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LOADS AND LOAD DISTRIBUTION

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NOTATION

LOADS AND LOAD DISTRIBUTION

A = area of stringer or beam
A_o = area enclosed by centerlines of elements (walls)
B = buoyancy
BR = vehicular braking force
b = width of beam
c_1 = constant related to skew factor
C = stiffness parameter
CE = vehicular centrifugal force
CF = centrifugal force
CR = creep
CT = vehicular collision force
CV = vessel collision force
D = a constant that varies with bridge type and geometry
D = width of distribution per lane
D = dead load
DC = dead load of structural components and nonstructural attachments
DD = downdrag
DW = dead load of wearing surfaces and utilities
d = depth of beam
d = precast beam depth
d_e = distance between the center of exterior beam and interior edge of curb or traffic barrier
E = earth pressure
EH = horizontal earth pressure load
EL = accumulated locked-in force effects resulting from the construction process, including the secondary forces from post-tensioning
EQ = earthquake
ES = earth surcharge load
EV = vertical pressure from deal load of earth fill
e = correction factor
e = eccentricity of a lane from the center of gravity of the pattern of beams
e_g = distance between the centers of gravity of the beam and deck
FR = friction
$f_{(L+I)}$ = live load plus impact bending stress
f_D = the sum of dead load bending stresses
g = a factor used to multiply the total longitudinal response of the bridge due to a single longitudinal line of wheel loads in order to determine the maximum response of a single beam
g = distribution factor
I = impact fraction
I = live load impact
I = moment of inertia
I = moment of inertia of beam

IC = ice load
ICE = ice pressure
IM = vehicular dynamic load allowance
J = St. Venant torsional constant
K = a non-dimensional constant
K_g = longitudinal stiffness parameter
L = live load
L = span of beam
L = simple span length (except cantilevers) when computing truck load moments
L = length of the loaded portion of span from section under consideration to the far reaction when computing shear impact due to truck loads
LF = longitudinal force from live load
LL = vehicular live load
LS = live load surcharge
m = multiple presence factor
N = group number
N_b = number of beams
N_B = number of beams
N_L = number of design lanes
N_L = number of loaded lanes under consideration
N_L = number of traffic lanes
n = modular ratio between beam and deck material
PL = pedestrian live load
Q = total factored load
Q_i = force effect
q_i = specified loads
R = reaction on exterior beam in terms of lanes
R = rib shortening
R_n = nominal resistance
S = beam spacing
S = shrinkage
S = center-to-center beam spacing
S = width of precast member
s = length of a side element
SE = settlement
SF = stream flow pressure
SH = shrinkage
T = temperature
TG = temperature gradient
TU = uniform temperature
t = thickness of an element
t_s = depth of concrete slab

V = distance between axles

W = edge-to-edge width of bridge

W = combined weight on first two truck axles

W = roadway width between curbs

W = overall (edge-to-edge) width of bridge measured perpendicular to the longitudinal beams

W = wind load on structure

WA = water load and stream pressure

WL = wind load on live load

WS = wind load on structure

X_{ext} = horizontal distance from the center of gravity of the pattern of beams to the exterior beam

x = horizontal distance from the center of gravity of the pattern of beams to each beam

β = coefficient, **Table 7.3.1-1**

γ = load factor, **Table 7.3.1-1**

γ_i = load factors specified in **Tables 7.3.2-1** and **7.3.2-2**

η = variable load modifier which depends on ductility, redundancy and operational importance

ϕ = capacity reduction or resistance factor

μ = Poisson's ratio, usually assumed equal to 0.20

θ = skew angle

Loads and Load Distribution

7.1 SCOPE

One main task in bridge design is to collect information on the various permanent and transient loads that may act on a bridge, as well as on how these forces are distributed to the various structural components. This chapter will introduce engineers to the general types of loads to which a bridge is subjected. It presents the load provisions of both the AASHTO *Standard Specifications for Highway Bridges* (referred to as *"Standard Specifications"* in the following) and *AASHTO LRFD Bridge Design Specifications ("LRFD Specifications").* The in-depth discussions will be limited to live load and its distribution to precast, prestressed concrete superstructure systems. Detailed discussion of other load effects, such as seismic forces and soil pressures, are covered in other chapters of the manual. Although both specifications form a consistent set of guidelines for bridge design, the engineer should be aware that many state DOTs have additional requirements for loads, load distribution or load combinations. Such requirements are not discussed in this chapter.

This chapter is based on the provisions of the Standard Specifications, 17th Edition, 2002, and the LRFD Specifications, 2nd Edition, 1998, with all of the Interim Revisions through and including the 2003 Interim Revisions.

