Oregon Department of Transportation

SECTION 1.3
Loads and Distributions

Revisions for the current edition are highlighted yellow. Deleted text is not marked.

Past editions of the Bridge Design Manual are available through our website.

1.3.1 Ductility, Redundancy and Operational Importance

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1.3 Loads and Distributions

1.3.1 Ductility, Redundancy and Operational Importance

Apply the ductility per *LRFD 1.3.3* without change.

Modify *LRFD 1.3.4* redundancy factor as follows:

- Do not use a redundancy factor less than 1.0.

C1.3.1

Modify *LRFD 1.3.5* operation importance factors as follows:

- Consider all bridges as **typical,** ($\eta^I = 1$) for the operational importance factor.

1.3.2 Dead Loads

Provide the dead load allowances shown in *Table 1.3.2-1* for all vehicular bridges.

C1.3.2

See *AASHTO 3.5.1* for dead loads not listed.

The utilities load stated is a minimum utility design load for future use on the bridge superstructure to be assumed when the weights of proposed utilities are unknown. There will be situations where utilities are heavier. Use project specific utility loads when they are defined. See *BDM 1.12.6*.

Table 1.3.2-1 Design Dead Load Unit Weights

Load Type	**Loading**	**Comments**
Asphaltic Concrete Pavement (ACP)	0.150 kcf	
Bridge Rails		See *BDM 1.13.5, 1.13.6.2* and *ODOT LRFR Manual*
PPC Overlay	0.135 kcf	See *BDM 1.9.3.1.5*
Protective Screening		Per standard drawings
SIP Deck Forms	0.010 ksf	Precast Prestressed – See *BDM 1.5.6.2.1.* CIP - Use when deck forms are not required to be removed. See *BDM 1.5.7.7*
Future Utilities	0.090 klf	Based upon a 12 inch diameter water filled pipe. See *BDM 1.12.6.*
Concrete*		
Lightweight Concrete	0.135 kcf	See *BDM 1.5.1*
Concrete (HPC-IC)	0.140 kcf	
Concrete (CIP, HPC)	0.145 kcf	For cast-in-place and precast
Concrete (Precast Prestressed)	0.150 kcf 0.155 kcf	f'c less than or equal to 8.0 ksi f'c greater than 8.0 ksi
Concrete (UHPC)	0.155 kcf	

Tables Notes:

*Unit weights are for unreinforced concrete. Add 0.005 kcf for reinforcing steel in concrete.

1.3.2.1 Wearing Surface

Provide the minimum present wearing surface (PWS) and future wearing surface (FWS) allowances shown in *Table 1.3.2.1-1* for all new vehicular bridges.

C1.3.2.1

Generally, FWS is not required for pedestrian structures. When the width of the pedestrian bridge allows trucks to pass regularly, FWS may be warranted. Consult the BDM Deck Specialist and *BDM 1.5.1*.

Table 1.3.2.1-1 Wearing Surface Dead Loads

Bridge Type	Present Wearing Surface (ksf)	Future Wearing Surface (ksf)
All new bridges with CIP concrete decks	0	0.025, (2 inches)
Side-by-side construction with PPC overlay	0.012, (1 inch) *	0.025, (2 inches)
Side-by-side construction with ACP (Local Agency projects only)	0.025 (2 inches)* **	0.040, (3 inches)

Table Notes:

* Provide additional PWS allowance as needed to account for camber and superelevation buildup. Determine actual thickness from roadway profile.

** Required PWS depth is intended to allow future pavement preservation via inlay while minimizing risk of damage to girders. Use a minimum 0.050 ksf (4 inches) thickness when a rolled membrane is used.

1.3.2.2 Load Rating of Existing Bridges

See *ODOT LRFR Manual 1.7* to determine when an existing load rating needs to be updated. Review the latest load rating and when needed conduct a pre-design load rating per *BDM 2.10.8.1*. When the existing load rating is not in accordance with LRFR standards, consult with ODOT's load rating unit before using.

For all non-load-path-redundant steel truss bridges, verify that the stress levels in all structural elements, including gusset plates, remain within applicable requirements whenever planned modifications or operational changes may increase stresses.

C1.3.2.2

It is critical to know the rating factor of each member anytime there is a change in capacity, condition, or dead load.

When the load rating of the existing structure is available, check the latest Bridge Inspection Conditions' Rating Report against condition rating used for load rating. When the current superstructure or substructure condition factors are below those used in the load rating, the current load capacity of the bridge may not be accurately reflected in the load rating. Review the asphalt thickness used (if any) in the bridge load rating versus the values in the inspection report. Also review the rating

date and identify if any bridges updates have occurred since this date.

A load rating that reflects the current condition of each bridge is valuable and is used to identify when a bridge needs load posting or strengthening. Rating of a structure decreases with an increase in dead load and may result in a bridge load posting.

See *BDM 2.10.8* for additional information.

Contact the Senior Load Rating Engineer for assistance in a load rating.

1.3.3 Live Loads

1.3.3.1 New Vehicular Traffic Structure

Use live loads specified by *AASHTO LRFD Bridge Design Specifications* and the following:

- Service and Strength I Limit States:
    - HL-93: Design truck (or trucks per *LRFD 3.6.1.3*) or the design tandems and the design lane load.
- Fatigue truck per *LRFD 3.6.1.4.*
- Strength II Limit State:
    - ODOT OR-STP-5BW permit truck.
    - ODOT OR-STP-4E permit truck.
    - EV3.

C1.3.3.1

ODOT permit loads are shown in ODOT LRFR Manual *1.5.1.5B*.

Figures 1.3.3.1-1 to *1.3.3.1-4* show live load effects for single span bridges with prismatic girders including a dynamic load allowance of 1.33, a multiple presence factor of 1.00, but without live load distribution factors. A load factor of 1.75 (Strength I) was applied to HL-93 design truck loads and a load factor of 1.35 (Strength II) was applied to ODOT OR-STP-4D, ODOT OR-STP-4E, ODOT OR-STP-5BW, and EV3 truck loads. The plots may be used to determine which design truck could control during a preliminary design given a span length for the project.

