3. LOADS AND LOAD FACTORS

The loads section of the AASHTO LRFD Specifications is greatly expanded over that found in the Standard Specifications. This section will present applicable loads and provide guidance to MnDOT's practice for the application of these loads.

3.1 Load Factors and Combinations [3.4.1]

The standard load combinations for LRFD design are presented in LRFD Table 3.4.1-1.

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 a culvert, the maximum load factor is used on the 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 a 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, do not use more than one load factor for any load component. For example, when checking uplift, a load factor of 0.90 or 1.25 should be used for the dead load on all spans. Designers should not try to use 0.9 on the span adjacent to the uplift point and 1.25 on the next span.

Designers must ensure that structures have been checked for adequacy in carrying all appropriate load combinations at all construction stages. For example, check a high parapet abutment 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 (a case which maximizes vertical load without horizontal earth pressure load). Designers are to investigate both cases. For complex structures, designers are responsible for providing one workable construction sequence in the bridge plan and checking for adequacy at all the construction stages. If the contractor proposes a different construction sequence, the contractor is responsible for confirming structure adequacy at all the construction stages.

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. The Extreme Event I load combinations will rarely control in Minnesota. Note that designs must also consider the load combinations for construction loading.

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. MnDOT does not typically use special vehicles for design. See Article 3.4 for more information.

Strength III: Load combination used to determine flexural and shear demands that include a design wind based on a 3-second gust wind speed of 115 mph.

Strength IV: Load combination relating to very high dead load to live load force effect ratios. Use the following modified Strength IV load combination, given in AASHTO LRFD Article C3.4.1:

[C3.4.1] $1.4DC + 1.5DW + 1.45LL$

Note that Strength IV only applies to superstructures. It does not apply to investigation of construction stages, substructures, retaining walls, or bearings.

Strength V: Load combination corresponding to normal vehicular use of the bridge concurrent with a design wind based on a 3-second gust wind speed of 80 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 design, deck overhang design, and pier design per the pier protection policy found elsewhere in this manual.

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 outer fiber tension stresses and web principal stresses in prestressed concrete structures.

Fatigue I: Load combination used for the design of structures subject to repetitive live load. It is used for checking infinite load-induced fatigue life.

Fatigue II: Load combination used for the design of structures subject to repetitive live load. It is used for checking finite load-induced fatigue life.

[3.4.2]   **Construction**: All appropriate load combinations must be considered by designers for construction loads. Use the load factors given in AASHTO LRFD Article 3.4.2 for construction loads.

3.2 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 MnDOT's policy for load modifiers.

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. Load modifiers need not be applied to construction load cases.

Table 3.2.1
Standard MnDOT 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	$500 \leq ADT \leq 40,000$
	1.05	Major river crossing or ADT > 40,000 on bridge or Mainline interstate on bridge

 * Beam type superstructures with 4 or more beams per span are considered redundant

 ** Use Importance load modifier for design of the superstructure only, except do not apply to deck designs for deck-on-girder type bridges. Use only on new bridges.

3.3 Permanent
Loads (Dead and
Earth) [3.5]

To reduce the number of load factors considered through the design process, use a value of 0.020 ksf for the future wearing surface load and combine with the other component dead loads (DC loads). Also, combine the load due to a concrete wearing course with other DC loads. Apply utility loads as DW loads with the appropriate AASHTO load factor.

Table 3.3.1 lists unit weights for a number of materials. Designers should note that several of these items differ slightly from the values contained in Section 3 of the LRFD Specifications.

Table 3.3.1
MnDOT Standard Unit Weights

Material	Unit Weight (kcf)
Bituminous Wearing Course	0.150
Cast-In-Place Concrete	0.150
Precast Concrete	0.155
Precast Box Culvert	0.150
Compacted Fill on Box Culverts	0.120
Standard Fill	0.120
Steel	0.490
Timber	0.050
Water	0.0624

3.4 Live Loads
[3.6]

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

For pedestrian bridges, in addition to the pedestrian live load, design for a maintenance vehicle live load equivalent to an H-5 truck for deck widths from 6 to 10 feet, and an H-10 truck for wider decks. Use of the dynamic load allowance is not required with the maintenance vehicle.