7.2 LOAD TYPES

In the design of bridge structure components, the engineer should consider all loads which the component must resist. These forces may vary depending on duration (permanent or transient), direction (vertical, transverse, longitudinal, etc.) and deformation (thermal, shrinkage and creep). Furthermore, the type of effect (bending, shear, axial, etc.) will sometimes influence the magnitude of such forces. A brief description of these forces is detailed below.

7.2.1 Permanent Loads

These loads are sustained by the bridge throughout its life. In general, permanent loads may be subdivided into the following categories.

7.2.1.1 Dead Loads

One of the first tasks in superstructure design is to identify all elements contributing to loads on the beams before composite deck concrete, if any, has cured (some concrete decks are designed to remain noncomposite). These noncomposite dead loads include the beams, weight of the deck slab, haunch, stay-in-place forms and diaphragms.

7.2.1.2 Superimposed Dead Loads

All permanent loads placed on the superstructure after deck curing is completed are usually designated superimposed dead loads. These include the wearing surface, parapets, railings, sidewalk, utilities and signage. In the *LRFD Specifications,* the load factors for wearing surface and utilities are higher than for other dead loads to recognize the increased variability of these loads.

7.2.1.3 Earth Pressures

These forces, which primarily affect substructure elements, are usually considered permanent loads. However, they may occasionally affect the superstructure elements at locations where substructure and superstructure interface (abutment backwall, etc.). Detailed equations are listed in both AASHTO specifications. Generally, these pressures do not affect superstructure design.

7.2.2 Live Loads

7.2.2.1 Gravity Vehicular Live Load

7.2.2.1.1 Number of Design Lanes

Unless otherwise specified, the number of design lanes should be determined by taking the integer part of: roadway width in ft between barriers or curbs divided by 12.0. The loads are assumed to occupy 10.0 ft transversely within a design lane.

7.2.2.1.2 Multiple Presence of Live Load

In view of the improbability of coincident maximum loading in all lanes, the following percentages of live loads are allowed in the STD Article 3.12, when using refined methods of analysis:

One or two loaded lanes	100%
Three lanes	90%
Four (or more) lanes	75%

LRFD Specifications Article 3.6.1.1.2 provides a multiple presence factor, m, which applies when using the refined method [LRFD Articles 4.4 and 4.6.3] or the lever rule for distribution of live load. When considering one loaded lane, the multiple presence factor must be used. For three or more loaded lanes, the multiple presence factor is optional. The extreme live load force effect is determined by considering each possible combination of number of loaded lanes multiplied by the corresponding factor given below. The multiple presence factors are not to be used with the approximate load assignment methods of LRFD Articles 4.6.2.2 and 4.6.2.3 because these factors are already incorporated in the distribution factors for both single and multiple lanes loaded.

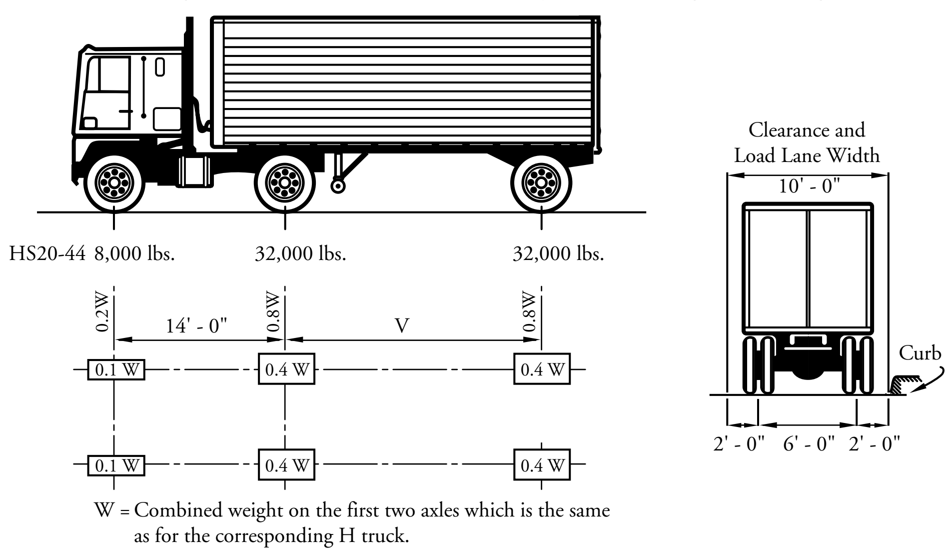
One loaded lane	m = 1.20
Two loaded lanes	m = 1.00
Three loaded lanes	m = 0.85
Four (or more) loaded lanes	m = 0.65

7.2.2.1.3 Highway Live Loading - Standard Specifications

[STD Article 3.7]

There are four classes of notional truck or lane loadings to be used in the design of medium- or long-span superstructures. The majority of bridges are designed for the

Figure 7.2.2.1.3-1
Standard HS Truck



W = Combined weight on the first two axles which is the same as for the corresponding H truck.

