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 (η_D)	1.00	Steel structures, timber bridges, ductile concrete structures
	1.05	Non ductile concrete structures
Redundancy (η_R)	1.00	Redundant
	1.05	Non Redundant
* Importance (η_I)	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-04.03.03.01 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-04.03.03.02 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-04.03.03.03 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-04.03.03.04 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-04.03.03.05 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-04.03.03.06 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-04.03.03.07 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-04.03.03.08 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-04.03.04 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-04.03.05 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-04.03.06 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-04.03.07 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-04.03.08 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-04.03.09 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-04.03.10 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-04.03.11 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-04.03.11.01 Sliding Bearings

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

IV-04.03.12 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-04.03.12.01 Vehicle Collision, CT [3.6.5]

Designers need to be concerned with vehicle collision loads. Unprotected structural elements that may be struck bluntly by a vehicle or train shall be designed to resist an equivalent static collision force of 400 kips. If unprotected, the vehicular collision force will govern the design of the piers. The preliminary plans shall be reviewed and the safety features discussed with the

roadway designers to ensure that the piers will be protected before neglecting this load. In most cases, designers will be able to consider the piers protected with safety features (barriers, etc.).

A large number of safety barriers were tested as part of NCHRP Report 350. The performance of the barriers was classified with different test levels ranging from TL-1 to TL-6.

Decks supporting safety barriers designed to contain errant vehicles on bridges shall be designed for collision forces consistent with roadway standards. In most cases, the minimum standard for safety barriers in North Dakota is Test Level 4 (TL-4). Under certain circumstances, (timber railings on secondary roads being an example) barriers may satisfy reduced test level requirements.

IV-04.03.12.02 Vessel Collision, CV [3.14]

The vessel collision force does not apply to North Dakota rivers.

IV-04.03.13 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-04.04.01.01 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-04.04.01.02 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-04.04.01.02.01 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-04.04.01.02.02 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.

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

IV-04.04.01.03 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-04.04.01.04 Pedestrian Bridge Live Load

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

IV-04.04.02 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-04.04.02.01 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

Prestressed losses shall be calculated using the refined method in AASHTO LRFD specifications published prior to the 2005 Interim.

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. For example, the TL-4 New Jersey barrier has a capacity $R_w = 95.8$ kips, which is well above the rail design collision force $F_t = 54$ kips for a Test Level 4 railing. 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 $1.1 \times F_t$.

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-04.11 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 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.

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. 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:

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

$$L = Pile\ Spacing$$

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 no 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 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. 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. 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 [3.6.5, AREMA 2.1.5.1 and C-2.1.5.1]

Piers Adjacent to Railways
Piers located within 50' of the centerline of railroad tracks are required to have crash walls incorporated into their design unless they are protected as specified in LRFD 3.6.5.1.

Piers located within 25' of the centerline of railroad tracks must either be of "heavy construction" or have crash walls.

A pier is considered to be of "heavy construction" when it meets all of the following:

- The cross-sectional area of each column is a minimum of 30 SF.
- Each column has a minimum dimension of 2.5'.
- The larger dimension of all columns is parallel to the railroad track.

Crash walls must meet the following geometric requirements:

- Top of crash wall shall extend a minimum of:
  - 6' above top of railroad track when pier is between 12' and 25' from centerline of tracks.
  - 12' above top of railroad track when pier is 12' or less from centerline of tracks.
- Bottom of crash wall shall extend a minimum of 4' below ground line.
- Crash wall shall extend one foot beyond outermost columns and be supported on footing.
- Face of crash wall shall be located a minimum of 6" outside the face of pier column or wall on railroad side of pier.
- Minimum width of crash wall is 2.5'.
- Minimum length of crash wall is 12'.

Piers of "heavy construction" and crash walls adjacent to railroad tracks shall be designed for a minimum load of 400 kips.

Piers Adjacent to Roadways
Piers located within 30' of the edge of roadway are required to have crash walls incorporated into their design unless they are protected as specified in LRFD 3.6.5.1. The crash walls adjacent to roadways shall be designed for a minimum load of 400 kips. It will be assumed that the soil will provide the required resistance. The crash wall shall be a minimum height of 4' above the ground.

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-04.11.05.01 Cantilever Retaining Walls

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

IV-04.11.05.02 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-04.11.05.03 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-04.11.05.04 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-04.11.05.05 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 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.

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]

New Jersey barrier meets railing Test Level 4 (TL-4, F_t = 54 kips).

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-04.14.03.01 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-04.14.03.02 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-04.14.03.02.01 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-04.14.03.02.02 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-04.14.03.02.03 Expansion Bearings

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

IV-04.14.03.03 Pot 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.