the wall
- Geometry of the wall (curved or straight)
- Type of material retained
- Geometry of the backfill (level or sloped)
- Magnitude of live load surcharge
- Whether or not traffic barriers will be incorporated into the top of the wall (vehicle collision loads)
- Whether or not noise walls will be supported on the wall
- Location of the water table
- Quality of subgrade material (supported on spread footings or pile foundations)

IV-04.11.06 Cantilever Retaining Walls

In many cases a conventional reinforced concrete retaining wall is the appropriate solution for a project.

IV-04.11.07 Counterfort Retaining Walls

Counterforted retaining walls are economical for wall heights over 40'. Counterforted walls are designed to carry loads in two directions. Earth pressures are carried laterally with horizontal reinforcing to thickened portions of the wall. The thickened portion of the wall contains the counterfort, which is designed to contain vertical reinforcement that carries the overturning loads to the foundation.

IV-04.11.08 Anchored Walls

Anchored walls are used when the height of the earth to be retained by the wall is considerable and when all other types of retaining walls prove to be uneconomical. In order to reduce the section of the stem, an anchoring system is provided at the back of the wall. Anchoring is typically accomplished by embedding a concrete block “dead man” in earth fill and connecting it to the stem of the wall with anchor rods. Alternatively, the anchors may be incorporated into soil nails or rock bolts. The feasibility of using anchored walls should be evaluated on a case-by-case basis after all other types of retaining walls have been ruled out as an option.

IV-04.11.09 Mechanically Stabilized Earth Walls

Mechanically stabilized earth walls are reinforced soil retaining wall systems that consist of vertical or near vertical facing panels, metallic or polymeric tensile soil reinforcement, and granular backfill. The strength and stability of mechanically stabilized earth walls is derived from the composite response due to the frictional interaction between the reinforcement and the granular fill. Mechanically stabilized earth systems can be classified according to the reinforcement geometry, stress transfer mechanism, reinforcement material, extensibility of the reinforcement material, and the type of facing.

MSE walls may be used in lieu of conventional gravity, cantilever, or counterfort retaining walls. MSE walls are more tolerant to differential movement than other retaining walls. MSE walls potentially offer significant cost advantages in comparison to other wall types, especially when wall heights exceed 10 feet.

There are certain situations in which the use of MSE walls needs to be further analyzed. If there is a potential for scour or erosion near the wall facing, an alternative wall system should be considered. Since excavation is required behind the wall to allow for the reinforcement elements, MSE wall should not be used when the area behind the wall is limited. Also if utilities will be placed within the reinforced zone another wall type should be considered.

The four major types of mechanically stabilized earth walls used by the NDDOT are shown below:

1. Segmental Precast Concrete Panels. The concrete wall panels shall be a minimum of 5½" in thickness. The tensile elements are typically metallic and are attached to the panels with a mechanical connection. The metallic tensile elements are sensitive to corrosion and therefore this wall facing is not appropriate unless the tensile elements are protected from surface water or corrosive soil. The use of walls with this facing is limited to situations in which the radius is greater than 50'. The design requirements will be described in the Special Provision and plan sheets associated with the project.

2. Modular Block Units. Modular Block units are small, rectangular concrete blocks with hollow or solid cores. The block units typically range from 4" to 8" in height and 8" to 18" in width. The tensile elements are polymeric and are sandwiched between the rows of blocks, creating a frictional connection. A wall with this facing is more tolerant to differential movement than segmental precast concrete panel walls. Modular Block MSE designs shall meet the design requirements of the Special Provision and the plan sheets associated with the project.

3. Welded Wire Grids. This facing is composed of rows of “L” shaped wire forms stacked one upon another. Geotextiles are wrapped behind the wire forms, similar to geosynthetic faced walls, to prevent loss of aggregate. When settlement is an issue, this wall facing can be used in conjunction with segmental precast concrete panels in a two stage MSE wall applications.

4. Geosynthetic Wrapped. This type of facing is a result of wrapping geotextile reinforcement fabric around the granular fill material, at the face of the wall, and tucking the fabric beneath the ensuing layer. These walls are typically considered temporary since they are susceptible to ultraviolet light degradation, vandalism and damage due to fire.

IV-04.11.10 Prefabricated Modular Walls

Prefabricated modular walls are gravity walls made of interlocking soil-filled concrete or steel modules or bins, rock filled gabion baskets, precast concrete units or modular block units (without soil reinforcement).

Prefabricated modular walls shall not be used under the following conditions:

- On curves with radius of less than 800', unless the curve could be substituted by a series of chords.
- Steel modular systems shall not be used where the ground water or surface runoff is acid contaminated or where deicing spray is anticipated.

Heights greater than 8'.

IV-04.12 Buried Structures [12.6.6]

IV-04.12.01 Design Theory and Assumptions

The outside corners of the box are considered rigid joints. The inside walls are assumed to be hinged on top and bottom and thus carry no moment. The floor and roof are continuous over the inside walls. The clear span plus 8" is used for span calculations.

The critical section for flexure in the roof and floor is considered at the faces of the walls. The critical section for flexure in the walls shall be taken at the faces of the floor and roof.

The critical section for shear on the roof is considered at a distance “d” (effective depth) into the span past the haunch. The critical section for shear on the floor is considered at a distance “d” from the faces of the walls. The critical section for horizontal shear in the walls shall be taken at the faces of the roof and floor.

The roof over the outside barrels of multiple barrel boxes is sloped when considered economical. The floor can either be a flat floor or a curved floor. The projection of the floor beyond the outside walls is ignored in the design calculations.

Moment distribution is used to determine redundant moments at the joints. The column analogy is used to determine the stiffness and carry-over factors for the curved floor and sloped roof. The moment of inertia of members is based on the gross concrete section.

IV-04.12.02 Method of Analysis

The approximate strip method is used for the design with the 1’-0” wide design strip oriented parallel to the direction of traffic (longitudinal direction.)

IV-04.12.03 Load Factors and Combinations

Maximum and minimum load factors for different load components shall be combined to produce the largest load effects. The box culvert shall be designed for the greater moments and shears resulting from the following two load conditions:

1. Dead Load + Live Load + Balanced Horizontal Earth Load

2. 0.8 x (Dead Load + Live Load + Unbalanced Horizontal Earth Load).

IV-04.12.04 Dead Loads

The F_t factor to adjust the vertical earth load carried by the culvert shall equal 1.00. It is intended to approximate soil-structure interaction effects and installation conditions (trench versus embankment).

Compacted fill on top of buried structures is assumed to have a unit weight of 0.120 kcf.

Concrete self-weight computations should be based on 0.150 kcf.

IV-04.12.05 Lateral Earth Pressure

Earth is assumed to have an equivalent fluid pressure of 40 psf per foot of depth. The designer shall check for an unbalanced lateral earth pressure. For the unbalanced condition use 40 psf on one side of the box and 20 psf on the other side. When unbalanced lateral earth pressure is added to the dead and live load, the total is reduced 20% due to improbability of occurrence.

IV-04.12.06 Live Load

The live loading shall consist of the design truck or tandem without the lane load. Box culverts shall be designed for a single loaded lane with the single lane multiple presence factor of 1.2 applied to the load.

The depth of fill shall be taken at the edge of the paved shoulder and includes the pavement thickness. When the depth of fill is less than 2', live loads shall be distributed to the top slab of the culvert as specified in Article 4.6.2.10. When the depth of fill is 2' or more, wheel loads shall be uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area, and increased by either 1.15 times the depth of the fill in foundation fill, or the depth of the fill in ordinary backfill.

Where such areas from several wheels overlap, the total load shall be uniformly distributed over the area defined by the outside limits of the individual areas.

For single-span culverts, the effects of the live load may be neglected where the depth of fill is more than 8’ and exceeds the span length; for multiple span culverts, the effects may be neglected where the depth of fill exceeds the distance between the inside faces of the exterior walls.

When the calculated live load based on the distribution of the wheel load through earth fills, exceeds the calculated live load based on slab distribution, the latter moment shall be used.

Live loads for multiple barrel boxes may be continuous or discontinuous by spans. The maximum moments are used.

The dynamic load allowance shall conform to Article 3.6.2.2.

IV-04.13 Railings

IV-04.13.01 Design Requirements

The FHWA requires all bridges carrying traffic on the National Highway System (NHS) to be crash tested in accordance with "NCHRP Report 350 Recommended Procedures for the Safety Performance Evaluation of Highway Features." There are six levels of service and testing depending on vehicle size and speed. A list of crash tested railings is found on the following FHWA Web sites:

- http://www.fhwa.dot.gov/bridge/bridgerail/
- http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/

Crash testing requirements may be waived if the railing in question is similar in geometrics to an approved crash tested rail and an analytical evaluation shows the railing to be crash worthy. This allows minor changes to crash tested railings without having to go through the time and expense of crash testing. Any such evaluation must be approved by the FHWA.

IV-04.13.02 Traffic Railing [Table A13.2-1]

For new bridges and deck replacements, use the 36" single slope barrier with vertical reinforcing steel consisting of #5 bars spaced at 8".

IV-04.14 Joints and Bearings

IV-04.14.01 Bridge Movements and Fixity

To determine movements for bearings and joints, the point of fixity must be established for the bridge or bridge segment. The point of fixity is the neutral point on the bridge that does not move horizontally as the bridge experiences temperature changes.

IV-04.14.02 Expansion Joints [Table 3.4.1-1]

Joint openings shall be designed for movements associated with a temperature range of 150°F (-30°F to 120°F) for steel bridges. For concrete bridges a temperature range of 80°F (0°F to 80°F) shall be used.

The LRFD Specification lists a load factor of 1.2 to be used in the calculation of movements. Based on past performance of joints, a load factor of 1.0 for temperature effects shall be used to size components.