Figure 1.3.3.1-1 Live Load Plus Impact for Single Span Prismatic Members – Bending Moment at Mid-Span, (span lengths 30 – 150 feet)

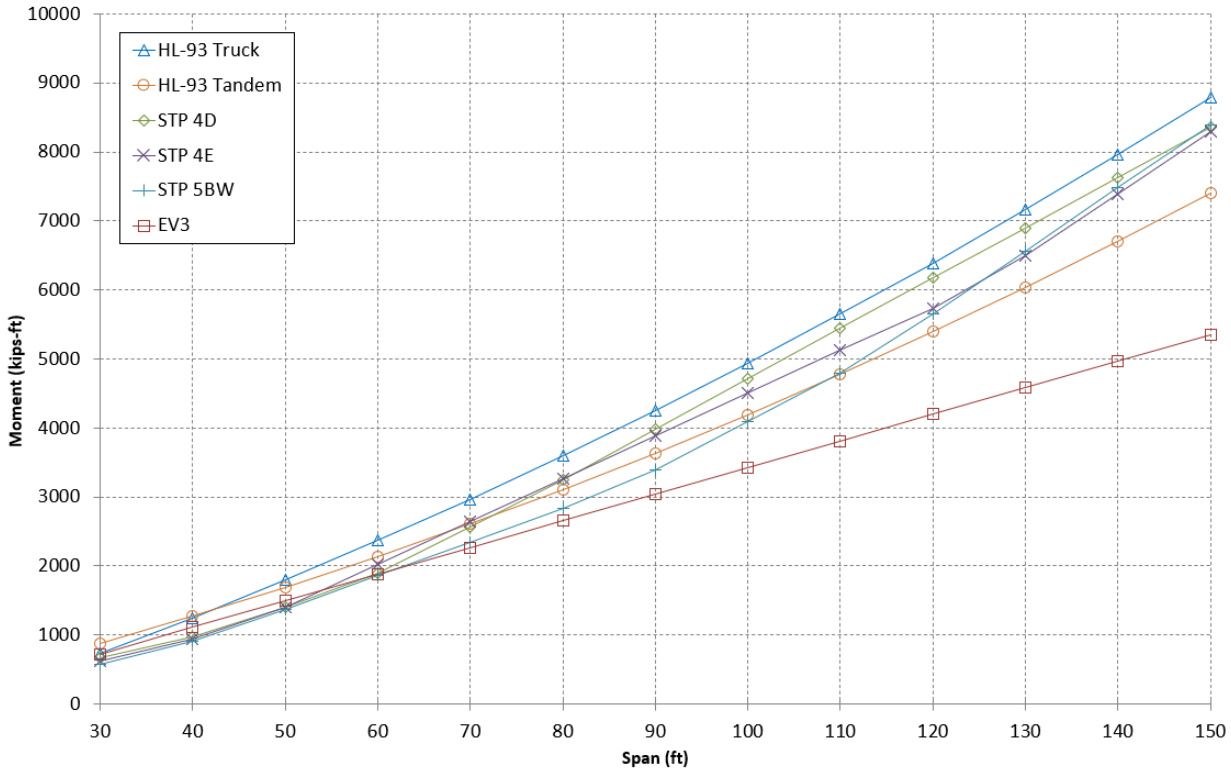


Figure 1.3.3.1-2 Live Load Plus Impact for Single Span Prismatic Members – Bending Moment at Mid-Span, (span length 150 – 300 feet)

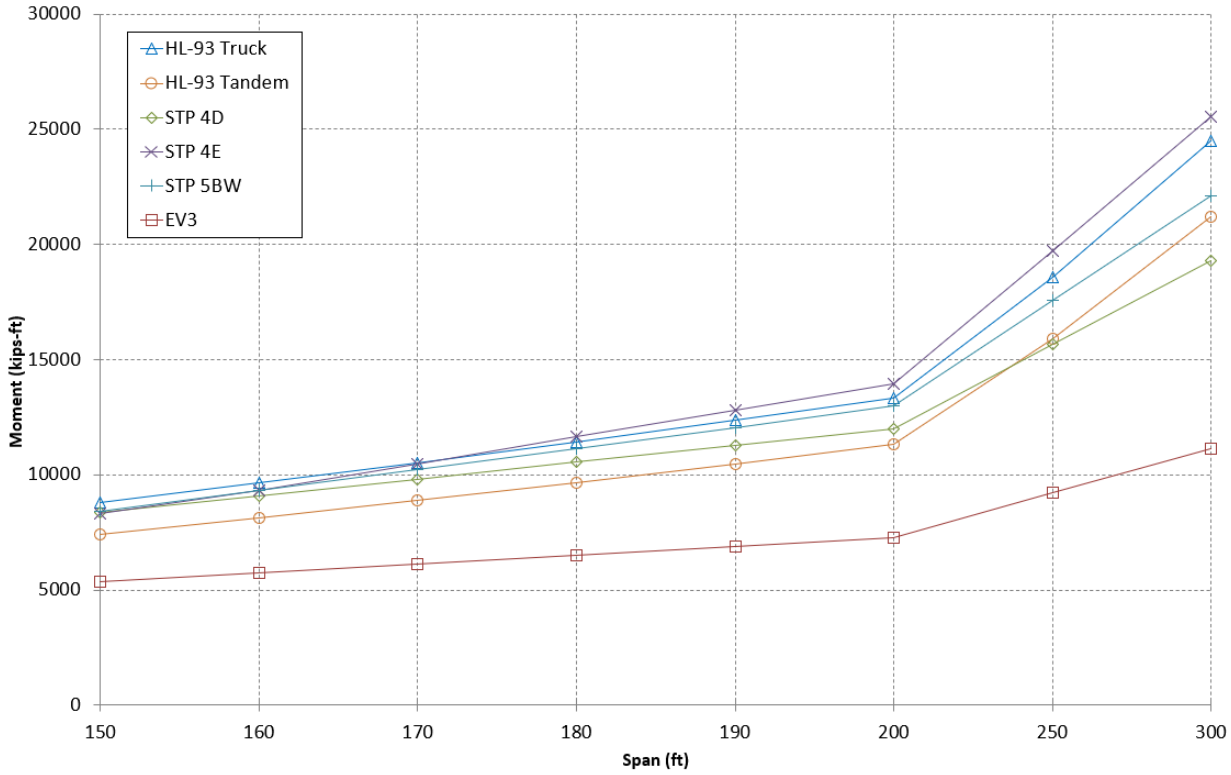


Figure 1.3.3.1-3 Live Load Plus Impact for Single Span Prismatic Members – Maximum Shear, (span length 20 – 150 feet)

