SECTION 3
LOADS AND LOAD FACTORS

3.1 GENERAL REQUIREMENTS

The following section is provided as CDOT practice for loads and load factors. The Designer shall coordinate with Staff Bridge regarding project-specific circumstances warranting deviations from standard practices referenced herein.

This section is complementary to the current CDOT Bridge Rating Manual, CDOT Bridge Detailing Manual, CDOT Standard Specifications for Road and Bridge Construction, and current Bridge Structural Worksheets.

3.2 CODE REQUIREMENTS

Unless otherwise modified by this section, the minimum requirement for loads and load factors shall be in accordance with Section 3 of AASHTO. This section of the BDM is intended to supplement AASHTO code requirements. Any requests to vary from methodologies presented herein will be discussed with Staff Bridge.

3.3 CONSTRUCTION LOADING

Construction loads act on the structure only during construction and are often not accurately known at the time of design. If specific construction loads have been assumed as a part of the design, these loads shall be documented in the plans. Otherwise, the Contractor's Engineer shall determine the magnitude and applicability of construction loads and provide falsework and temporary supports as necessary to ensure the stability and constructability of the structure during construction. ***AASHTO C3.4.2.1***

Transient construction loads shall meet all legal load limits or be approved by CDOT's permit office for both new and existing structures.

3.4 DEAD LOADS

3.4.1 Stay-in-Place Metal Deck Forms

In accordance with Section 9.13.3 of this BDM, form flutes shall not be filled with concrete. A minimum of 5 psf (non-composite) shall be used to account for stay-in-place metal deck forms, when they are allowed.

3.4.2 Wearing Surface

The following unit weight shall be used in the design of CDOT structures:

Asphalt Unit Weight: 146.67 lb/ft^3

This unit weight results in 36.67 psf for 3-inch asphalt overlays. This unit weight is equivalent to the roadway standard of using 110 pounds per square yard per inch of thickness for quantities.

3.4.3 Utilities

Utility loads shall include the dead load of both the basic utility and all connections, supports, casings, and other required appurtenances.

Waterlines carried in a casing shall be evaluated at the extreme event level for the potential of waterline failure, resulting in the casing being filled with water.

An allowance of 5 pounds per square foot of composite load shall be included for new bridges within urban areas to account for future utilities. For rural bridges, the potential for future utilities should be discussed with the Local Agency and the CDOT Project Manager. Refer to Section 4.4 of this BDM for distribution of utility loads.

3.4.4 Girder Concrete

3.4.4.1 Concrete Unit Weight

The unreinforced concrete unit weight for use in calculating dead loads shall be 145 pcf per AASHTO Table 3.5.1-1.

- For reinforced CIP concrete, a minimum of 5 pcf is added to the unreinforced weight to account for reinforcing which results in the typical 150 pcf.
- For shop produced precast girders, a minimum of 5 pcf shall be added to the unreinforced weight to account for reinforcing. The unreinforced weight for load purposes shall be calculated per AASHTO Table 3.5.1-1 using the final girder strength f'c. For Class PS concrete to account for variability in actual concrete strength, a minimum unit weight of 155 pcf shall be used. This is based on 10 ksi concrete strength and 5 pcf for reinforcement.

3.4.4.2 Weight of Curved Precast U Girders

The Designer is responsible for accounting for the increased self-weight due to inside faces of webs being chorded for curved precast U girders. The Designer should confer with local suppliers concerning the inside web form geometry required for specific project parameters.

3.5 COLLISION LOAD

3.5.1 Policy

CDOT structures shall be evaluated for Collision Force (CT) as detailed in Sections 3.5.2, 3.5.3, and 3.5.4 of this BDM. In certain cases, structures may be deemed exempt from CT loads based on the criteria within the commentary of AASHTO 3.6.5.1, including Equation C3.6.5.1-1 and Table C3.6.5.1-1. Exemption from CT loads will be allowed only with Unit Leader approval in coordination with the State Bridge Engineer and should be documented in the Structure Selection Report.

3.5.2 New Bridges

The preferred strategy for new bridges is to meet the clearance and protection requirements set forth in AASHTO. Exposed supporting elements of new bridges that can be hit by errant or oversized vehicles shall be designed for a Collision Force (CT) of 600 kip. The application shall be in accordance with AASHTO.

AASHTO 3.6.5

This design criterion typically applies to pier columns and non-redundant through type superstructure elements, such as through trusses or through arches.

Columns subject to train impact shall be designed in accordance with the AREMA *Manual for Railway Engineering* and the UPRR/BNSF *Guideline for Railroad Grade Separation Projects*.