IV-04.14.03 Bearings [4.7.4.4, 3.10.9]

The purpose of a bridge bearing is to transmit loads from the superstructure to the substructure while facilitating translation and rotation. Four types of bearings are typically used.

1.) Expansion Bearing:
    Transfers vertical load
    Allows lateral movement in two directions
    Allows longitudinal rotation

2.) Guided Expansion Bearing:
    Transfers vertical load and lateral load in one direction
    Allows lateral movement in one direction
    Allows longitudinal rotation

3.) Limited Expansion Bearing:
    Transfers vertical load and lateral load
    Allows limited lateral movement in one direction
    Allows longitudinal rotation

4.) Fixed Bearing:
    Transfers vertical load and lateral load
    Resists lateral movement
    Allows longitudinal rotation

The width of pier caps and abutment seats shall be checked to ensure the minimum support length requirements for Seismic Performance Category 1 are satisfied.

IV-04.14.04 Loads and Movements

Bearings shall be designed for movements associated with a temperature range of 150°F (-30°F to 120°F) for steel bridges. For concrete bridges a temperature range of 80°F (0°F to 80°F) shall be used.

Elastomeric bearings shall be designed for service loads and without Dynamic Load Allowance (IM).

Uplift at bearings is not permitted. Bearings shall be checked for uplift using the Strength 1 load combination with the minimum load factor for dead load.

IV-04.14.05 Elastomeric Bearings

Elastomeric bearings are to be designed using Method A of the AASHTO LRFD Specifications. Designs shall be based on an elastomer with a durometer hardness of 55. The minimum shear modulus (G) for this material is 115 psi. The maximum shear modulus is 165 psi.

IV-04.14.06 Minimum Compressive Load [14.7.6.4, 14.6.3.1]

LRFD 14.7.6.4 requires that elastomeric bearings be secured against horizontal movement when $^1/_5$ of the minimum vertical load is less than the factored horizontal shear force H_u generated in the bearing due to temperature movement.

$$H_{bu} = G \cdot A_{pad} \cdot \frac{\Delta_u}{h_{rt}}$$

Therefore

$$\frac{P_{min}}{5} \geq G \cdot A_{pad} \cdot \frac{\Delta_u}{h_{rt}}$$

Based on past performance, a load factor equal to 1.0 is used with a 75°F temperature change and the maximum shear modulus to calculate H_u. Also, we know that $A_{pad} = A \cdot B$.

Then the minimum required compressive load is:

$$P_{min} \geq 5 \cdot 0.165 \cdot A \cdot B \cdot \frac{1.0 \cdot \Delta_h}{h_{rt}}$$

which becomes

$$P_{min} \geq 0.825 \cdot A \cdot B \cdot \frac{1.0 \cdot \Delta_h}{h_{rt}}$$

IV-04.14.07 Fixed Bearings

Fixed elastomeric bearings shall be designed for a maximum compressive stress of 0.880 ksi. This includes a 10% increase for fixity.

IV-04.14.08 Expansion Bearings

Expansion elastomeric bearings are reinforced and shall be designed for a maximum compressive stress of 1.00 ksi or less.

IV-04.14.09 Pot or Disk Bearings

Pot or disk bearings shall be used where the loads are too high or the movements and rotations are too large to be readily accommodated with elastomeric bearings.

All applicable design loads and movements for pot bearings must be provided in the contract documents. Due to a variety of preferences among pot bearing fabricators, explicit details are not provided in the plans. Instead, the fabricator determines the sizes of all of the bearing components, from the masonry plate to the sole plate. As a guide, the following equation may be used to estimate the height (rounded to the nearest ¼ inch) of the assembly for design:

Height (inches) = 6.5 + Load (kips) / 400

The following note shall be included on the appropriate substructure sheets when pot bearings are used:

> *Final construction elevations for bridge seats shall be determined based on the actual height of pot bearing assemblies furnished by the Contractor. Any required adjustment of seat elevations shall be made by the Contractor at no cost to the Department.*

Fixed pot bearings provide rotation, but no movement.

Guided expansion pot bearings allow for free movement in one direction and provide rotational capacity. However, movement perpendicular to the free movement direction is restrained. For curved bridges, it shall be assumed the free movement direction to be along a chord connecting the ends of the beam. Guide bars must resist a minimum of 10% of the vertical load applied to the bearing.

Expansion pot bearings provide for rotation and unguided movement in all horizontal directions.

IV-05.01 Railroad Structures

Generally the Construction and Maintenance Agreements of grade separations between the North Dakota Department of Transportation (NDDOT) and the Railroad and the City, in some cases, involves various divisions within NDDOT, the responsibilities regarding these agreements will be as follows:

1. Communication with the Railroad as to the location of the bridge, the vertical and horizontal clearances, locations of piers and abutments will be the responsibility of the Bridge Division with concurrence of the Design Division.

2. After the clearances and span arrangements have been approved by the Railroad, Bridge Division will notify Design Division. The respective Design Section will supply the right of way limits to the Right-of-Way Section.

3. The Bridge Division will prepare the Construction and Maintenance Agreement and submit to the Railroad along with Exhibit C (Nondiscrimination Provisions).

4. The Right-of-Way Section in Design will negotiate with the Railroad for necessary right of way (Exhibit A).

5. The Railroad will execute the agreement and return it to the Bridge Division along with Exhibit A, Exhibit B (cost estimate for work to be done by the Railroad), Exhibit C and Exhibit D (flagging requirements and costs).

6. Bridge Division will have the agreement approved by Legal and signed by the parties involved:

7. Progressive billings will be sent from the Railroad to the Bridge Division.

8. The Bridge Division will submit the bill to the appropriate district for review and concurrence. Upon District approval, Bridge will prepare the progressive estimate.

9. The Right-of-Way Section will be responsible for the payment of right of way.

10. Once the project is completed, the District will make a final inspection with the railroad and advise Construction and the local road authority.

11. Construction will process formal acceptance with the Railroad. Copies sent to FHWA, Bridge, Design, Finance, District, and the local road authority.

12. Upon receipt of the final bill and acceptance from Construction, the Bridge Division will prepare the final progress payment less one percent retainage and request an audit of the Railroad.

13. Upon receipt of the final audit report, Bridge Division will process the final estimate of payment.

Appendix IV-06 A	Predetermination of Pile Lengths Based on Analysis of Soil Properties
Appendix IV-06 B	Installation of Utilities on Highway Structures
Appendix IV-06 C	Pedestrian & Shared-Use Facilities on Structures
Appendix IV-06 D	Preliminary Engineering Meeting Agenda
Appendix IV-06 E	Integral Straight Abutment Design (LRFD Specifications)
Appendix IV-06 F	Checklist for Bridge Plans (SFN 17180)
Appendix IV-06 G	Interchange Clearance Diagram
Appendix IV-06 H	Single Slope Barrier Details

Appendix IV-06 A Predetermination of Pile Lengths Based on Analysis of Soil Properties

1. APPLICATION:

By inspection of the soil borings, break the subsoil into specific zones or stratums. This can be done individually or as a group, depending upon the horizontal cross-sectional similarity of the material. Utilize the standard penetration test results, soil description, and the laboratory determination of the physical properties of the soil in establishing zones.

Lengths are determined by computing supporting capacity of the zones penetrated. Estimated penetration will be at the elevation which the computed capacity equals the required capacity times a safety factor of three.

Formulas for computing supporting capacity are as follows:

Cohesive Soils

This category covers clays with an apparent angle of internal friction less than 15°. Capacity is primarily a function of the shear strength of the soil which is established from the unconfined compression tests.

$R_f = ½ Q_u Z A_p T \Delta d$ (1)

$R_t = ½ Q_u Y A_t$ (2)

Where:

R_f = Increment of supporting capacity developed through skin friction (lbs)

R_t = Increment of supporting capacity developed through tip resistance (lbs)

Q_u = Confined compressive strength (lbs/sq ft)

Z = Skin friction coefficient (dimensionless)

Y = End bearing coefficient (dimensionless)

A_p = Average surface area of pile (sq ft/ft) of penetration in zone

A_t = Area of pile tip (sq ft)

T = Taper coefficient (dimensionless)

Δd = Embedded length of pile in zone (ft)

Where unconfined compressive strength data is inadequate, use Chart J for estimate of Q_u based on the standard penetration test blow count.

For values of Z, based on the standard penetration test, use Chart A or Chart C. Chart C has Z' values designated for use in the Lake Agassiz Basin Area (Red River Valley). For remainder of the state, use Z values, Chart A.

For values of Y, based on the standard penetration test (N), use Chart B.

The taper coefficient "T" is considered unity in all areas except in Lake Agassiz Basin clays with "N" less than 20. For values of T, based on the amount of pile taper, use Chart D. When using constant cross-sectional area piles, use taper = O or T = 0.6.

Granular Soils

This category covers sands with an angle of internal friction (φ) considered to be between 25° - 45°. Capacity is considered primarily a function of the confining pressures.

$R_f = n\ K_\varphi\ P_d\ \text{Sin}\ A_p\ \Delta d$ (3)

$R_t = N_q\ A_t\ P_D$ (4)

Where:

n = Skin friction coefficient (dimensionless) used for cases where φ (friction angle of soil) differs from (arc tan of coefficient of friction between soil and pile)

K_φ = Factor relating vertical soil pressure to soil pressure acting on pile walls (dimensionless)

P_d = Vertical effective pressure in soil at any depth d (lbs/sq ft)

P_D = Vertical effective pressure in soil at pile tip (lbs/sq ft)

N_q = Factor relating vertical soil pressure to supporting pressure beneath the pile tip

(R_f, R_t, A_p, A_t, and Δd are same as for cohesive soils)

The value of confining pressure P_d for a zone is determined by one or both of the following formulas:

Above Water Table: For examples of determining P_d see pages 56 and 228 of FHWA publication "Soils and Foundations Workshop Manual."