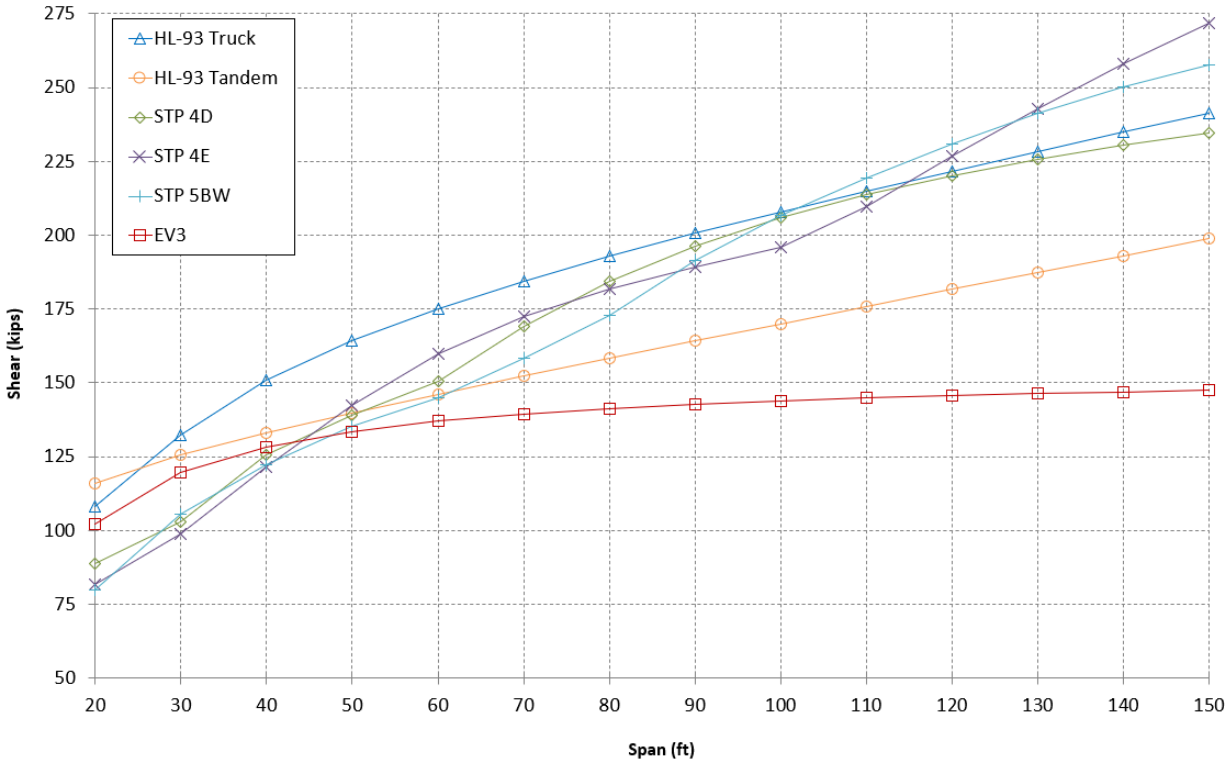
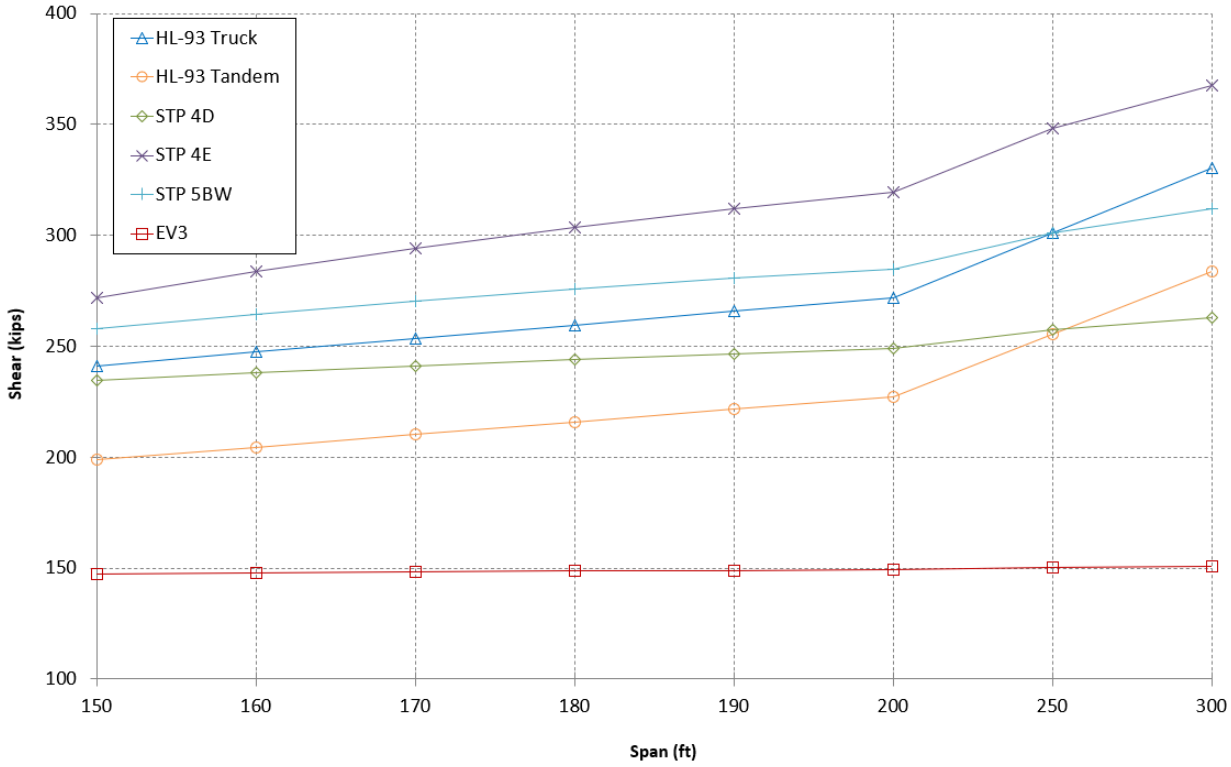