V = Variable spacing – 14 feet to 30 feet inclusive. Spacing to be used is that which produces maximum stresses.

Figure 7.2.2.1.3-2
Standard HS Lane Load

* **FOR CONTINUOUS** span bridges an additional concentrated load should be used in determining maximum negative moment only (AASHTO 3.11.3). The second load should be placed in another span of the series. For simple span bridges and for the computation of maximum positive moment in continuous span bridges, a single concentrated load is used as shown.

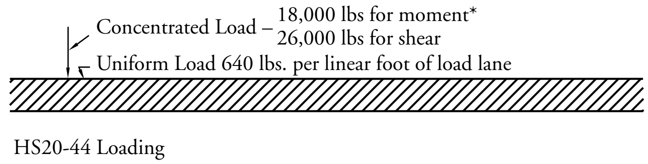
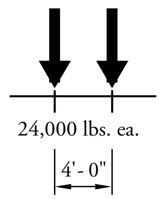


Figure 7.2.2.1.3-3
Tandem Loading
(Alternate Military)



HS20-44 loading shown in **Figure 7.2.2.1.3-1** and **Figure 7.2.2.1.3-2**. The lane loading usually controls beam design for spans longer than approximately 140 ft. For simple spans, the variable distance between rear axles, V, should be set at the 14 ft minimum. In continuous spans, the distance V is varied to create the maximum negative moment. In checking for lane loading in continuous spans, two concentrated loads are used to maximize negative moment (STD Article 3.11.3).

A tandem load, known as the Alternate Military Loading, **Figure 7.2.2.1.3-3**, is also required in the design of U.S. Interstate System bridges. This loading simulates heavy military vehicles and may control beam design in the case of spans shorter than approximately 40 ft.

Some states have begun using the HS25 design loading which represents a 25 percent increase over the standard HS20 truck and lane loadings. Furthermore, in order to provide for potential overweight trucks, some states have developed additional live load configurations known as permit design loadings. These loadings may control the design of prestressed beams and slab design.

7.2.2.1.4 Design Vehicular Live Load - LRFD Specifications

[LRFD Art. 3.6]

The vehicular live loading on bridges, designated as HL-93, consists of a combination of the:

Design truck OR Design tandem
AND
Design lane load

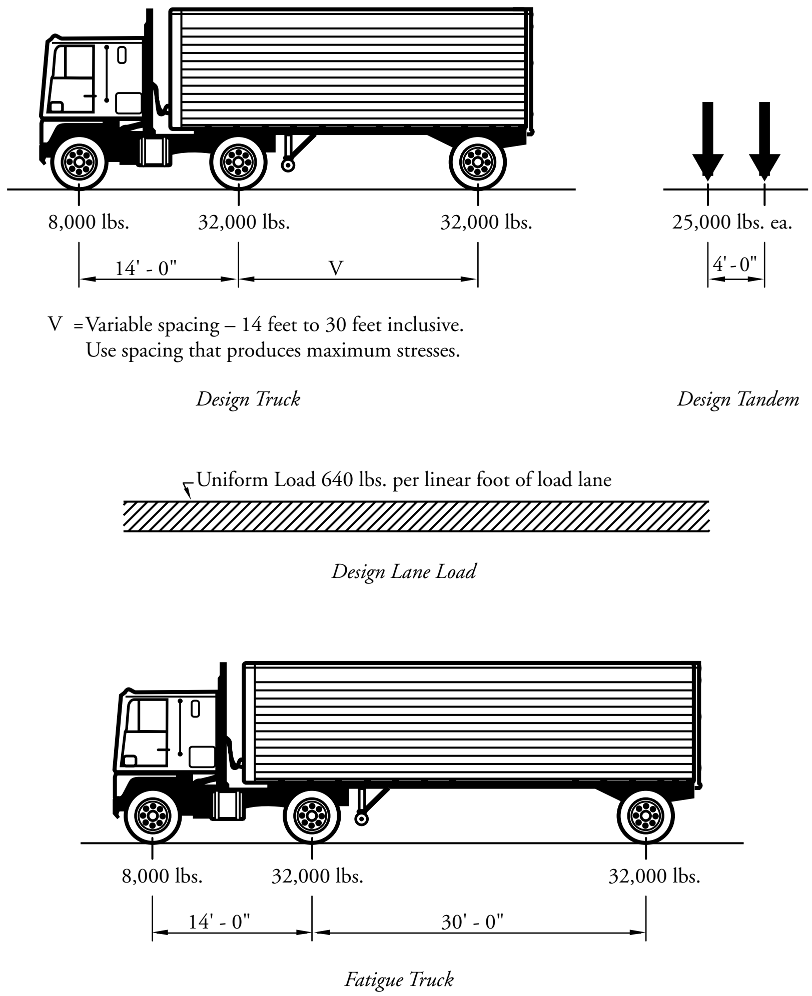
The design truck is the HS20 vehicle used in the *Standard Specifications,* **Figure 7.2.2.1.4-1**. The design tandem consists of a pair of 25.0 kip axles spaced 4.0 ft apart. In either case, the transverse spacing of wheels is taken as 6.0 ft. The design lane load consists of a uniform load of 0.64 klf in the longitudinal direction. It is distributed transversely over a 10.0 ft width.