Where appropriate, additional live loads should be considered. Additional live loads might include:
- MnDOT bridge inspection vehicle loads on bridges with large overhangs.
- MnDOT standard permit trucks on complex bridge types such as curved steel or post-tensioned concrete boxes. Discuss with the Bridge Ratings Engineer.
- Incorporate a live load surcharge into the design when construction or maintenance equipment will operate adjacent to retaining walls and abutments.

3.4.1 HL-93 Live
Load, LL
[3.6.1.2]

Use the design truck, fatigue truck, design tandem, truck train and lane loads described in the LRFD Specifications.

For simple spans, Tables 3.4.1.1 and 3.4.1.2 at the end of this section list the unfactored moments and shears for HL-93 loading on span lengths between 1 and 200 feet.

For continuous beam spans, internal studies have led to MnDOT modifications to the double truck live load given in LRFD Article 3.6.1.3.1. The modifications ensure adequate load ratings for the MnDOT standard permit trucks. In lieu of 90% of the HL-93 double truck stated in the LRFD Specifications, use the following live load for determining negative moments and interior pier reactions:

- For bridges with longest span ≤ 60 feet, apply 125% of the HL-93 double truck with dynamic load allowance plus lane load.
- For bridges with longest span > 60 feet, apply 110% of the HL-93 double truck with dynamic load allowance plus lane load.
- Do not use the double tandem loading described in LRFD Article C3.6.1.3.1.

Note that these modifications apply to continuous beam spans only. For simple spans, follow LRFD Article 3.6.1.3.1 as written for determination of interior pier reactions.

3.4.2 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. Note that the LRFD Specifications require a 1.2 factor to be used for the design of structures carrying a single lane of traffic.

3.4.3 Dynamic Load Allowance, IM [3.6.2]

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 should note that the base values are reduced for buried components and for wood structures.

3.4.4 Pedestrian Live Load, PL [3.6.1.6]

Pedestrian live loads vary with the function of the bridge. For conventional highway bridges with sidewalks wider than two feet, use an intensity of 0.075 ksf.

For pedestrian bridges, refer to the *Guide Specifications for Design of Pedestrian Bridges* for the pedestrian live load to be used.

3.4.5 Braking Force, BR [3.6.4] [3.6.1.1.1]

Use judgment when applying braking forces to a structure. For one-way bridges, apply the braking force in all AASHTO defined design lanes. For bridges striped as two-lane, two-way bridges, apply the braking force in one direction in both traffic lanes. For two-way bridges with more than two striped traffic lanes, determine the traffic direction with the greatest

width (including width of any sidewalks and pedestrian trails adjacent to traffic) and apply the braking force to the number of AASHTO defined design lanes that fit within that width.

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

For pier design, braking forces are to be applied at a height 6 feet above the roadway surface and in a longitudinal direction. In bridges where there is not a moment connection between the superstructure and substructure (i.e., beam bridges on bearings), the braking force can be assumed to be applied to the pier at the bearings.

3.4.6 Centrifugal Force, CE [3.6.3]

Similar to braking forces, multiple presence factors are to be applied to the centrifugal force, while the dynamic load allowance is not applied.

Apply the centrifugal force at a height of 6 feet above the top of the deck.

3.4.7 Live Load Application to Buried Structures

For buried structures, a lane plus a design truck or tandem is applied to the roadway and distributed through the fill. If the fill is 2 feet or less, the live load is applied as a footprint to the top of the structure. For fills over 2 feet, the footprint load spreads out through the soil fill. Refer to Article 12.2.3 of this manual for more information on application of live load to box culverts.

3.4.8 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.

3.5 Water Loads, WA [3.7]

Some of the hydraulic event terminology used in the MnDOT hydraulic report differs from that used in the AASHTO LRFD Specifications (LRFD):

- The MnDOT "design flood" for a structure is based on the average daily traffic that passes over the structure with the maximum design flood being a 50-year flood. (Refer to Section 3.2 of the MnDOT Drainage Manual for more information.) This is used as part of a roadway and surrounding property risk assessment done by the Hydraulics Section.