$P_d = (W_s \times H_s)$

Below Water Table:

$P_d = (W_s - W_s/\text{Sp Gr}) \times H_s$

Where:

W_s = Dry weight of soil (lbs/cu ft, weight of soil above ground water)

H_s = Height of soil from ground surface to effective depth (ft)

Since P_d is a function of H_s, its value will vary from top to bottom of the zone. If the pile will penetrate the full depth of the granular zone, the effective depth will be at a point one-half the full depth of the zone. For partial embedment in a granular zone, use one-half the embedded depth in that zone. For values of P_D to determine R_t, use effective depth to pile tip.

The friction angle, φ, is considered a function of the standard penetration test below count (N) and the confining pressure (P_d). Values for φ can be obtained by reading vertically upward on Chart E to the applicable P_d curve, then horizontally to Chart F.

For values of K, based on the soil friction angle (φ) and the pile displacement (cu ft/ft), use Chart F.

For values of n, based on the soil friction angle (φ) and sand on pile friction angle ( ), use Chart G.

For values of N_q, based on the soil friction angle (φ), use Chart H.

Cohesive - Granular Soils

This category covers intermediate sand-clay mixtures. Values of actual φ are considered to be between 20° - 25°. This is the most difficult category to analyze. At present, capacity is considered a function of both the confining pressures and the shear strength of the soil zone in question.

$R_f = [(ZC) + (n\ K_\varphi\ P_d\ \mathrm{Sin})]\ A_p\ \ \Delta d$ (5)

$R_t = [(YC) + (N_q\ P_D)]\ A_t$ (6)

Where:

C = Cohesion (lbs/sq ft)

All other terms are as defined under cohesive and granular soils.

For values of cohesion (C), based on the unconfined compression (Q_u), use Chart I.

The sand portion is considered independently when determining values of n, K_φ, P_d, P_D, and N_q. Use same procedure as outlined under granular soils.

For the clay portion, determine values of Y and Z as outlined under clay soils.

2. GENERAL NOTES:

a. Each zone which pile is assumed to penetrate delivers a portion of the total bearing capacity based on the appropriate R_f values determined by application of the formula for that particular zone.

   Tip resistance (R_t) is computed on values obtained from the soil zone which tip is finally embedded. As ΣR_f approaches the required bearing, solve for the applicable tip supporting capacity (R_t) in zone which tip is embedded by use of formulas (2), (4), or (6). For clays, rearrangement of formula (1) to solve for Δd will determine penetration into the lowest embedded zone. For sands and sand-clay mixtures, solution by trial and error is usually the easiest method for determining penetration into the lowest embedded zone.

b. In some cases where piles are embedded in soft silt or clay and obtain most of their bearing capacity by friction, it is necessary that the computation of the safe design load be supplemented by a computation of the ultimate bearing capacity of the entire group.

   As a rule of thumb, this group bearing capacity reduction should be made on piles which have a spacing closer than 4 pile diameters.

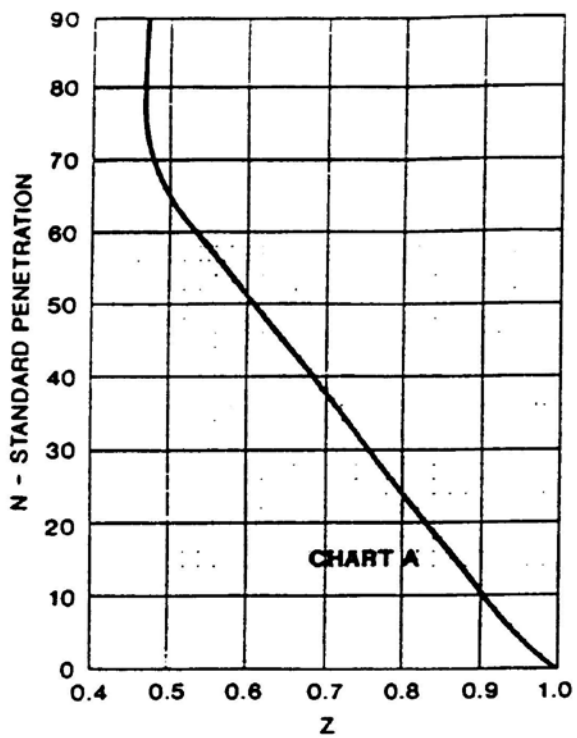
   Formula: Converse-Labarre method as recommended by AASHTO.

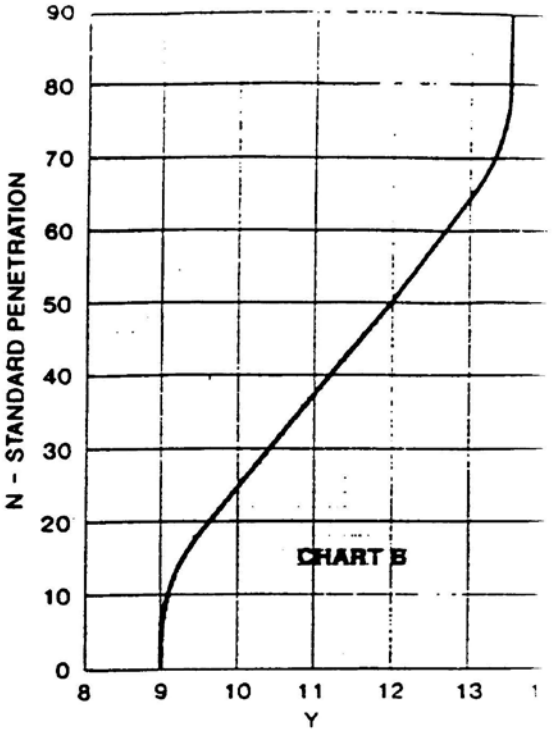
   No reduction due to grouping is computed when piles are end bearing piles. For groups which partake of both actions, only the portion in friction is reduced.

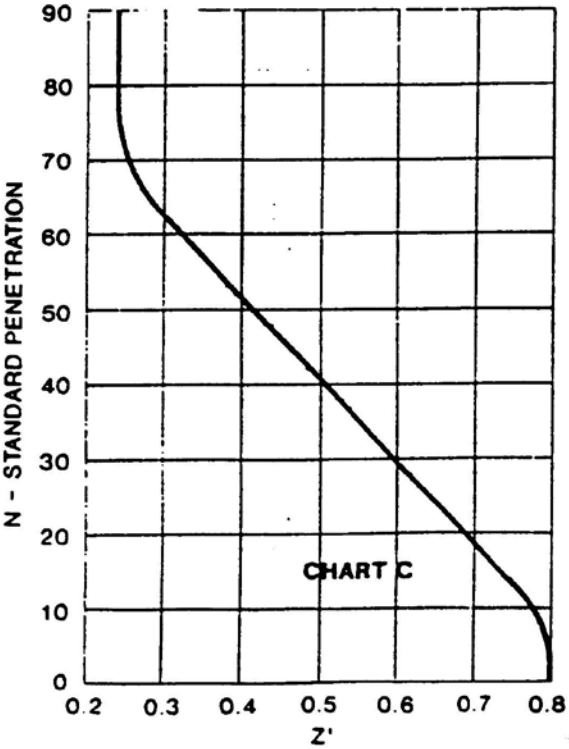
c. When determining the average pile area per foot (A_p) neglect inside flanges and web of H-beam piles and use as a rectangle.

d. Vertical pressures at the pile point while resting on any hard layers of coal or rock is assumed to spread uniformly within a cone (or rectangle) the sides of which are inclined 60° to the horizontal. Safe supporting capacity of any underlying soft material should be analyzed by methods applicable to spread footing design.

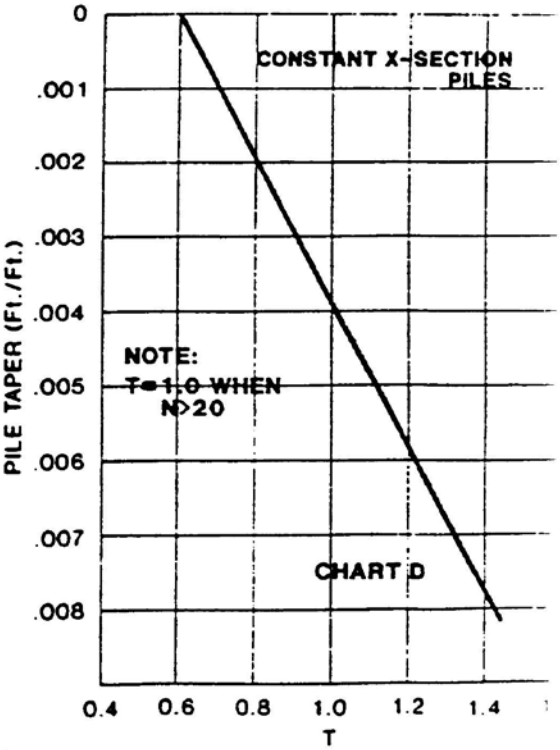
e. Negative friction induced by long term settlement of abutment fills overlaying soft cohesive soils should be accounted for when predetermining abutment pile lengths.