Figure 1.3.3.1-4 Live Load Plus Impact for Single Span Prismatic Members – Maximum Shear, (span lengths 150 – 300 feet)



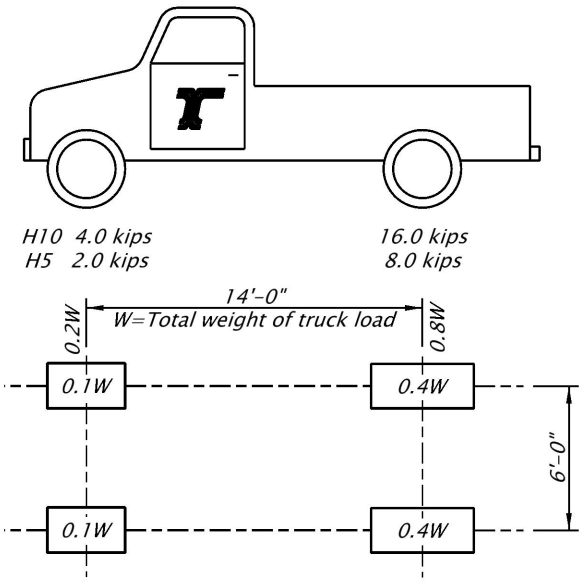
1.3.3.2 Pedestrian Structures

For bridges designed for pedestrian or bicycle traffic only, use a live load of 0.090 ksf. When an agency design vehicle is not specified, use *AASHTO* Standard H-5 or H-10 Truck loading as shown in *Figure 1.3.3.2-1* to check the longitudinal beams. A vehicle impact allowance is not required.

For a pedestrian or bikeway bridge clear deck width less than 7 feet do not consider the maintenance truck. See also the *AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges.*

Figure 1.3.3.2-1 H-5 or H-10 Truck Loading

Clear deck width 7 to 10 feet	10,000 lb. (H5 Truck)
Clear deck width over 10 feet	20,000 lb. (H10 Truck)



C1.3.3.2

1.3.3.3 Widening of Vehicular Traffic Structures

When widening an existing structure, design the widening using the loading in *BDM 1.3.3.1*.

Do not use live loading less than the design live load for the existing structure.

C1.3.3.3

1.3.3.4 Structure Strengthening

Design strengthening projects for the maximum load effect from the following permit trucks using the *AASHTO LRFD Bridge Design Specifications* Strength II Limit State:

- ODOT OR-STP-4D.
- ODOT OR-STP-5BW.
- ODOT OR-STP-4E.
- EV3.

For strengthening of prestressed concrete structures, ensure the requirements of Service I and III Limit States are satisfied using HL-93 loading.

Do not include FWS in strengthening design. When strengthening, determine existing wearing surface loads according to *ODOT LRFR Manual 2.2.8*.

C1.3.3.4

For design loads for structural repair projects, refer to the *Bridge Maintenance Design Manual*. See BDM 1.15.1 and 1.15.2 for additional criteria for repair and strengthening bridges.

See *BDM 1.3.3.1* and *ODOT LRFR Manual 1.5* for vehicle descriptions and *LRFD Table 3.4.1-1* for Load Factors.

The EV3 truck only controls the strengthening design on shorter span structures (less than 50 feet) and is not meant to impact every strengthening design.

1.3.4 Construction and Temporary Loading

Specifications dictate the contractor to design temporary works, except seismic loading, according to the *AASHTO Guide Design Specifications for Bridge Temporary Works, 2nd Edition, 2017*, unless specified otherwise therein.

Specifications dictate the contractor to construct temporary works according to *AASHTO Construction Handbook for Bridge Temporary Works, 2nd Edition, 2017*, unless included in the *Oregon Standard Specifications for Construction and Special Provisions* or specified therein.

Demonstrate at least one feasible means of construction, i.e. the incomplete structure is stable and not overstressed throughout a commonly used construction process relying on available equipment. Document the method in the calculation book but do not show results in contract documents, except as noted below:

- Jacking plans typically show estimated jacking loads. See *BDM 1.3.4.1*.
- Work platform and containment contract documents typically offer loading limits that will not overload the existing structure, and only if these limits are exceeded require stamped calculations showing that the proposed work platform & containment doesn't overstress the structure. See *BDM 1.3.4.3*.
- Geotechnical shoring plans may show estimated loading. See *GDM 15*.

C1.3.4

The intent of this section is to provide guidance to designers so they can effectively specify design parameters and review contractor designed temporary works submittals.

See *BDM 1.16.2* for Bridge Temporary Works.

When reviewing shop drawings for erection plans, refer to *AASHTOs Guide Specifications for Wind Loads on Bridges During Construction.*

When design-bid-build plans are advertised, the Agency is warranting that the contract documents represent a buildable design.

Longstanding industry practice requires the Agency to avoid dictating the contractor's means and methods, except when specific means and methods are necessary. Stability of the structure during active construction is the contractor's responsibility.

It is not practical to require a contractor to provide a structural analysis of an existing structure.

Refer to *AASHTO 2.5.3* for additional clarification on constructability.

- Proposed temporary bracing for prestressed girders typically is detailed in contract plans.
- Contract documents may show limits for crane loading on existing structures.
- Instances where constructability dictates it is necessary.

1.3.4.1 Temporary Support and Jacking Load

Provide a jacking load diagram on the plans that identifies proposed jack locations and sizes. Provide a jacking load table on the plans that identifies the loads, load combinations, and other load requirements for the development of the jacking plan. Use the jacking load table shown in *Table 1.3.4.1-1*. The table may need to be modified for project specific designs.