3.5.3 Existing Structures

Existing structures shall be evaluated for CT loads in accordance with AASHTO. The preferred strategy for existing structures is to meet the clearance and protection requirements. If clearance and protection are impractical, the columns shall be evaluated for a CT force of 600 kip. The application shall be in accordance with AASHTO.

AASHTO 3.6.5

The Engineer shall consider retrofitting the column system to achieve the required load capacity. The existing foundation should be evaluated, along with the column system, to ensure proper load carrying capacity.

The structure may be alternatively checked for adequate redundancy to resist collapse from the loss of the members that have inadequate strength to resist the CT load. This is done by modeling the structure without the inadequate members, with the structure subjected to a load of at least 1.0 DL and 0.5 LL+I.

3.5.4 Temporary Works

Temporary falsework towers that are within 30 ft. of through traffic shall be designed to resist a 600 kip impact load without collapse of the supported structure or shall be protected by concrete barriers or rigid steel barriers with a minimum 2-ft. shoulder. In cases where loss of the temporary tower would cause collapse of the supported structure the tower shall be protected with a barrier and have a 2-ft shoulder.

The barriers shall have a minimum 2-ft. clear zone of intrusion from the tower to the back face of the barrier. For speeds between 35 mph and 45 mph, the barrier shall either be at least 54 in. tall or have a 10-ft. clear zone of intrusion and be at least 42 in. tall. If the speed is expected to be over 45 mph, if the ADTT exceeds 10,000 vehicles per day, or if the through traffic is railroad or light rail traffic, then the barrier shall have the strength, stability, and geometry required for a TL-5 barrier. Guardrails protecting falsework towers or piers shall continue at full rail height for at least 30 ft. either side of the tower and shall be configured with full height rigid barriers to prevent vehicles from running around the rail end and hitting the tower from the opposite side of the rail. If ends transition into lower approach rails rather than crash cushions or barrels, that approach rail shall be a rigid rail type (such as Type 7) and shall not end for at least an additional 170 ft. This extension of the approach rail prevents a vehicle mounting and straddling a barrier from reaching the tower or pier.

3.6 VEHICULAR LIVE LOAD

Vehicular live load shall be in accordance with AASHTO. ***AASHTO 3.6.1.2***

Bridges should be designed such that future removal of medians and/or sidewalks is considered in the design and rating of the bridge. Simultaneous loading of the sidewalk dead load and vehicle live load is recommended when barrier separation is not present to cover the likelihood of errant trucks mounting the sidewalks or medians. Pedestrian load need not be applied in addition to the vehicle live load in this case.

Live load factors for Service III shall be in accordance with AASHTO Table 3.4.1-4. See Section 5.5.1 of this BDM for further explanation of applicability of the different live load factors.

The Colorado Permit Vehicles and Emergency Vehicle (EV3) shall be evaluated at Strength II. Figure 3-1 shows the axle weights and axle configuration that represent the Colorado Permit Vehicle. The Colorado Permit vehicle is used to determine the Overload Color Code for bridges. These are moving live loads using the same live load distribution factors, number of lanes loaded, and impact factors as the HL-93 truck. The intent for including these Vehicles in the design is to get an operating rating value of 1 or greater.

Deck slabs do not need to be designed for these Vehicle wheel loads.

An operating rating for the Permit Vehicle shall be provided on the Bridge Rating Summary Sheet (see the CDOT Bridge Rating Manual).

Additional design vehicles, such as Specialized Hauling Vehicles (SHVs), Notational Rating Load (NRL), Emergency Vehicles (EVs), and other legal loads shall be evaluated in accordance with the CDOT Bridge Rating Manual and the AASHTO *Manual for Bridge Evaluation*.