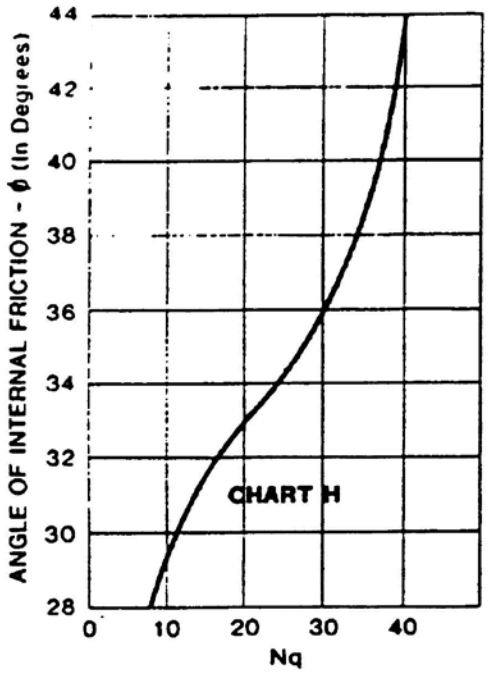


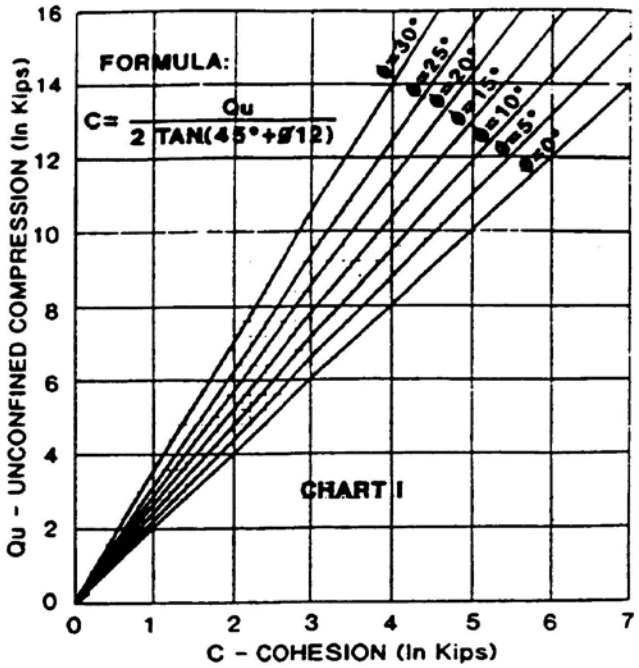


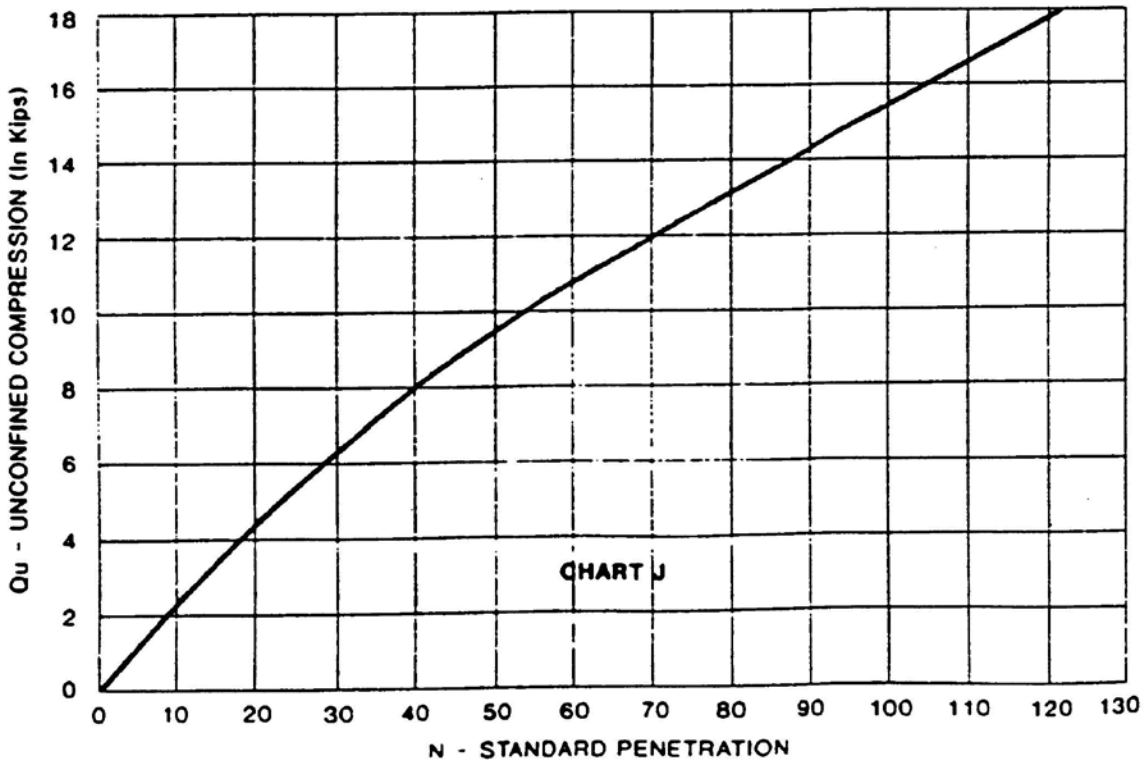
(LAKE AGASSIZ BASIN ONLY)

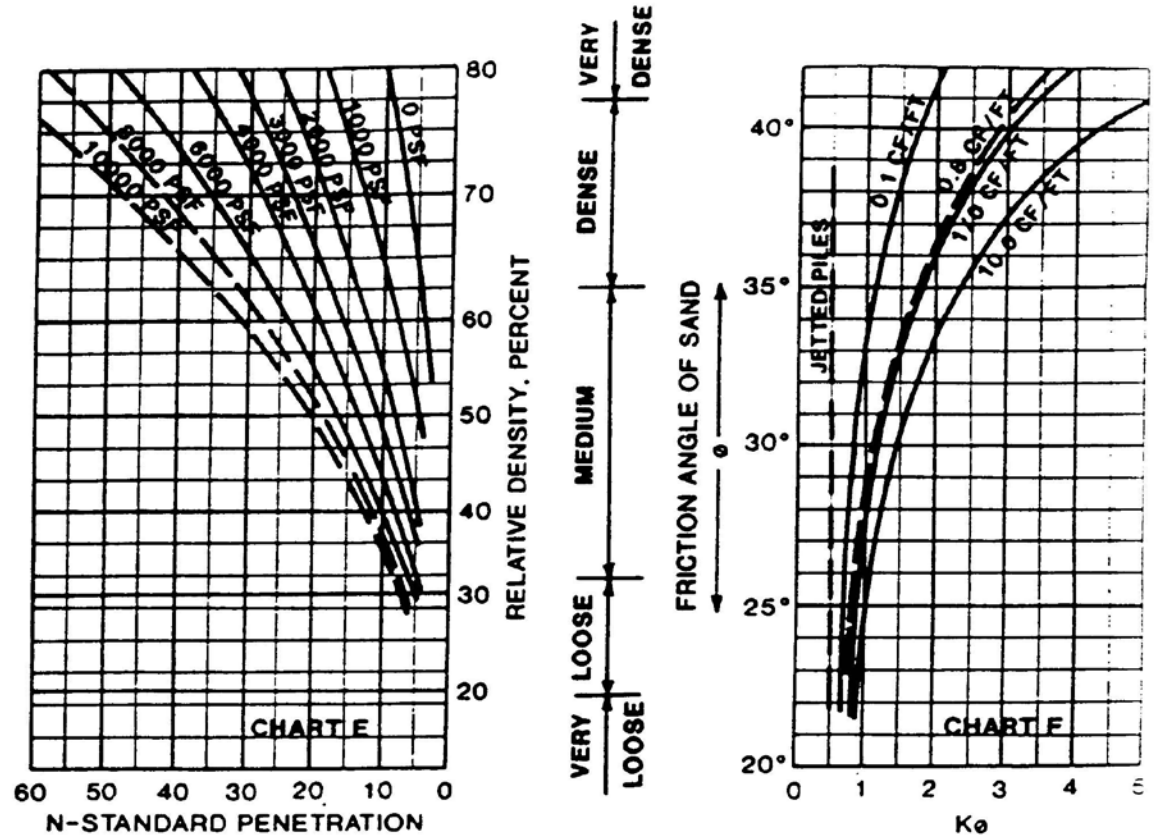


(LAKE AGASSIZ BASIN ONLY)


ANGLE OF INTERNAL FRICTION - ϕ (In Degrees)
44
42
40
38
36
34
32
30
28
0
10
20
30
40
Nq
CHART H


FORMULA:
C= Qu / 2 TAN(45°+ϕ/2)
ϕ=30°
ϕ=25°
ϕ=20°
ϕ=15°
ϕ=10°
ϕ=5°
ϕ=0°
Qu - UNCONFINED COMPRESSION (In Kips)
16
14
12
10
8
6
4
2
0
0
1
2
3
4
5
6
7
C - COHESION (In Kips)
CHART I


Qu - UNCONFINED COMPRESSION (In Kips)
18
16
14
12
10
8
6
4
2
0
0
10
20
30
40
50
60
70
80
90
100
110
120
130
N - STANDARD PENETRATION
CHART J