For horizontal construction loads, design to *AASHTO Guide Design Specifications for Bridge Temporary Works, 2nd Edition, 2017 Section 2.3.4.4.*

C1.3.4.1

The loads specified in the jacking load table are intended to be used by the contractor's engineer to design the temporary work structures.

Determine forces generated by jacking and a jacking location during design. The existing structure elements must be able to resist the jacking forces. When the jacking location is limited by the existing structure situation, propose a shoring structure extending from the existing structure or supported by a designed foundation for jacking.

Consider uplift on structures with short end spans.

Loads provided in *Table 1.3.4.1-1* are intended to be calculated loads with no allowance for "initial break-loose" force. When selecting jack capacity, experience has shown a jacking force at least 25 percent higher than calculated loads are required.

Table 1.3.4.1-1 Example Jacking Load Table

Location	Service Dead Load (Kips/Girder)	Service Live Load (Kips/Girder)	Factored Dead Load (1.5 x DC & DW) (Kips/Girder)	STR II (Kips/Girder)
Bent 1				
Bent 2				

Table notes:

Keep construction live load below the live load shown in the table.

Service and factored live loads include a 25% dynamic load allowance.

Modify *AASHTO 3.4.3.1* as follows:

Specify design forces for jacking when the structure is not in service as 1.5 times the permanent load reaction at the bearing, adjacent to the point of jacking.

Do not include FWS in jacking design. When jacking, determine existing wearing surface loads according to *ODOT LRFR Manual 2.2.8.*

Specify design forces for jacking in service at the bearing, adjacent to the point of jacking, for lanes in service, using permit trucks from *BDM 1.3.3.4* as *AASHTO* Strength II Limit State modified as follows:

- 25 percent IM load allowance.

Determine live load forces consistent with the traffic control plan.

A structure is considered in service when it is open to traffic during the jacking operation.

Permanent loads consist of all dead loads, self-weight, bridge rails, wearing surface, etc.

When structure is not in service, designers may assume construction live loading does not exceed traffic live loading as specified in *BDM 1.3.3.4*.

1.3.4.2 Bridge Raising

Use the loading criteria and jacking load table specified in *BDM 1.3.4.1* for designing bridge raising operations, except as modified in this section.

Close the bridge to traffic during active bridge raising operation.

C1.3.4.2

Permanent loads consist of: superstructure and substructure (above the chip-in point when using the chip-in method), superimposed dead loads (utilities, signs and other dead loads that will remain on the bridge during the raising operation). Field verify all dead loads at Project Initiation.

Wind loads during construction are considered low risk and are typically ignored since operations are low to the ground.

Consider whether the bridge should be open to traffic during inactive construction periods. It is preferred to close the bridge to traffic for the duration of construction. See *BDM 1.16.1* for additional information on bridge raising. Modify *SP 00255* as needed to incorporate bridge raising operations.

1.3.4.3 Falsework

When falsework falls within the live load path, design to *AASHTO* Strength II Limit State, except as modified in *SP 00540*.

C1.3.4.3

Refer to *SP 00540* for additional loading requirements at highway and railroad traffic openings.

Refer to the *Glossary* for a definition of Falsework. See *BDM 1.16.3.3* for additional information on Falsework.

Where wind loads during construction are expected to exceed this lateral load, consider increasing the lateral load to account for relevant wind loads.

1.3.4.4 Work Access Platforms

When work access platforms are necessary for more than five consecutive shifts:

- Evaluate a single configuration of work platform loading.
- Include reasonable limits for the work platform on the plans.
- Specify allowable point loads, and where they can be applied in *SP 00253.09*.
- Evaluate load rating to determine the remaining capacity of the structure that can be used for containment or work platforms.

When wind loading is considered, specify the maximum allowable containment size in *SP 00253*.

When evaluating a load rating, obtain the System Factor, Condition Factor, and Evaluation Live Load Factor from the most recent load rating when available. When there is not an existing load rating, complete a load rating to ensure the existing structure can carry the work access platform or containment. When necessary, update an existing load rating per *BDM 1.3.2.2*.

For non-load rated elements, evaluate the structure for the ability to carry temporary work access platform using Strength II loading. Use a construction live load factor of 1.5 and construction live loading from *AASHTO Guide Design Specifications for Bridge Temporary Works Table 2.3.4.3.3-1*.

Assume work platforms and containment extend 5 feet beyond the sides of the structure and the full length of each span.

C1.3.4.4

The contractor does not need to evaluate the existing structure when they remain within the prescribed limits. Evaluate the work platform to ensure a single configuration is constructible. It is not feasible to consider every option available to a contractor.

Contact the Senior Load Rating Engineer with questions. Refer to *SP 00253* for wind speeds.

An example of very light duty is cable supported corrugated/steel decking.

An example of light duty is frame and tube supported decking system.

Medium and heavy duty systems are typically custom designs.

It may be necessary to separate the work platforms or containment into zones for structural reasons. Carefully size the zones to allow efficient work by the contractor.

1.3.5 Sidewalk Loading

C1.3.5

For sidewalks not separated from traffic by a structural rail, account for the potential for a truck to mount the sidewalk. Design the sidewalk for the greater of:

- 0.075 ksf pedestrian loads considered simultaneously with the vehicular load in the adjacent lane as stated in *3.6.1.6* of the *LRFD Bridge Design Specifications*. Per *LRFD 3.6.2.1*, impact does not apply to pedestrian loads.
- The *LRFD* design truck placed with a line of wheels 2 feet from the face of rail. Do not apply a lane load with the design truck, but do include impact. Consider this load only under the Strength I limit state. Do not consider trucks or vehicle loads in adjacent lanes. Do apply the multiple presence factor for this case.

In addition to the above cases, ensure the supporting member (exterior girder) is adequate for HL-93 loading when the sidewalk is removed and an approved barrier type (*BR290* or *BR208*) is placed at the edge of deck per *BDM 1.13.5*.

1.3.6 Vehicular Collision Forces: CT

Modify *LRFD 3.6.5* as follows:

Where the design choice is to redirect or absorb the collision load, and a mulit-column pier is used, use pier systems with three or more columns.