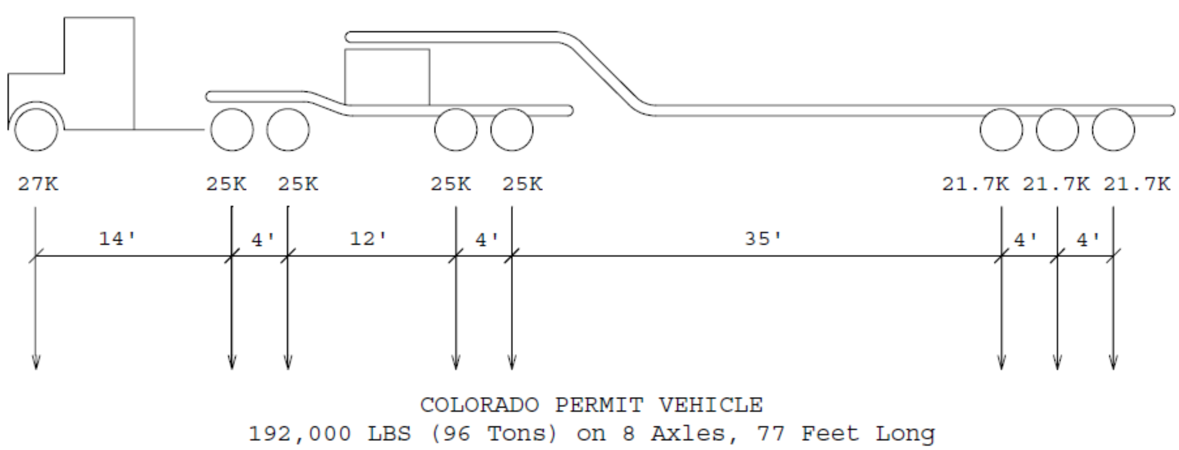

Figure 3-1: Colorado Permit Vehicle

3.7 VEHICULAR LIVE LOAD ON CULVERTS

CDOT considers surcharge from lane loads in the design of box culverts. To maintain consistency with CDOT's M-standards, surcharge loads from lanes shall be applied to the walls and bottom slabs of culverts using the Boussinesq stress distribution.

AASHTO 3.6.1.2.6 and 3.6.1.3.3

Thrust may be considered in the design of box culverts (precast or cast-in-place). If thrust is considered in the design, the rating is to incorporate thrust and design assumptions are to be included within the design plans.

For arch culverts, soil structure interaction with refined analysis shall be used for vehicular load and for identifying positive arch action.

3.8 DECK OVERHANG LOAD

Bridge deck overhangs shall be designed for horizontal loads resulting from vehicle collision in accordance with AASHTO. For deck overhang greater than 1/3 of the girder spacing, special attention shall be paid to shear capacity and concrete screed machine load during deck pour.

AASHTO 3.6.1.3.4

The AASHTO Chapter methodology for determining impact loads on the overhang as shown in the BDM examples is very conservative. Recent research has shown that the combined impact tension load on deck reinforcing at the flowline of concrete barriers may be as low as 14.8 kip/ft for a TL-4 rail. This value would be increased 100% at rail expansion joints. These reduced values are due to torsion and yield line capacities not currently shown in the code.

3.9 CENTRIFUGAL FORCES

For piers and abutments with a pin connection between the superstructure and substructure, centrifugal forces may be assumed to act horizontally at the roadway surface. For piers and abutments with a moment resisting connection between superstructure and substructure, the eccentricity of the centrifugal force shall be considered. Centrifugal forces shall be distributed to substructure elements based on their relative individual longitudinal stiffness.

AASHTO 3.6.3

3.10 BRAKING FORCE

For piers and abutments with a pin connection between the superstructure and substructure, braking forces may be assumed to act horizontally at the roadway surface. For piers and abutments with a moment resisting connection between superstructure and substructure, the eccentricity of the braking force shall be considered. Braking forces shall be distributed to substructure elements based on their relative individual longitudinal stiffness.

AASHTO 3.6.4

CDOT has experienced loss of backfill material (voids) behind abutments of existing bridges due to water intrusion over time. In addition, cyclical temperature movements of bridges may cause gaps between backfill and abutments. Due to these considerations, relying on passive earth pressure behind abutments to resist braking loads is cautioned. If passive earth pressure behind abutments is considered, AASHTO Table C3.11.1-1 should be used to estimate the participation of passive earth pressure relative to pier stiffness.

3.11 FATIGUE LOAD

Due to the uncertainty of future traffic volumes, the maximum ADT per lane of 20,000 vehicles shall be used when evaluating fatigue. In lieu of site-specific fraction of truck traffic data, the values of AASHTO Table C3.6.1.4.2-1 may be applied to obtain ADTT for use in Equation 3.6.1.4.2-1.

AASHTO 3.6.1.4.2

3.12 STREAM FORCES AND SCOUR EFFECTS

Stream forces shall be designed in accordance with Section 3.7.3 of AASHTO. Debris raft loads need only be applied on structures within high debris channels as determined by the Hydraulic Engineer.

AASHTO 3.7.2, 3.7.3, and 3.7.5

Scour of bridge foundations should be evaluated at two levels:

- Strength I – Evaluate 100-year scour in conjunction with maximum dead load factors, live load, and stream forces. If the 100-year scour limits undermine beyond the back of abutment, impeding live load from approaching the structure, live load may be reduced.
- Extreme Event II – Evaluate 500-year scour in conjunction with minimum dead load factors and stream forces. Live load may be reduced if the approaches to the bridge are impassable due to scour. The extreme event check should verify that the bridge will not collapse.