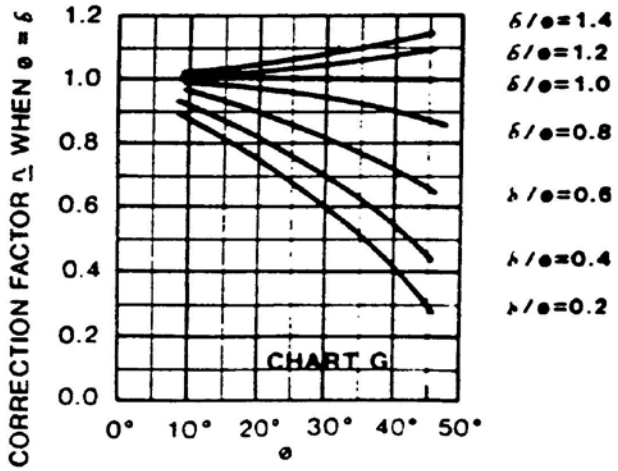
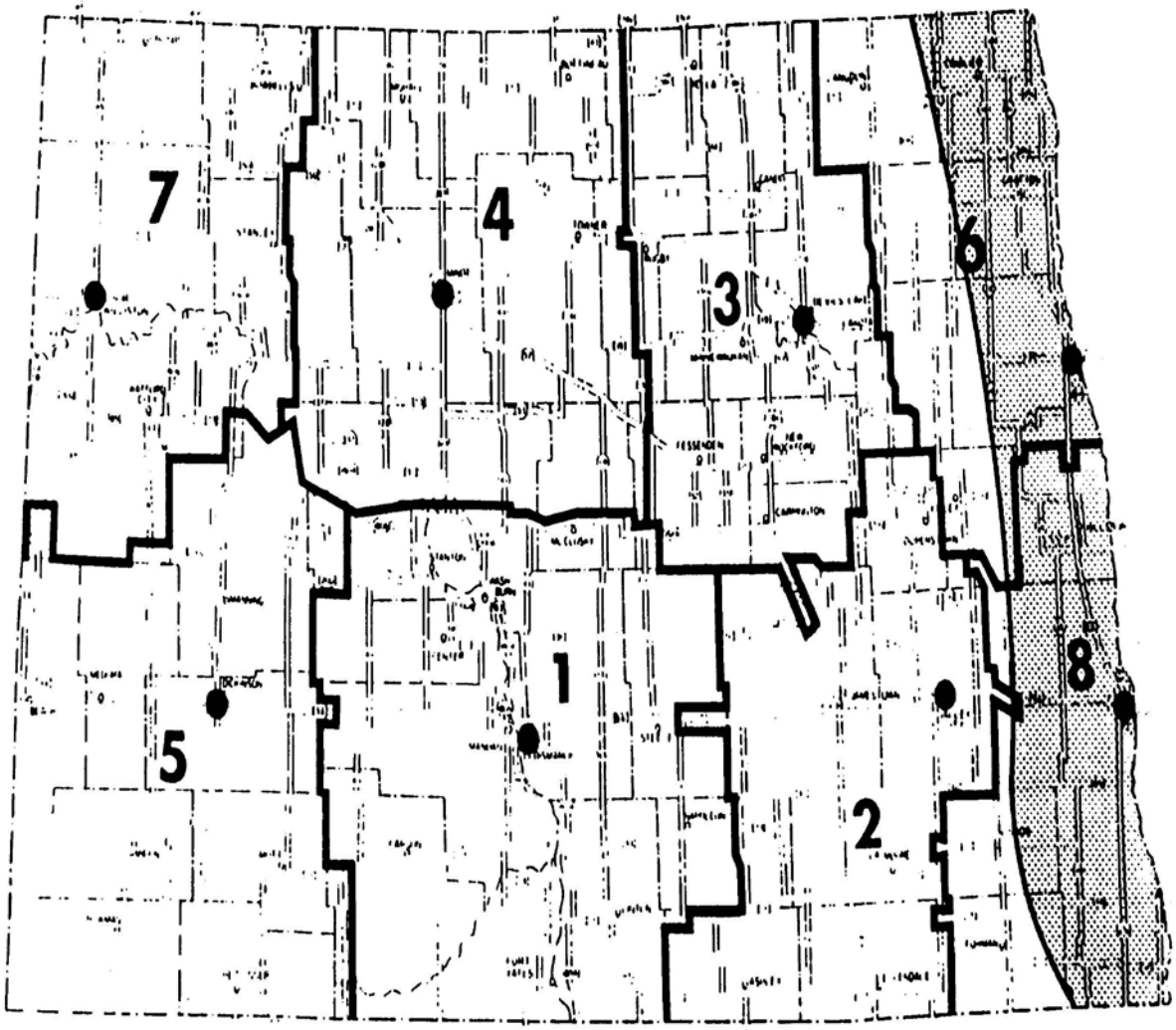


Figure 9. Friction factors for piles in granular soil; friction angle δ for sand on steel=18.5° (for piles with no appreciable taper).

assumed δ on wood=16°

NOTE: Charts E thru F taken from Michigan State Highway Department "Recommendations For Pile Design And Driving Practices."

North Dakota Department of Transportation



-Lake Agassiz Basin

District Boundaries

Appendix IV-06 B Installation of Utilities on Highway Structures

1. General Features

   a. Attachments of utility facilities to bridge structures should be avoided where it is reasonable to locate them elsewhere. However, where other locations prove to be difficult and unreasonably costly, attachment to a bridge structure will be considered, provided the attachment can be made without materially affecting the structure, the safety of traffic, the efficiency of maintenance of the structure, the efficiency of bridge inspections, its appearance, and provided the structure can support the additional load.

   b. Generally, utility installations must be attached to the bridge structure beneath the structure's floor, between the outer girders or beams or within a cell, and at an elevation above low superstructure steel or masonry.

   c. The location of utility facilities on a structure which will interfere with access to parts of the structure for painting or repair is prohibited. Manholes for utility access will not be permitted in the bridge deck.

   d. The utility installation on the bridge must be mounted so as not to reduce the vertical clearance above a river, stream, pavement, or top of rails. Utility attachments to the outside of bridges will not be permitted unless there is no reasonable alternative.

   e. Utility facilities must be firmly attached to the bridge structure and padded, where necessary, to eliminate noise and abrasion due to vibrations.

   f. Installation of utility facilities through the abutment or wingwall of an existing bridge is prohibited.

   g. In locations where a utility facility attached to a structure is carried beyond the back of the bridge abutment, the facility must curve or angle out to its proper alignment outside the roadbed area as quickly as is practical.

   h. Utility facilities may be attached to structures by hangers or roller assemblies suspended either from inserts in the underside of the bridge floor or from hanger rods clamped to a flange of a superstructure member. Bolting through the bridge floor or concrete beams is prohibited. Welding of attachments to steel members, or bolting through such members is prohibited. Where there is transverse bridge steel extending sufficiently from the underside of the bridge floor to provide adequate clearance, utility facilities may be installed on rollers or neoprene-padded saddles mounted atop such transverse members.

   i. The design of a utility facility attached to a highway structure must include satisfactory provisions for lineal expansion and contraction due to temperature changes. Line bends or expansion couplings may be used for this purpose. Materials used for attaching a utility facility to the structure must be compatible with the structural material to eliminate the possibility of corrosion.

j. A utility facility and associated appurtenances attached to a highway structure must be painted when requested by the Department. The type and color of the paint will be approved by the district engineer.

k. Each proposed bridge attachment will be considered on its individual merits.

2. New Bridge Structures

a. Where the Department plans to construct a new bridge structure, the design of the structure will, upon request of a utility company, be reviewed for accommodation of existing or proposed utility installations consistent with the requirements set forth herein. The utility company may be required to reimburse the state for any additional costs associated with accommodation of the utility facility on the new structure.

b. Installation of a facility by a utility company on a new structure must be coordinated with the bridge construction so as not to interfere with the operations of the highway contractor.

3. Pipelines

a. Pipelines, except those requiring cathodic protection and those carrying natural gas, must be encased throughout the bridge and the casing must be carried beyond the back of the bridge abutment, and effectively opened or vented at each end. The casing pipe must be designed to withstand the same internal pressure as the carrier pipe. Pipeline with extra wall thickness may be permitted, in lieu of casing, by the Design Engineer if designed by the specifications approved by the U.S. Department of Transportation's Hazardous Material Regulation Board.

b. The carrier pipe must be pressure tested before start-up in accordance with the latest edition of applicable industry codes, or appropriate regulations of an agency of the federal government.

c. Emergency shut-off valves must be installed on all pipeline attachments to a highway structure where such pipeline carries gas, liquid petroleum, or other hazardous materials under pressure. The shut-off valves should preferably be of automatic design and placed within an effective distance on each side of the structure, unless the pipeline is equipped with nearby shut-off valves or operates under effective control of automatic devices.

d. Pipelines carrying liquids subject to freezing must be protected to prevent the liquids from freezing.

4. Power and Communication Lines

a. Electric power and communication lines attached to a highway structure must be insulated from the structure, and carried in protective conduit or pipe throughout the bridge and to underground locations at each end of the structure. Exposed metallic conduit carrying electrical cables must be grounded separately from the structure.

b. Attachments for electric power and communication lines must provide sufficient clearance for convenience and safety during maintenance and repair of bridge structure or other utility installations on the bridge.