Specify pier protection that consists of a minimum 42 inch high MASH TL-5 rigid barrier with standard pin anchorage to subgrade (see *RD500* series), located such that the top edge of the traffic face of the barrier is 3.25 feet or more from the face of the pier component being protected.
Based on the clear distance from the face of the pier component to the traffic face of the barrier, use minimum column sizes as follows:

- For 0 to 3.25 feet clear distance, use a minimum 48 inch circular (or equivalent square) column with 1 1/8 percent minimum longitudinal steel reinforcement and #4 spiral reinforcement with 4 inch pitch.

C1.3.6

The distance from the front face of barrier to obstacle is called the Zone of Intrusion (ZOI) and varies by TL. Prior to 2020, tthe standard detail for barrier protection of a structure (column, wall, traffic support pole, or other structure) places the curb face of the barrier 4 feet minimum from the face of the structure. This detail dates back at least as far as the mid 1990's. *AASHTO 9th Edition (2020)* updated this spacing to 3.25 feet to match current MASH research. The ZOI dimension is intended to allow room for **rollover** when a truck impacts the barrier. 3.25 feet clears most ZOI for different casescases but barrier impacts can create a rollover scenario that exceeds 3.25 feet. Therefore, consider clear distances exceeding 3.25 feet roadway width standards are not impacted.

- For 3.25 feet or greater clear distance, use a minimum 36 inch circular (or equivalent square) column with 1 1/8 percent minimum longitudinal reinforcement and #4 spiral reinforcement with 4 inch pitch.

Where the 3.25 feet clear distance cannot be achieved, consult with the roadway designer to confirm what clear distance can be provided. When the design shoulder width is encroached upon by the placement of the concrete barrier, consult the roadway designer to develop the best solution to protect the bridge columns.

In all cases, provide 3 inch minimum clear distance from the back of pinned barrier to the face of the structure.

When 3.25 feet or more clear distance can be provided, the proposed barrier placement detail will meet standards and no additional consultation with the roadway designer is necessary. It should be understood that such a detail may pose some risk of structure impact, even when small.

Any clear distance less than 3.25 feet results in additional risk for catastrophic impact of the structure being protected. Therefore, consider reducing roadway shoulder width. The desired result should be to find an acceptable balance between roadway risk (due to inadequate shoulder width) and structure risk (impact to the structure being protected).

When evaluating the roadway and structure risk, the following factors should be taken into account:

- Alignment of the roadway – straight vs. curved, inside of curve vs. outside of curve.
- Length of roadway width reduction (e.g., single sign support vs. long retaining wall).
- Ability of the structure to absorb a hit (single column vs. wall abutment).
- Traffic volume – higher traffic volume means higher risk of an incident.
- Consequences of structure failure.

Reducing clear distance includes accepting significant risk. The 3 inch minimum clear distance is intended to minimize the amount of horizontal impact load that would be transferred from the impacted barrier to the structure being protected. The void between the back of the barrier is typically filled with pea gravel. Note that 3 inch concrete surfacing is often provided at the top of

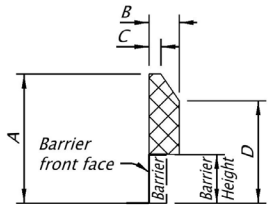
barrier for aesthetic purposes. This surfacing is expected to disintegrate upon impact and so would not be expected to transmit unacceptable forces to the structure being protected. In cases where reduction of the clear distance width provides an unacceptable risk against rollover, the barrier can be transitioned from a safety shape to vertical. Vertical barrier will reduce vehicle rollover and, in some cases, can provide an installation with an acceptable level of risk.

Intrusions zones for MASH TL3 and TL4 barrier are shown in *Figures 1.3.6-1* and *1.3.6-2* are based on NCHRP Report 1018.

Intrusion zones for NCHRP 350 TL3 and TL4 barrier are diagramed in Guidelines for Attachments to Bridge Rails and Median Barriers Midwest Roadside Safety Facility (MwRSF).

Earth mounds are no longer an acceptable method of column protection. At this time, existing earth mounds do not need to be removed.

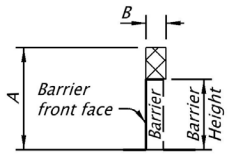
Figure 1.3.6-1 MASH TL-3 Zones of Intrusion



Barrier-Top Height (in.)	Dimension (in.)			
	A	B	C	D
29	77	18.0	7	61
32	77	18.0	5	61
34	*74*	*18.0*	*4*	*61*
36	71	*18.0*	3	61
38	*68*	*16.9*	*2*	*61*
40	*65*	*14.4*	*1*	*61*

NOTE:
Cells noted in unshaded cells represent values from simulation and/or full-scale testing. Italicized values shown in gray were interpolated based on data trend lines.

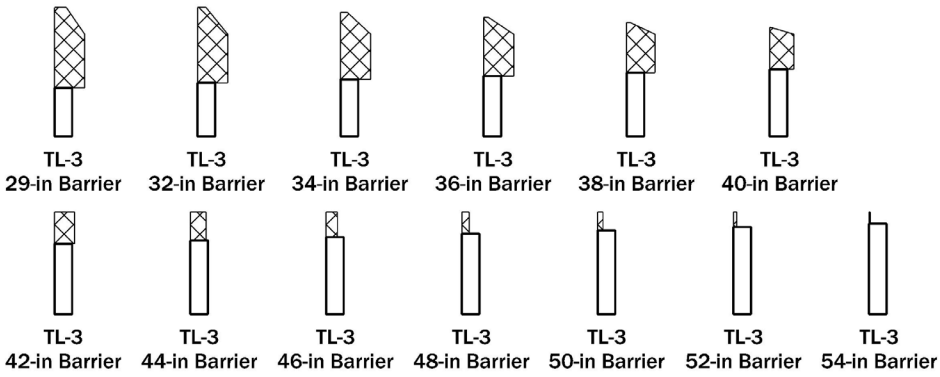
MASH TL-3
ZOI dimensions, barrier heights less than 42 in.