[2.6.4.4.2 and 3.7.5]

- The LRFD "design flood" for a structure is the lesser of the overtopping or 100-year flood. Use the LRFD "design flood" water and scour elevations (the 100-year flood is called out as the "basic flood" in the MnDOT hydraulic report) when analyzing piers for stream pressure loads under the strength and service limit states.

- The "check flood for scour", as defined by LRFD, is the lesser of the overtopping or 500-year flood. Use the LRFD "check flood for scour" water and scour elevations when analyzing piers for stream pressure loads under the extreme event limit state as follows:

  o Check piers using Extreme Event II for the full "check flood for scour" water and scour elevations. Do not include any BL, IC, CT, or CV loads for this check.
  o Check piers using Extreme Event II for applicable BL, IC, CT, or CV loads. For this case, use 50% of the water and scour from the "check flood for scour".

Design structural elements for both the no scour condition and the anticipated scour condition.

3.6 Wind Loads [3.8]

Wind loads are based on the design 3-second gust wind speeds given in LRFD Table 3.8.1.1.2-1. Use a design 3-second gust wind speed of 115 mph for the Strength III limit state.

3.6.1 Wind Load on Structure, WS [3.8.1.2 & 3.8.2]

For design of substructures, use the following guidance regarding wind loads applied to ornamental metal railing or chain link fence:
- For Standard Figures 5-397.160 and .161, Ornamental Metal Railing with Fence (Design T-3), assume 50% of the combined rail/fence surface area is solid.
- For Standard Figures 5-397.162 and .163, Ornamental Metal Railing (Design T-4), assume 30% of the rail area is solid.
- Calculate the rail surface area for other standard and non-standard ornamental metal rails.
- For chain link fence, assume 30% of the fence area is solid.
- When determining the moment arm for pier design due to wind acting on the superstructure, assume the wind pressure acts on the full height of the ornamental metal rail or chain link fence.

Do not use these loads for ornamental metal railing or chain link fence design. Refer to LRFD Section 13 for railing design.

The vertical overturning wind load described in LRFD Article 3.8.2 must also be considered in design.

3.6.2 Wind on Live Load, WL
[3.8.1.3]

Consider the force effects of wind on live load for the Strength V and the Service I load combinations.

Apply the wind on live load forces at a height 6 feet above the top of the deck. In bridges where there is not a moment connection between the superstructure and substructure (i.e., beam bridges on bearings), the longitudinal component of the wind on live load force can be assumed to be applied to the pier at the bearings.

3.7 Earthquake Effects, EQ
[3.10]

All of Minnesota is in Seismic Zone 1 with acceleration coefficients varying between 2 and 3 percent. With very small acceleration coefficients, earthquake forces will rarely govern the design of MnDOT structures. However, Seismic Zone 1 structures must satisfy AASHTO requirements pertaining to the length of superstructure bearing seats and the horizontal design connection force between the superstructure and substructure.

[4.7.4.4]

For expansion bearings, check that the actual length of bearing seat, N_{act}, satisfies LRFD Article 4.7.4.4 using a Percentage N equal to 75.

[3.10.9.2]

For fixed bearings and anchors, MnDOT has modified the required horizontal connection force given in AASHTO. Design for a minimum horizontal connection force equal to 15% of the Strength I limit state vertical reaction.

3.8 Ice Loads, IC
[3.9]

The design ice load is 1.5 feet of ice with a crushing strength of 32.0 ksf. Assume the ice load is applied at a height two-thirds of the distance from the flowline elevation to the lesser of the 100-year flood or overtopping flood high water elevation. Use a friction angle θ_f equal to 0 degrees between the ice and pier nose.

3.9 Earth Pressure, EV, EH or ES
[3.5.1, 3.5.2]
[3.11.5, 3.11.6]

For cast-in-place cantilever concrete retaining walls, refer to the "Basis of Design" found on standard plan sheet 5-297.639 for determination of earth pressure loads. For other types of retaining walls, follow the current *AASHTO LRFD Bridge Design Specifications*.