Appendix IV-06 C Pedestrian & Shared-Use Facilities on Structures

Highway Bridges that have a Pedestrian or Shared-use Path and Traffic Underneath

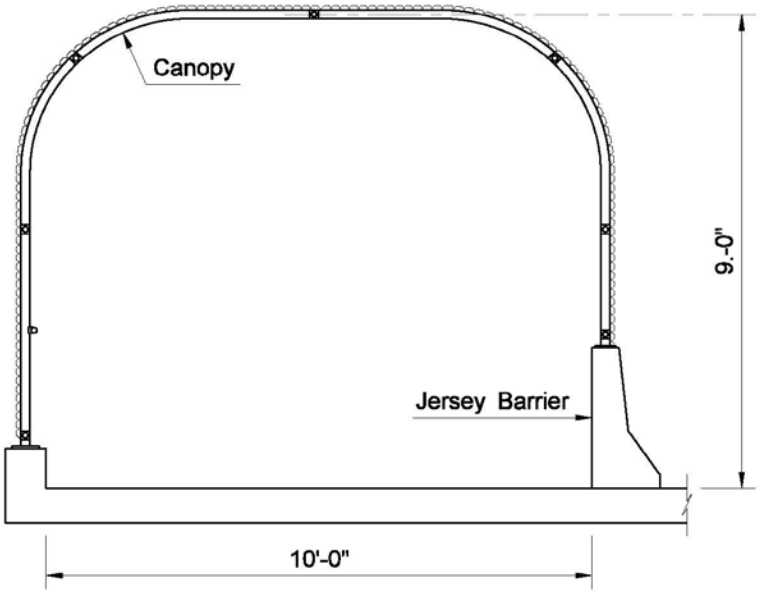


Figure 1A

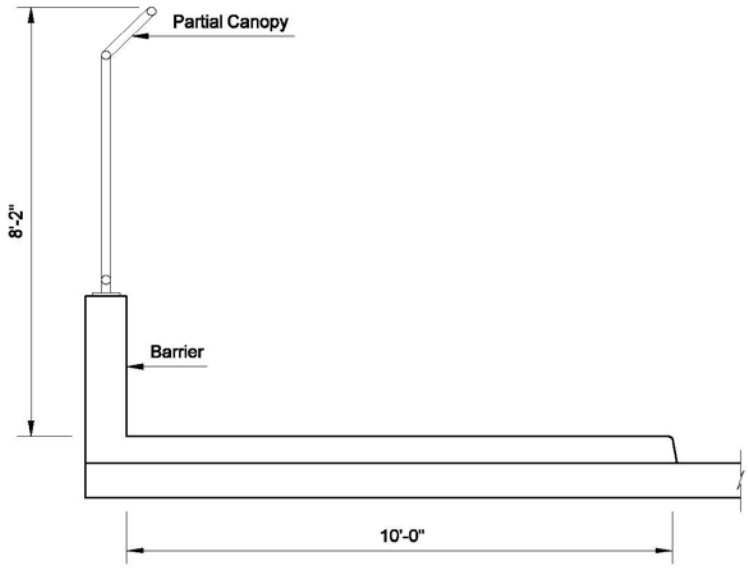
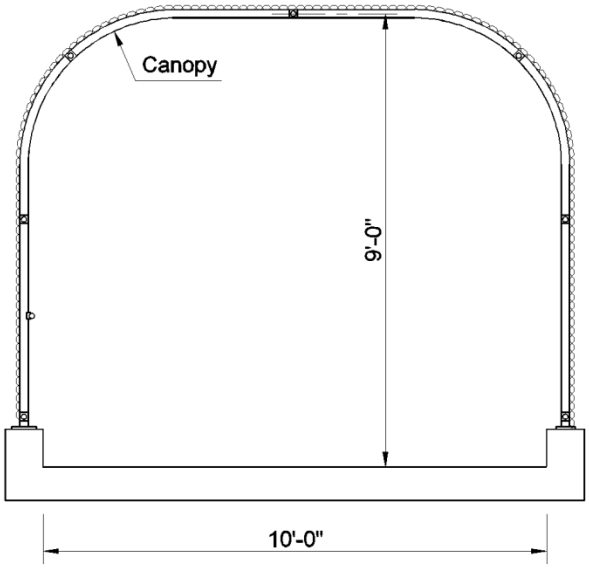
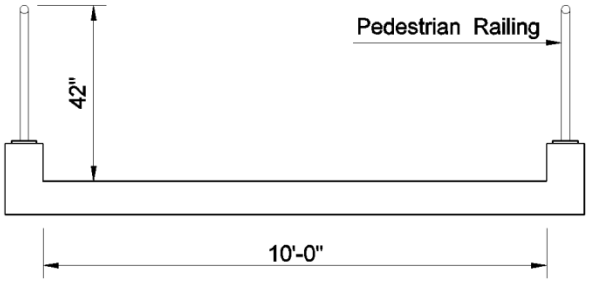


Figure 1B



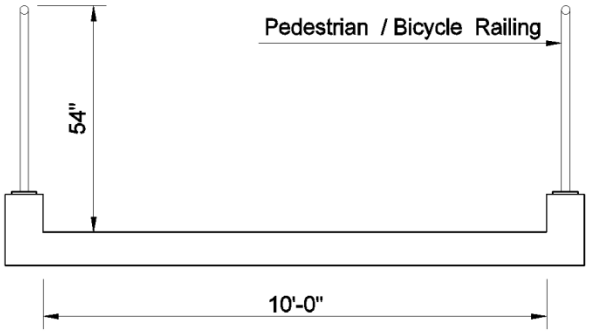
Any Pedestrian or Shared-use Bridge that has Traffic Underneath

Figure 2A



Pedestrian Bridge that has No Traffic Underneath

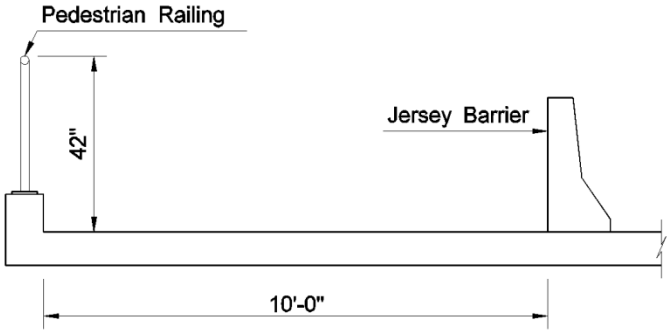
Figure 2B



Shared-use Bridge that has No Traffic Underneath

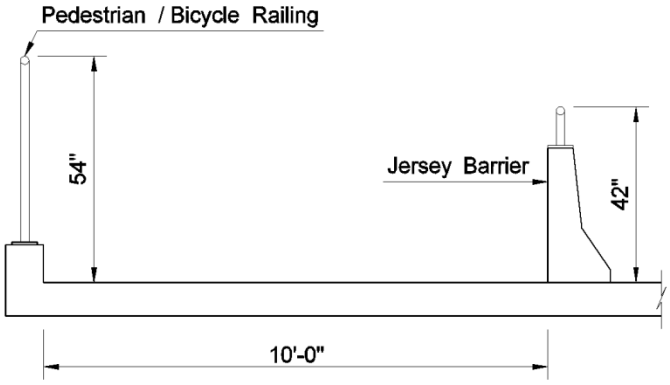
Figure 2C

Highway Bridges (High or Low Speed) with a Pedestrian or Shared-use Path with No Traffic Underneath



Pedestrian Sidewalk

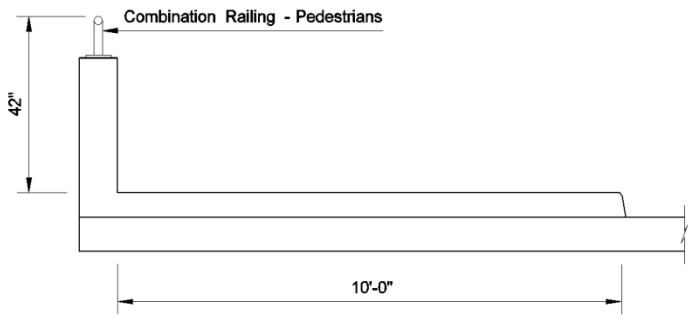
Figure 3A



Shared-use Path

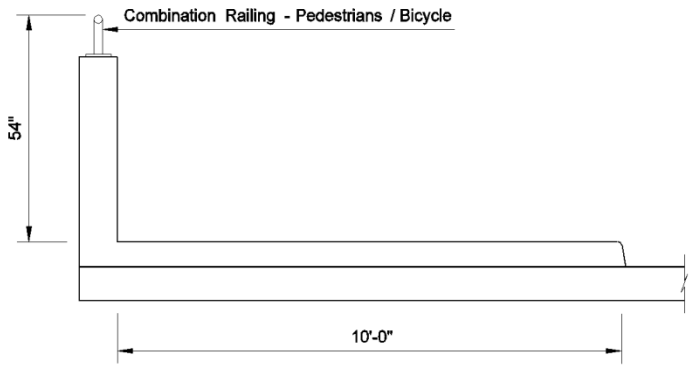
Figure 3B

Low Speed Highway Bridges with a Pedestrian Sidewalk or Shared-use Path and No Traffic Underneath



Pedestrian Sidewalk

Figure 4A



Multiuse Sidewalk

Figure 4B

Appendix IV-06 D Preliminary Engineering Meeting Agenda

Project:
Bridge Number:
Bridge Name:

A. Bridge Length:

B. Number of Spans:

C. Clear Roadway Requirements:

D. Beam/Girder Type & Spacing:

E. Embankment Endslope:

F. Abutment Type:

G. Pier Type:

H. Slope Protection:

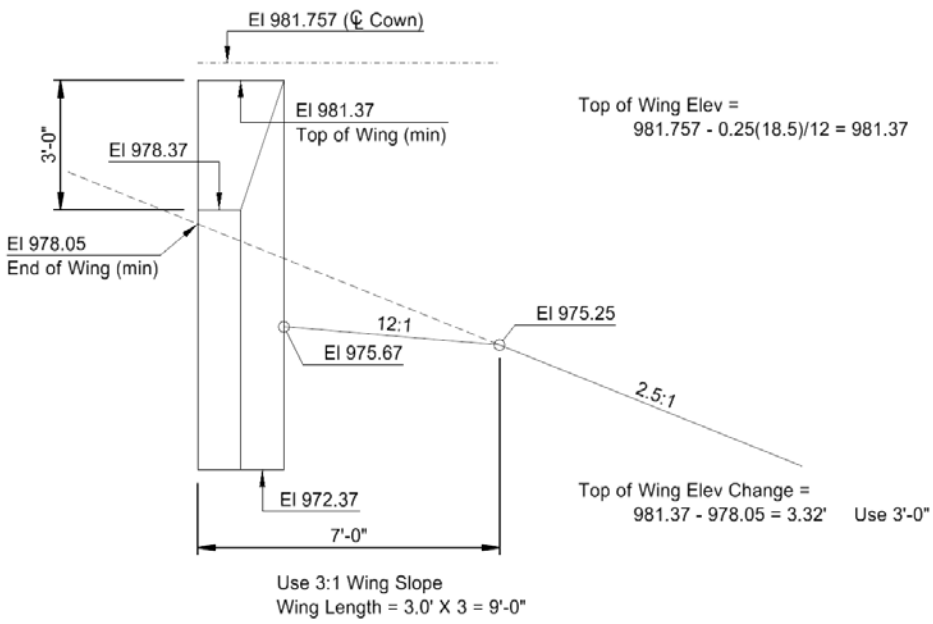
I. Approach Slabs:

J. Aesthetics:

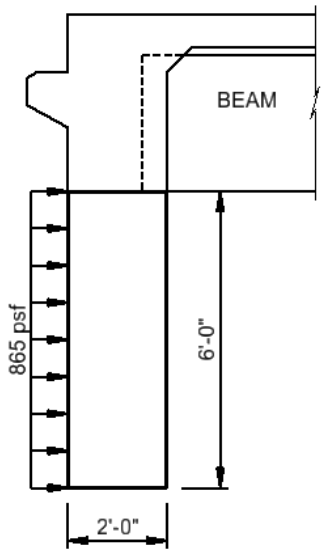
K. Other Criteria:

Appendix IV-06 E Integral Straight Abutment Design (LRFD Specifications)

WING DIMENSIONS



WALL DESIGN (VERTICAL REINFORCING)



Design as a cantilever about bottom of beam.

$$\text{Design Moment, M} = \frac{865\frac{\text{lbs}}{\text{ft}^2}(6.0\text{ft})^2}{2}\left(\frac{1}{1000\frac{\text{lbs}}{\text{kip}}}\right) = 15.57\frac{\text{kip}-\text{ft}}{\text{ft}}$$

Load & Resistance Factor Design

$$EH = 1.50 \qquad \text{Bridge Division Policy (Table 3.4.1-2)}$$

$$M_u = 1.50 \times 15.57\frac{kip-ft}{ft} = 23.4\frac{kip-ft}{ft}$$

Check capacity of #5's @ 12" spacing (min reinforcing).

LRFD 5.7.3.2 $\qquad f'_c = 3ksi \qquad f_y = 60ksi \qquad b = 12in$

$$d_s = 24in - 2in - \frac{0.625in}{2} = 21.69in$$

$$A_s = 0.31in^2$$

$$a = \frac{0.31in^2\ (60\frac{kip}{in^2})}{0.85\left(3\frac{kip}{in^2}\right)12in} = 0.61in$$

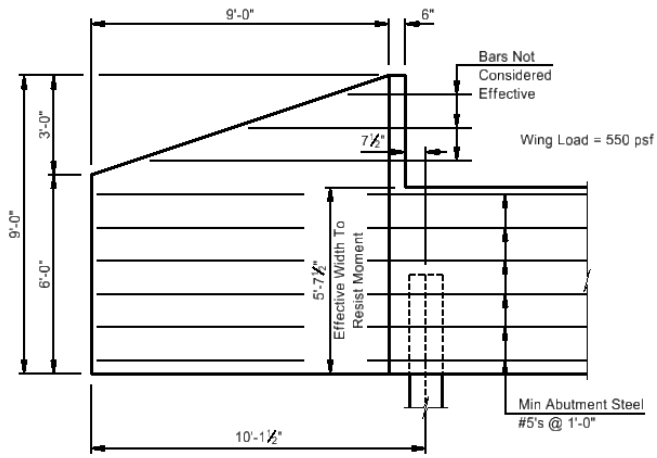
$$\varphi M_n = \varphi\ A_S\ f_y\left(d_s - \frac{a}{2}\right) = 0.9\left(0.31\frac{in^2}{ft}\right)60\frac{kip}{in^2}\left(21.69in - \frac{0.61in}{2}\right) = 357.985\frac{kip-in}{ft}$$
$$= 29.8\frac{kip-ft}{ft}$$

$$M_u < \varphi M_n \ \therefore \text{Use } \#5's \text{ @ } 12" \text{ spacing for vertical reinforcing}$$

Solve for maximum cantilever length (L) using #5's @ 12".

$$\varphi M_n = \frac{(1.50)\ 0.865\frac{kip}{ft^2}\ (L^2)}{2} = 29.8\frac{kip-ft}{ft} \rightarrow L = 6.78ft$$

WING DESIGN (HORIZONTAL REINFORCING)



When positioning pile in the abutment try to locate the outermost piles under the edge of the slab. The wing moments are calculated assuming the wing is cantilevering from the pile.

$$\text{Bending Moment, M}$$
$$= 550\frac{\text{lbs}}{\text{ft}^2}\,(5.625\text{ft})\,\frac{(10.125\text{ft})^2}{2} + 550\frac{\text{lbs}}{\text{ft}^2}\,(0.375\text{ft})9.5\text{ft}\,(5.375\text{ft})$$
$$+ 550\frac{\text{lbs}}{\text{ft}^2}\,(3.0\text{ft})\,\frac{1}{2}\,(9.0\text{ft})\,4.125\text{ft} + 550\frac{\text{lbs}}{\text{ft}^2}\,(3.0\text{ft})\,0.5\text{ft}\,(0.875\text{ft})$$
$$= 200{,}460\text{ft} - \text{lbs}$$

$$M_u = 1.50 \times 200{,}460\text{ft} - \text{lbs}\left(\frac{1}{1000\frac{lbs}{kip}}\right) = 300.7\text{kip} - \text{ft}$$

$$b = 5' - 7\tfrac{1}{2}" = 67.5\text{in} \qquad d_s = 24\text{in} - 2\text{in} - 0.625\text{in} - \frac{0.625\text{in}}{2} = 21.06\text{in}$$

$$f'_c = 3\text{ksi} \qquad f_y = 60\text{ksi} \qquad \varphi = 0.9$$

Use LRFD equations (5.7.3.1.1-4, 5.7.3.2.1-1 & 5.7.3.2.2-1).

$$\frac{M_u}{\varphi} = A_s\, f_y\, d_s - \frac{A_s^2\, f_y^2}{1.7\, f'_c\, b}$$

$$\frac{A_s^2\, f_y^2}{1.7\, f'_c\, b} - A_s\, f_y\, d_s + \frac{M_u}{\varphi} = 0$$

Solve for A_s using quadratic equation.

$$x = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

$$A_s \geq \frac{\left[f_y\, d_s - \sqrt{\left(f_y\, d_s\right)^2 - \frac{4\, f_y^2\, M_u}{1.7\, f'_c\ b\, \varphi}}\right]}{2\left(\frac{f_y^2}{1.7\, f'_c\, b}\right)}$$

$$A_s \geq \frac{60\text{ksi}\,(21.06\text{in}) - \sqrt{[60\text{ksi}\,(21.06\text{in})]^2 - \frac{4\,(60\text{ksi})^2\,(300.7\ \times 12)\text{kip}-\text{in}}{1.7\,(3\text{ksi})\,67.5\text{in}\,(0.9)}}}{2\left(\frac{(60\text{ksi})^2}{1.7\,(3\text{ksi})\,67.5\text{in}}\right)}$$

$$A_s \geq 3.26\text{in}^2$$

Area of additional steel required. $3.26 - 1.86 = 1.40\text{in}^2$

Add 4 - #6 bars. $A_s = 1.76\text{in}^2$

Add 4 - #6 bars front and back face. Bars that extend from the wing to the endwall are not considered effective.

Find capacity without using added bars.

$$a = \frac{1.86\text{in}^2\,(60\frac{\text{kip}}{\text{in}^2})}{0.85\,(3\frac{\text{kip}}{\text{in}^2})\,67.5\text{in}} = 0.65\text{in}$$

$$\varphi M_n = \varphi\, A_S\, f_y\left(d_s - \frac{a}{2}\right) = 0.9\,(1.86\text{in}^2)\,60\frac{\text{kip}}{\text{in}^2}\left(21.06\text{in} - \frac{0.65\text{in}}{2}\right) = 2{,}082.623\text{kip}-\text{in}$$
$$= 173.6\text{kip}-\text{ft}$$

Solve for length from end of wing (L) where added bars are no longer needed using the average wing height.

$$\varphi M_n = \frac{(1.50)\,7.5\text{ft}\left(0.550\frac{\text{kip}}{\text{ft}^2}\right)L^2}{2} = 173.6\text{kip}-\text{ft} \rightarrow L = 7.49\text{ft}$$

Extend distance d_s = 21.06in. $7.49\text{ft} - \left(\frac{21.06}{12}\right)\text{ft} = 5.74\text{ft}$

Start added #6's 5'-8" from end of wing.