Barrier-Top Height (in.)	Dimension (in.)	
	A	B
42	61	12.0
44	*61*	*9.4*
46	*61*	*6.8*
48	*61*	*4.3*
50	*61*	*3.2*
52	*58*	*2.0*
54	58	0.8

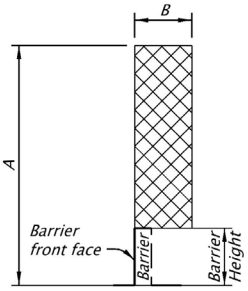
NOTE:
Cells noted in unshaded cells represent values from simulation and/or full-scale testing. Italicized values shown in gray were interpolated based on data trend lines.

MASH TL-3
ZOI dimensions, barrier heights 42 in. or taller



Comparison view of recommended ZOI envelopes:
MASH TL-3 impact conditions

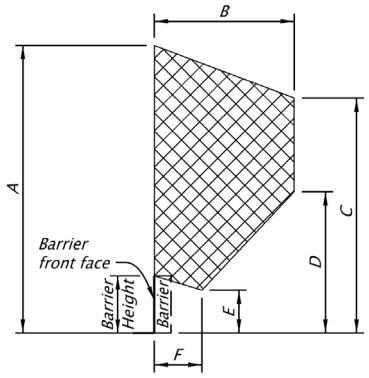
Figure 1.3.6-2 MASH TL-4 Zones of Intrusion



Barrier-Top Height (in.)	Dimension (in.)	
	A	B
36	115	36.0
42	111	24.0
46	*111*	*8.5*
48	N/A	0.0

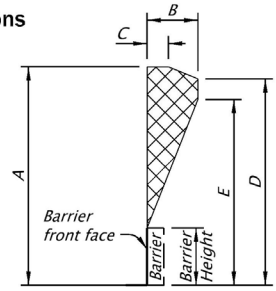
NOTE:
Cells noted in unshaded cells represent values from simulation and/or full-scale testing. Italicized values shown in gray were interpolated based on data trend lines.

MASH TL-4 cab ZOI dimensions



Barrier-Top Height (in.)	Dimension (in.)					
	A	B	C	D	E	F
36	181	88	148	89	27.0	30
42	181	63	146	103	36.5	24

MASH TL-4 ZOI dimensions: barrier heights less than 48 in.



Barrier-Top Height (in.)	Dimension (in.)				
	A	B	C	D	E
48	137.5	32.0	13.5	130	117
64	*137.5*	*25.0*	*13.5*	*129*	*117*
76	*136.0*	*20.0*	*13.5*	*129*	*116*
90	135.0	13.5	129	129	116

NOTE:
Cells noted in unshaded cells represent values from simulation and/or full-scale testing. Italicized values shown in gray were interpolated based on data trend lines.

MASH TL-4 box ZOI dimensions: barrier heights 48 in. or taller

1.3.7 Thermal Forces

C1.3.7

Use the temperature ranges shown in *Table 1.3.7-1.* Figure the rise and fall in temperature from an assumed temperature at time of erection.

Table 1.3.7-1 Temperature Ranges for Thermal Forces

Section *	Metal Structures	Concrete Structures	Annual Mean Temperature
Section I	+10° F. to +110° F.	+22° F. to +72° F.	52° F.
Section II	-10° F. to +120° F.	+12° F. to +82° F.	52° F.
Section III	-30° F. to +120° F.	0° F. to +82° F.	47° F.

* Section I designates that portion of the state west of the coast range.
Section II designates the valley region between the coast range and Cascade Mountains.
Section III designates the Cascade Mountains and all of eastern Oregon. For structures in the Columbia River Gorge, use Section III.

1.3.8 Wind Load

Determine wind load according to *LRFD 3.8.1*. Determine the design 3 second gust wind speed used in the determination of design wind loads on bridges and walls from *Table 1.3.8-2*. Use *Figure 1.3.8-1* to identify areas of special wind regions in *Table 1.3.8-2.*

C1.3.8

The wind speed map is adapted from the *2022 Oregon Structural Code* and *2022/2023 CPP Special Wind Region Study for Oregon* to account for locations in the state with special wind regions.

When verified wind speed using the ASCE 7 Hazard Tool. The risk category from building to mean recurrence interval (MRI) is as shown in *Table 1.3.8-1*.

Table 1.3.8-1 Risk Category from Building to Mean Recurrence Interval (MRI)

Building Risk Category	Description	MRI (year)	Wind Speed Correspond to
I	Low hazard to human life in case of failure	300	approximately a 15 percent probability of exceedance in 50 years (Annual Exceedance Probability = 0.00333)
II	Most Residential and Commercial Dwellings	700	approximately a 7 percent probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143)
III	Substantial risk to human life in case of failure	1700	approximately a 3 percent probability of exceedance in 50 years (Annual Exceedance Probability = 0.000588)
IV	Essential Facilities	3000	approximately a 1.6 percent probability of exceedance in 50 years (Annual Exceedance Probability = 0.000333)

Typical bridges are designed for 3-second gust wind speed, at an elevation of 33.0 ft, for wind exposure Category C, with an MRI of 700 years (See *LRFD 3.8.1.1.2*).

In all areas with full exposure to ocean winds, use the highest wind velocity for that MRI category.

In areas in Multnomah and Hood River counties with full exposure to Columbia River Gorge winds, use the highest wind velocity for that MRI category.

Examine mountainous terrain, gorges, and ocean promontories for unusual wind conditions.

For wind acting horizontally on the supports, signs, luminaires, traffic signals, the selection of the MRI accounts for the consequences of failure. A "typical" support could cross the travelway during a failure thereby creating a hazard for travelers (MRI = 700 years). All supports that could cross lifeline travelways are assigned a high risk

category to consider the consequences of failure (MRI = 1700 years). Supports that cannot cross the travelway are assigned a low risk and 300-yr MRI. (See *LRFD Specification for Structural Supports for Highway Signs, Luminaires, and Traffic Signals Section 3.1* and *3.8*).