For applications with level backfill other than retaining walls, simplified equivalent fluid methods can be used for determination of lateral earth

pressure loads (EH).   For parapet and semi-integral abutment stems, design for an active earth pressure of 0.033 kcf equivalent fluid weight. For level backfill applications where at-rest earth pressures cannot be relieved, design for an equivalent fluid weight of 0.060 kcf. Assume that the horizontal resultant for lateral earth pressures acts at a height of H/3.

For integral abutments and semi-integral abutment diaphragms, design for passive earth pressure loads. See Article 11.1.1 of this manual for load application.

For the vertical earth loads (EV) applied to pier footings, use a maximum load factor of 1.35 and a minimum load factor of 0.90.

3.10 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.

As an alternative to AASHTO, the *CEB-FIP Model Code for Concrete Structures, 1990*, may be used to determine time dependent effects of concrete in post-tensioned structures.

3.10.1 Temperature Effects

One of the most ambiguous tasks for bridge designers is the determination of the appropriate temperature range and corresponding deformations for use in calculating force effects on a structure.   Past MnDOT practice has been to design concrete frames for a 45°F temperature fall and a 35°F temperature rise, a temperature range smaller than what the bridge will actually experience during its service life. This method dates back to the 1920s, and the reduced temperature range should be considered a "rule of thumb" that was applied to typical bridges using simplified analysis methods of the time.   No notable performance issues have been attributed to application of a lower thermal temperature range when applied to pier frames or relatively short span bridges. On complicated, longer span bridge frames, longitudinal thermal effects become a larger issue that designers should not ignore. Therefore, the following policy is to be used for application of thermal loads on typical and non-typical bridges.

Typical Bridges
Typical bridges include:
- routine multiple span prestressed beam, steel beam, and slab bridges

- bridges with two or fewer fixed piers
- bridges with piers less than 30 feet tall

[3.12.2.1]   For typical bridges, use LRFD Procedure A for internal pier frame forces due to thermal expansion. For concrete frames, Procedure A allows for a temperature range of 80°F. Use a base construction temperature of 45°F, which corresponds to designing for thermal force effects due to a 45°F temperature fall and a 35°F temperature rise. In addition, apply the strength limit state load factor of 0.5 for calculation of thermal force effects and use gross section properties in the analysis. The 0.5 load factor accounts for the reduction in thermal forces due to cracking of the concrete.

[3.12.2.2]   For longitudinal effects, use a temperature range of 150°F (-30°F to 120°F), which is the approximate range given by LRFD Procedure B for Minnesota's climate. Use a base construction temperature of 45°F and apply the strength limit state load factor of 0.5 for calculation of thermal force effects while using gross section properties in the analysis. Also, see Article 14.1 of this manual for guidance on fixity and thermal movements.

Design expansion joint openings for movements associated with a temperature range of 150°F (-30°F to 120°F). For strip seal expansion joints, use a load factor for movement of 1.0. (Note that this value differs from the LRFD Specifications based on past performance of joints in Minnesota.) For modular expansion joints, use a load factor for movement of 1.2 per LRFD Article 3.4.1. See Article 14.2 of this manual for more guidance on expansion joints.

Design bearings for movements associated with a temperature range of 150°F (-30°F to 120°F) and a base construction temperature of 45°F. For computation of movement for the elastomeric pad minimum compressive stress check, use a load factor of 1.0. For computation of movement to determine minimum elastomer thickness, use a load factor of 1.3. (Note that these load factors differ from the LRFD Specifications and are based on past performance of elastomeric bearings in Minnesota.) For computation of movement for design of pot and disc bearings, use a load factor of 1.2.

Non-Typical Bridges
Non-typical bridges are those with tall or slender piers or those with long spans. For these bridges, the pier stiffness is critical in determining movements and forces, and a refined analysis must be used to reduce force effects due to thermal movements and other loads.