Assume zero moment at 0.2 point of pile span. For an 8'-0" pile span.

$$10.125\text{ft} - 5.6667\text{ft} + 0.2\ (8.0\text{ft}) + \left(\frac{21.06}{12}\right)\text{ft} = 7.81\text{ft}$$

Length of added #6's = 7'-10"

Appendix IV-04 F Checklist for Bridge Plans (SFN 17180)

CHECKLIST FOR BRIDGE PLANS
North Dakota Department of Transportation, Bridge
SFN 17180 (Rev. 6-2009)

Bridge Number	Project Number	Checked by	Date

Layout Sheet: (Items marked with * may appear on another sheet)

- [ ] 1. Bridge Code
- [ ] 2. Project No. Upper Right
- [ ] 3. North Arrow
- [ ] 4. Centerline Roadway Indicated
- [ ] 5. Begin and End Sta. and Elevations
- [ ] 6. Intermediate Sta. and Elevations
- [ ] 7. Roadway Width
- [ ] 8. Overall Length
- [ ] 9. Span Lengths
- [ ] 10. Fixed and Exp. Supports Indicated
- [ ] 11. Vertical Clearance
- [ ] 12. Horizontal Clearance
- [ ] 13. Structural Clearance Line
- [ ] 14. Test Piles Shown (if used)*
- [ ] 15. Boring Log Locations
- [ ] 16. Pile Loads*
- [ ] 17. Min. Pile Penetration*
- [ ] 18. Spread Footing Loads*
- [ ] 19. Bearing Plate Layout*
- [ ] 20. Berm Width
- [ ] 21. Berm Elev. (12 to 1)
- [ ] 22. Riprap - Slope Protection
- [ ] 23. Bottom of Footing Elev.
- [ ] 24. Substructure Units Numbered
- [ ] 25. Vertical Curve Data*
- [ ] 26. Benchmarks*
- [ ] 27. Screed Elevations (Each Girder)*
- [ ] 28. Original Ground Line
- [ ] 29. Piling Length Indicated
- [ ] 30. Correct Spec. Nos.*
- [ ] 31. List Special Provisions*
- [ ] 32. Enter Quantities*
- [ ] 33. Pounds of Structural Steel*
- [ ] 34. Enter Drwg. Nos.*
- [ ] 35. Design Loading (& Design Method)
- [ ] 36. Layout Titles
- [ ] 37. Project Number and Station
- [ ] 38. County
- [ ] 39. Drawing Number
- [ ] 40. Skew Indicated
- [ ] 41. Pay Quantity Limits*
- [ ] 42. Seepage Trench Shown
- [ ] 43. Approach Slab Shown
- [ ] 44. Datum Line
- [ ] 45. Bridge Cross Section*
- [ ] 46. Design Future Wearing Surface (F.W.S.)*
- [ ] 47. PE Stamp
- [ ] 48. List of Standards*
- [ ] 49. Hydraulic Data*

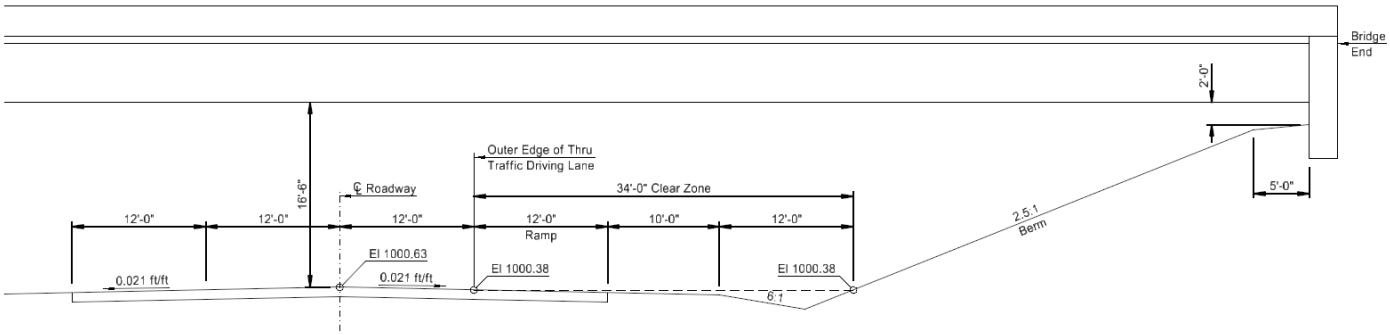
Note Sheet

- [ ] 1. Scope of Work
- [ ] 2. Miscellaneous Item Costs
- [ ] 3. Predrill thru Embankment
- [ ] 4. Embankment in Place Before Driving Pile
- [ ] 5. Reing. Steel Dimensions
- [ ] 6. Concrete Surface Finish
- [ ] 7. Cl. of Concrete and Type Coment
- [ ] 8. Approved Finishing Machine
- [ ] 9. Slope Protection
- [ ] 10. Riprap
- [ ] 11. General Pile Note
- [ ] 12. Removal of Existing Structure
- [ ] 13. Salvage and Disposal
- [ ] 14. Channel Excavation
- [ ] 15. Design Stresses (psi)
- [ ] 16. Classes of Excavation
- [ ] 17. Pile Hammer Size
- [ ] 18. Penetrating Water Repellent Trt. (optional)
- [ ] 19. Concrete Removal (Units and Quant.)
- [ ] 20. Anti-Graffiti
- [ ] 21. Bridge Approach Slabs
- [ ] 22. Deck Concrete Thickness Variations
- [ ] 23. Type of Structural Steel
- [ ] 24. Barrier Joint Spacing
- [ ] 25. Shop Drawing Requirements
- [ ] 26. Design Strengths
- [ ] 27. Design Method (Load Factor)

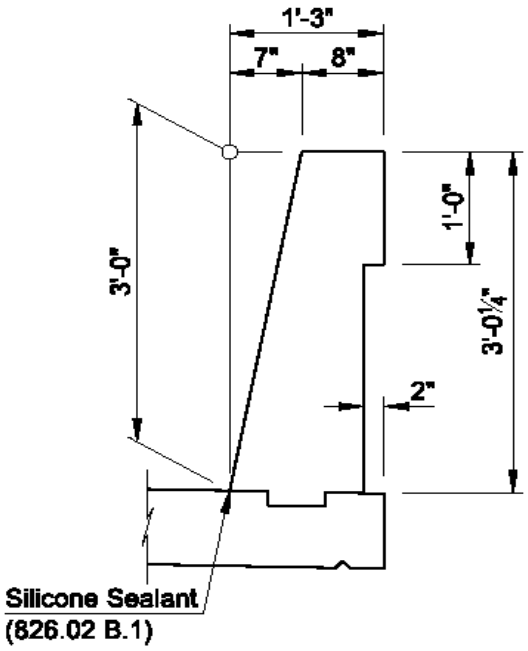
Other Detail Sheets

- [ ] 1. Detail Exp. Jt. Bevels
- [ ] 2. Field Riser Diagram
- [ ] 3. Riser Diagram Note
- [ ] 4. Blocking Diagram
- [ ] 5. Shop Camber Diagram
- [ ] 6. Concrete Placing Sequence
- [ ] 7. Field Bolt Placement
- [ ] 8. Drain Hole Location
- [ ] 9. Charpy V-Notch Test Req.

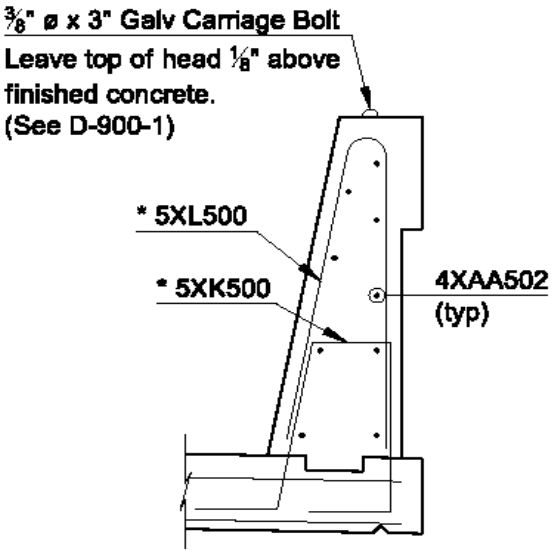
Appendix IV-06 G Interchange Clearance Diagram



Appendix IV-06 H Single Slope Barrier Details



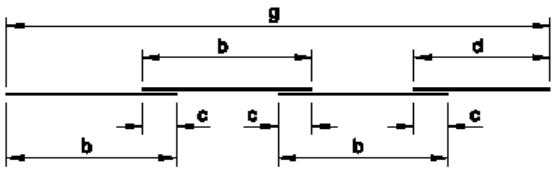
SHOWING DIMENSIONS



* Provide a 1½" clearance
to the barrier reinforcing.

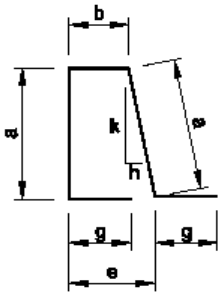
SHOWING REINFORCING

BARRIER DETAIL

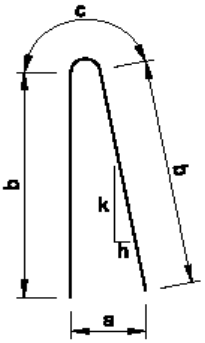


c = Lap Splice (typ)
e = # of "b" Length Pieces in a Set
Total Length per Set = e x b + d

AA



K



L

BILL OF REINFORCING STEEL, GRADE 60														
LETTER PREFIX OF BAR MARK DENOTES SHAPE ~ SEE BAR DETAILS														
LOCA-TION		SIZE	MARK	NO. EACH /SET	NOMINAL LENGTH	DETAILING DIMENSIONS								
						a	b	c	d	e	f	g	h	k
SUPERSTRUCTURE	EPOXY													
		5	XK500	0	4'-11"	1'-6"	7"			10"		8"	2.3	12
		5	XL500	0	5'-7"	9"	2'-7"	5"					2.3	12
		4	XAA500	0	0'-0"		60'-0"	2'-0"	0'-0"	0		0'-0"		

Area of Barrier = 2.5519 sf
Weight of Barrier = 383 plf
Center of Gravity of Barrier = 0.546' (from edge of deck)
The "K" bars & "L" bars have a 8" maximum spacing.
All reinforcing bars are epoxy coated.

SLAB THICKNESS	"K" VARIABLE "a"
7"	1'-5"
7½"	1'-6"
8"	1'-6"
8½"	1'-7"
9"	1'-7"
9½"	1'-8"
10"	1'-8"