All other service, strength, and extreme event combinations need not be checked concurrent with the 100-year or 500-year scour limits.

3.13 SEISMIC LOADING

For bridges and other structures within Seismic Zone 1, the minimum connection requirements of AASHTO shall apply.

AASHTO 3.10.9.2

For all other seismic zones, both force-based and displacement-based analysis methods are allowed. A geotechnical investigation must be completed for bridges to determine the site class of the foundation materials. When using Extreme Event I, the load factor on live load should be 0.50. The 0.50 live load factor signifies a low probability of the concurrence of the maximum vehicular live load and the extreme event case.

Seismic analysis is not required for mechanically reinforced earth (MSE) and Geosynthetic Reinforced Soil (GRS) walls if Staff Bridge Structural Worksheets are used. These worksheets contain damage avoidance details such as rail anchor slab/beam, coping, and shiplap panel joints that cannot be revised without approval by Unit Leader in coordination with the MSE Wall SMEs. See Section 2.16 of this BDM for further details.

3.14 TEMPERATURE / THERMAL FORCES

Structures shall be designed for the temperature ranges detailed in Section 14 of this BDM.

3.15 EARTH PRESSURES AND SETTLEMENT EFFECTS

Appropriate earth pressures and predicted settlement should be provided in a geotechnical investigation. The Geotechnical Engineer shall evaluate criteria for settlement periods and potential down drag effects.

Consideration should be given to lateral earth pressures from surcharge loads in accordance with AASHTO 3.11.6, modified on a project-specific basis. For structures that support vehicular live loads within the stated criteria of AASHTO 3.11.6.4, the load factor on the surcharge shall be in accordance with LS in AASHTO Table 3.4.1-1. For walls designed for a nominal surcharge to account for backfilling operations, the load factor on the assumed surcharge may be taken as 1.50. The lower load factor represents the temporary nature of this surcharge effect and reflects the construction load factor in AASHTO 3.4.2.1. ***AASHTO 3.11.6***

A combination of mechanically reinforced earth (MSE) with a non-collapsible void or a gap with low density polystyrene can be considered when reduced earth pressure effects are required.

Settlement shall be evaluated at the service limit state with a load factor of 1.0 applied to all applicable loads. Transient loads may be omitted from settlement analysis.

Effects of abutment settlement on bridges using Geosynthetic Reinforced Soil (GRS) abutments shall be evaluated during the structure selection stage. See Section 11 of this BDM for additional requirements.

3.16 PEDESTRIAN LOADING

Pedestrian load should be considered in accordance with AASHTO and the AASHTO *LRFD Guide Specifications for the Design of Pedestrian Bridges.* ***AASHTO 3.6.1.6***

3.17 BLAST LOADING

The potential for blast loading shall be evaluated and documented during the structure selection process and coordinated with Staff Bridge on a project-specific basis. ***AASHTO 3.15***

3.18 WIND LOADS

Wind loads shall be in accordance with AASHTO. Staff Bridge shall be consulted for structures within special wind regions not covered by AASHTO. ***AASHTO 3.8***

3.19 FENCE LOADS

Table 3-1 shows the minimum load for which fences on bridges and other structures shall be designed unless site conditions justify a different load condition. Refer to Section 13 of this BDM for additional information. Calculated load values were generated using the *Chain Link Fence Wind Load Guide*, 2007. Snow loads are based on energy momentum equations by calculating the power available from a snowplow moving at 45 to 50 mph to determine the maximum amount of snow that could be continuously thrown. This provides the momentum of the snow thrown per second. Dividing the momentum by time yields the snow impact loads that are shown in Table 3-1.

Table 3-1: Fence Loads

Fence Type	Chain Link Opening	Wind Load	Snow Impact Load*
36" Chain Link splash guard	3/8"	31 psf	96 plf 1'-6" up from bottom of fence
60" Chain Link	1"	14 psf	96 plf 1'-6" up from bottom of fence
68" Chain Link	2"	8 psf	96 plf 1'-6" up from bottom of fence
92" Chain Link	2"	8 psf	96 plf 1'-6" up from bottom of fence

* The required mesh opening for CDOT snow fence is 3/8”.

3.20 REFERENCES

Chain Link Fence Manufacturers Institute. 2007. *Chain Link Fence Wind Load Guide*.