[3.12.2.2]

For non-typical bridges, use a temperature range of 150°F (-30°F to 120°F) for longitudinal effects, which is the approximate range given by LRFD Procedure B for Minnesota's climate. When analyzing bridges with this larger thermal range, the designer must consider the following in the analysis:

- Pier stiffness – Use refined method to determine the appropriate percentage of gross stiffness along the height of the pier.
- Bearing fixity and flexibility – Account for the stiffness of expansion bearings in determination of the overall bridge movements.
- Construction method, staging, temperature range at erection, and its effect on the connectivity of the structural system.
- Foundation stiffness – Elastic shortening of the piles provides a significant relaxation to forces applied to the pier. Also, horizontal displacements of piling will provide moment reduction.
- For joint and bearing sizing, use a 150°F range at Service Limit State conditions. Use a thermal movement load factor of 1.2. Also use this movement to determine horizontal force requirements for guided bearings.
- For Strength Limit State, use a thermal load factor of 1.0 with the 150°F range for longitudinal force effects. For transverse effects within individual pier frames, an 80°F range with a 45°F base construction temperature may be used.

A 3-D model of the bridge with appropriate elastic restraints at supports may be required (especially for curved bridges) to determine the direction of movement, magnitude of thermal forces, and interaction between piers for determination of the appropriate cracked section reduction in stiffness. The final solution may require several iterations and may be bracketed using an upper-bound and lower-bound stiffness matrix (i.e., - gross sections, partially cracked sections, etc.) so that the final solution falls within an acceptable range for the particular structure.

In cases where several piers are fixed to the superstructure, consideration of ambient temperature at anticipated time of construction (including adjustments for closure pours as necessary) should be considered. Setting of bearings and joints within the structure may require special provisions that call for contractor submittals which state the intended method of bearing and joint installation to obtain a neutral position at the mean temperature.

Some non-typical bridges will consist of multiple units (where a unit is defined as the number of spans between expansion joints) with multiple bridge types, where not all units are non-typical. For example, a major

river crossing may consist of 3 units: a multi-span slab type approach unit, a single main span tied arch unit, and a pretensioned concrete beam approach unit. If the approach units fit the typical bridge category, a refined analysis for pier stiffness determination is not required for the approach units. However, use of a thermal movement load factor of 1.2 is still required for joint and bearing sizing in the typical units.

3.10.2 Shrinkage Effects

Use a design relative humidity to 73% for concrete shrinkage computations.

3.11 Pile Downdrag, DD

For situations where long friction piles or end-bearing piles penetrate through a soft, compressible, top layer of material, long term settlement of the soft layer may introduce a downdrag load to the pile as it grips the pile through negative skin friction. An estimate of the downdrag load will be given in the Foundation Engineer's Memo and the amount of downdrag load to consider in design will be specified in the Foundation Recommendations.   See Section 10.1.2 of this manual for more discussion on downdrag.

3.12 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 should be designed to resist the friction force required to mobilize the bearing.

3.12.1 Sliding Bearings

LRFD Table 14.7.2.5-1 provides design coefficients of friction for PTFE sliding surfaces.

3.12.2 Soil/Backwall Interface and Soil/Footing Interface

Use LRFD Table 3.11.5.3-1 to obtain the coefficients of friction between the backwall/footing and soil. When cohesionless backfill is used behind a vertical or near vertical wall, the friction between the backwall and the backfill can be ignored.

When evaluating the sliding resistance between a concrete and soil interface, a coefficient of 0.80 shall be used. For cases where a shear key is utilized, the portion of the failure plane with soil on both sides should be evaluated with a coefficient of friction of 1.00.

3.13 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 can not occur when the waterway is iced over.

For the extreme event cases with ice (IC) or vessel collision (CV), evaluate bridges for 50% of the 500 year scour event depth.

3.13.1 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 protected or be designed to resist the collision force Review the Preliminary Plans to determine what is required. Also, see Section 11.2.3 of this manual for complete pier protection policy and requirements.

There are two documents which contain crash test criteria for bridge railings and barriers. They are *NCHRP Report 350* and the more recent *Manual for Assessing Safety Hardware.* The performance of barriers is 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 on bridges carrying high speed traffic in Minnesota is Test Level 4 (TL-4). Under certain circumstances, reduced test level requirements may be acceptable. For example, TL-3 may be adequate for buried structures. See Section 13 of this manual for additional guidance.

3.13.2 Vessel Collision, CV [3.14]

Structures within reaches of the Mississippi, Minnesota, and St. Croix rivers, and Lake Superior deemed navigable by the Corps of Engineers shall be designed to resist vessel collision loads.