Table 1.3.8-2 Basic Design Wind Speed, V

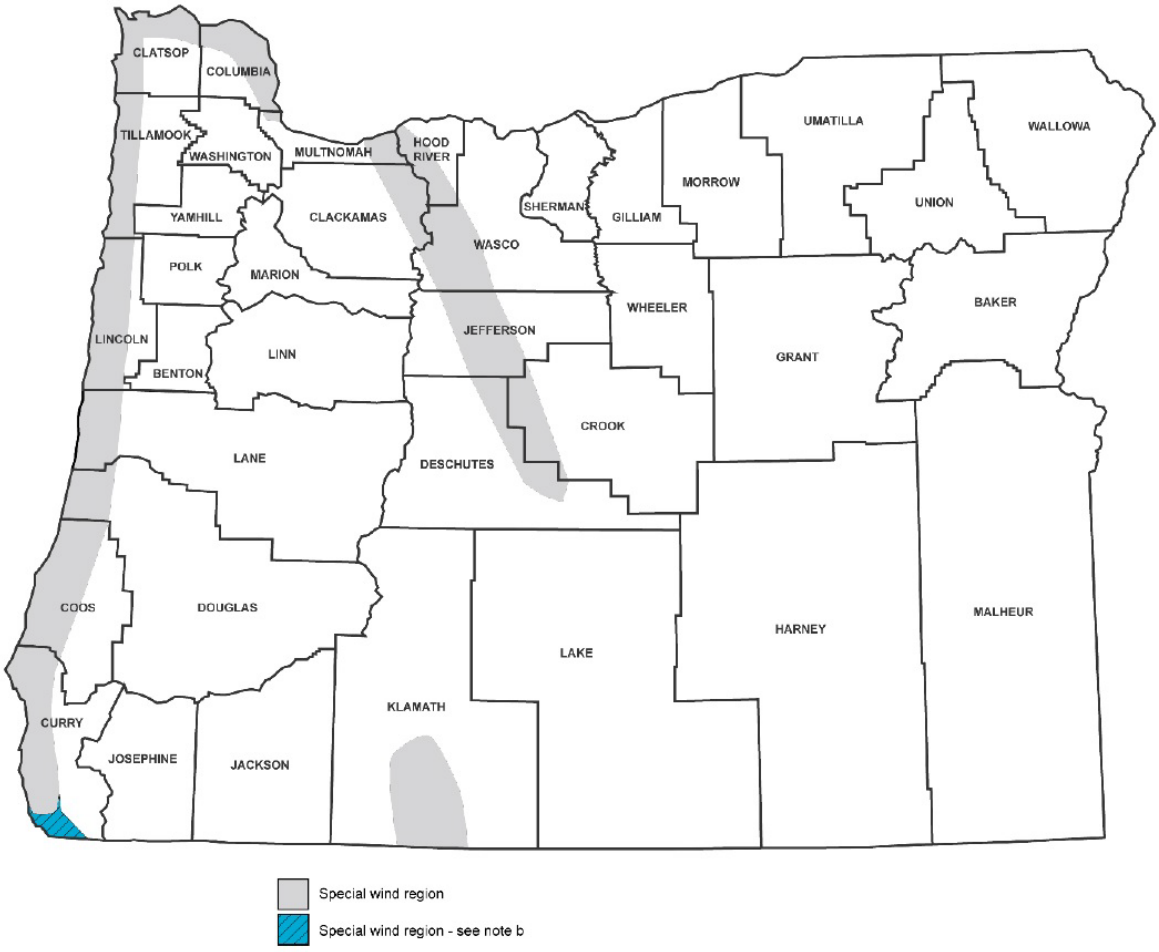
COUNTY	300-Year MRI Basic Design Wind Speed, V (MPH)	700-Year MRI Basic Design Wind Speed, V (MPH)	1700-Year MRI Basic Design Wind Speed, V (MPH)	3000-Year MRI Basic Design Wind Speed, V (MPH)
Baker	97	103	110	114
Benton	90	96	102	107
Clackamas	92	98	105	109
Clackamas special wind region[a]	92	98	105	109
Clatsop	91	96	102	107
Clatsop special wind region[a]	115	120	130	135
Columbia	91	97	103	107
Columbia special wind region[a]	91	97	103	107
Coos	89	95	101	106
Coos special wind region[a]	115	120	130	135
Crook	93	100	106	111
Crook special wind region[a]	93	100	106	111
Curry	88	94	101	105
Curry special wind region[a]	115	120	130	135
Deschutes	93	99	106	110
Deschutes special wind region[a]	93	99	106	110
Douglas	91	97	103	108
Douglas special wind region[a]	115	120	130	135
Gilliam	94	100	107	111
Grant	95	101	108	113
Harney	94	101	108	112
Hood River	92	98	105	109
Hood River special wind region[a]	92	98	105	109

Jackson	90	96	103	107
Jefferson	93	99	106	110
Jefferson special wind region[a]	93	99	106	110
Josephine	89	95	102	106
Klamath	91	98	104	108
Klamath special wind region[a]	91	98	104	108
Lake	93	99	106	111
Lane	91	98	105	110
Lane special wind region[a]	115	120	130	135
Lincoln	90	96	102	106
Lincoln special wind region[a]	115	120	130	135
Linn	92	98	104	108
Malheur	96	102	109	113
Marion	92	98	104	108
Morrow	94	101	108	112
Multnomah	92	98	105	110
Multnomah special wind region[a]	92	98	105	110
Polk	90	97	103	107
Sherman	93	99	106	111
Tillamook	91	96	102	107
Tillamook special wind region[a]	115	120	130	135
Umatilla	95	102	109	113
Union	96	102	109	113
Wallow[a]	97	103	110	115
Wasco	93	99	106	110
Wasco special wind region[a]	93	99	106	110
Washington	91	97	103	107
Wheeler	94	100	107	111
Yamhill	91	97	103	107

Tables notes:

a. Refer to *Figure 1.3.8-1* for mapped special wind regions.

Figure 1.3.8-1 Special Wind Regions - Oregon



a. Sites on the perimeter of the identified special wind regions shall be verified using ASCE 7 Hazard Tool.
b. This portion of the special wind region in Curry County extends 15 miles inland from the Pacific Coast and is not identified on the ASCE 7 Hazard Tool.