3.14 Uplift

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 should 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.

[Table 3.4.1-2]

When evaluating a structure for uplift the load factors for permanent load should be reviewed. Minimum and maximum factors shall be combined for different elements to generate the most conservative or largest uplift force effect.

3.15 Construction Loads

The designer must consider construction loads during design. The diaphragm spacing and top flange dimensions in the positive moment region of the steel beam superstructures are based on the construction load stage. Specialty structures such as segmental concrete bridges have unique construction loads to consider during design that are explicitly defined. Unless project specific information is available or necessary, use the following loads:

Formwork

For conventional formwork (plywood, etc.) assume a uniform dead load of 0.010 ksf. In addition to dead loads, design concrete formwork for a construction live load of 0.050 ksf.

Structural Elements

Structural elements that support formwork are assumed to have a larger tributary area and consequently are to be designed for a smaller construction live load of 0.020 ksf.

Consider reconstruction loads when designing end diaphragms. At abutments, design end diaphragms to carry vertical jacking forces during bearing replacement.

3.16 Deflections
[2.5.2.6.2]

MnDOT's maximum permitted live load deflection for highway bridges without sidewalks is $L/800$. For highway bridges with sidewalks, the limit is reduced to $L/1000$.

For typical deck-on-beam bridges that meet the LRFD Table 4.6.2.2.2b-1 and 4.6.2.2.2d-1 "Range of Applicability", use the following load distribution when computing deflections:

Live Load:
$$\text{Live Load Distribution Factor} = \text{MPF} \cdot \left[\frac{\# \text{ Lanes}}{\# \text{ Beams}}\right]$$

Dead Load:
$$\text{Dead Load (per beam)} = \left[\frac{\text{Total DC}}{\# \text{ Beams}}\right]$$

For deck-on-beam bridges that fall outside the LRFD Table 4.6.2.2.2b-1 and 4.6.2.2.2.d-1 "Range of Applicability", a 3D model may be used to determine deflections.

Table 3.4.1.1

Maximum Unfactored HL-93 Live Load Moments, Shears, and Reactions

Simple Spans, One Lane, w/o Dynamic Load Allowance or Multiple Presence Factor

Span (ft)	Moments				Shears and End Reactions		
	Truck (kip-ft)	Tandem (kip-ft)	Lane (kip-ft)	Span Pt. (%)	Truck (kip)	Tandem (kip)	Lane (kip)
1	8.0	6.3	0.1	0.50	32.0	25.0	0.3
2	16.0	12.5	0.3	0.50	32.0	25.0	0.6
3	24.0	18.8	0.7	0.50	32.0	25.0	1.0
4	32.0	25.0	1.3	0.50	32.0	25.0	1.3
5	40.0	31.3	2.0	0.50	32.0	30.0	1.6
6	48.0	37.5	2.9	0.50	32.0	33.3	1.9
7	56.0	43.8	3.9	0.50	32.0	35.7	2.2
8	64.0	50.0	5.1	0.50	32.0	37.5	2.6
9	72.0	62.5	6.5	0.50	32.0	38.9	2.9
10	80.0	75.0	8.0	0.50	32.0	40.0	3.2
11	84.5	92.0	9.3	0.40	32.0	40.9	3.5
12	92.2	104.0	11.1	0.40	32.0	41.7	3.8
13	103.0	115.9	13.4	0.45	32.0	52.3	4.2
14	110.9	128.3	15.5	0.45	32.0	52.9	4.5
15	118.8	140.6	17.8	0.45	34.1	43.3	4.8
16	126.7	153.0	20.3	0.45	36.0	43.8	5.1
17	134.6	165.4	22.9	0.45	37.6	44.1	5.4
18	142.6	177.8	25.7	0.45	39.1	44.4	5.8
19	150.5	190.1	28.6	0.45	40.4	44.7	6.1
20	158.4	202.5	31.7	0.45	41.6	45.0	6.4
21	166.3	214.9	34.9	0.45	42.7	45.2	6.7
22	174.2	227.3	38.3	0.45	43.6	45.5	7.0
23	182.2	239.6	41.9	0.45	44.5	45.7	7.4
24	190.1	252.0	45.6	0.45	45.3	45.8	7.7
25	198.0	264.4	49.5	0.45	46.1	46.0	8.0
26	210.2	276.8	53.5	0.45	46.8	46.2	8.3
27	226.1	289.1	57.7	0.45	47.4	46.3	8.6
28	241.9	301.5	62.1	0.45	48.0	46.4	9.0
29	257.8	313.9	66.6	0.45	48.8	46.6	9.3
30	273.6	326.3	71.3	0.45	49.6	46.7	9.6
31	289.4	338.6	76.1	0.45	50.3	46.8	9.9
32	307.0	351.0	81.1	0.45	51.0	46.9	10.2
33	324.9	363.4	86.2	0.45	51.6	47.0	10.6
34	332.0	375.0	92.5	0.50	52.2	47.1	10.9
35	350.0	387.5	98.0	0.50	52.8	47.1	11.2
36	368.0	400.0	103.7	0.50	53.3	47.2	11.5
37	386.0	412.5	109.5	0.50	53.8	47.3	11.8
38	404.0	425.0	115.5	0.50	54.3	47.4	12.2
39	422.0	437.5	121.7	0.50	54.8	47.4	12.5
40	440.0	450.0	128.0	0.50	55.2	47.5	12.8

Table 3.4.1.2

Maximum Unfactored HL-93 Live Load Moments, Shears, and Reactions

Simple Spans, One Lane, w/o Dynamic Load Allowance or Multiple Presence Factor

Span (ft)	Moments				Shears and End Reactions		
	Truck (kip-ft)	Tandem (kip-ft)	Lane (kip-ft)	Span Pt. (%)	Truck (kip)	Tandem (kip)	Lane (kip)
42	485.2	474.8	139.7	0.45	56.0	47.6	13.4
44	520.9	499.5	153.3	0.45	56.7	47.7	14.1
46	556.5	524.3	167.6	0.45	57.4	47.8	14.7
48	592.2	549.0	182.5	0.45	58.0	47.9	15.4
50	627.8	573.8	198.0	0.45	58.6	48.0	16.0
52	663.4	598.5	214.2	0.45	59.1	48.1	16.6
54	699.1	623.3	230.9	0.45	59.6	48.1	17.3
56	734.7	648.0	248.4	0.45	60.0	48.2	17.9
58	770.4	672.8	266.4	0.45	60.4	48.3	18.6
60	806.0	697.5	285.1	0.45	60.8	48.3	19.2
62	841.6	722.3	304.4	0.45	61.2	48.4	19.8
64	877.3	747.0	324.4	0.45	61.5	48.4	20.5
66	912.9	771.8	345.0	0.45	61.8	48.5	21.1
68	948.6	796.5	366.2	0.45	62.1	48.5	21.8
70	984.2	821.3	388.1	0.45	62.4	48.6	22.4
75	1070.0	887.5	450.0	0.50	63.0	48.7	24.0
80	1160.0	950.0	512.0	0.50	63.6	48.8	25.6
85	1250.0	1012.5	578.0	0.50	64.1	48.8	27.2
90	1340.0	1075.0	648.0	0.50	64.5	48.9	28.8
95	1430.0	1137.5	722.0	0.50	64.9	48.9	30.4
100	1520.0	1200.0	800.0	0.50	65.3	49.0	32.0
110	1700.0	1325.0	968.0	0.50	65.9	49.1	35.2
120	1880.0	1450.0	1152.0	0.50	66.4	49.2	38.4
130	2060.0	1575.0	1352.0	0.50	66.8	49.2	41.6
140	2240.0	1700.0	1568.0	0.50	67.2	49.3	44.8
150	2420.0	1825.0	1800.0	0.50	67.5	49.3	48.0
160	2600.0	1950.0	2048.0	0.50	67.8	49.4	51.2
170	2780.0	2075.0	2312.0	0.50	68.0	49.4	54.4
180	2960.0	2200.0	2592.0	0.50	68.3	49.4	57.6
190	3140.0	2325.0	2888.0	0.50	68.5	49.5	60.8
200	3320.0	2450.0	3200.0	0.50	68.6	49.5	64.0

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