The extreme force effect for the vehicular live load is the larger of the following:

- The combined effect of the design tandem with the design lane load, or
- The combined effect of one design truck with the variable axle spacing with the design lane load, and
- For continuous members, for both negative moment between points of dead load contraflexure and reaction at interior piers only: the combination of 90% of the effect of two design trucks (spaced a minimum of 50.0 ft between the lead axle of

one and the rear axle of the other truck) with 90% of the effect of the design lane load. The distance between the 32.0 kip axles of each truck shall be taken as 14.0 ft.

Figure 7.2.2.1.4-1
LRFD Design Vehicular Live Loads (HL-93) and Fatigue Load



Axles which do not contribute to the extreme force effect under consideration shall be neglected. Both the design lanes and the position of the 10.0 ft loaded width in each lane shall be positioned to produce extreme force effects. The design truck or tandem shall be positioned transversely so that the center of any wheel load is not closer than 2.0 ft from the edge of the design lane when designing beams.

Unless otherwise specified, the lengths of design lanes, or parts thereof, which contribute to the extreme force effect under consideration shall be loaded with the design lane load. Only those portions of the span which contribute to maximizing the force effect should be loaded. Influence lines can be used to determine those portions of the span which should be loaded for maximum effect.

7.2.2.1.5
Impact or Dynamic Load Allowance

In STD Article 3.8, the amount of the impact allowance or increment is expressed as a fraction of the live load and is determined using the formula:

$$I = 50/(L + 125) \qquad \text{[STD Eq. 3-1]}$$

where

I = impact fraction (maximum 0.30)

L = simple span length (except cantilevers) when computing truck load moments

= for shear due to truck loads: the length of the loaded portion of span from section under consideration to the far reaction. Note: In practice, the use of variable impact to calculate shear for simple and continuous spans is not used, rather the span length of the section under investigation is used.

In *LRFD Specifications* Article 3.6.2, the static effects of the design truck or tandem are multiplied by (1 + IM/100), where IM is the Dynamic Load Allowance as given for different bridge components below: [LRFD Table 3.6.2.1-1]

Deck joints: All limit states 75%

All other components:

Fatigue and Fracture Limit State 15%
All Other Limit States 33%

This dynamic allowance is not applied to the design lane load or to pedestrian loads.

7.2.2.1.6 Fatigue Load

In the *Standard Specifications,* there are no provisions for any special fatigue loading in the case of prestressed beams. In the *LRFD Specifications*, there is a new provision for a single fatigue truck, **Figure** 7.2.2.1-4, but with a constant spacing of 30.0 ft between the 32.0-kip axles. The applicable dynamic load allowance is 15%. When the bridge is analyzed using approximate methods, the distribution factor for one traffic lane is to be used and the force effect is to be divided by 1.20 (except if the lever rule is used).

7.2.2.2 Other Vehicular Forces

7.2.2.2.1 Longitudinal (Braking) Forces

These forces result from vehicles accelerating or braking while traveling over a bridge. Forces are transferred from the wheels to the deck surface.

In the *Standard Specifications,* provision is to be made for a longitudinal force of 5% of the live load (without impact) in all lanes carrying traffic headed in the same direction. The center of gravity of such force is assumed to be located 6 ft above the slab and is transmitted to the substructure through the superstructure. Usually, the effect of braking forces on superstructures is inconsequential.

In the *LRFD Specifications,* the braking forces are taken as the greater of:

• 25% of the axle weights of the truck or tandem
• 5% of the truck plus lane load
• 5% of the tandem plus lane load

This braking force is placed in all lanes carrying traffic headed in the same direction. The multiple presence factor, m, is applicable here.

7.2.2.2.2 Centrifugal Forces

This effect must be considered for bridge structures on horizontal curves. The ratio of this force to the truck (or tandem) axle loads is proportional to the square of the design speed and inversely proportional to the curve radius. This force is applied at 6.0 ft above the roadway surface. Usually, concrete decks resist centrifugal forces within their own plane, and transmit them to the substructure through end diaphragms.

7.2.2.2.3 Vehicular Collision Forces

These forces need to be considered whenever piers or abutments are not adequately protected to prevent vehicle or railway collisions.

7.2.2.3 Pedestrian Loads

In the *Standard Specifications,* the sidewalk area is loaded with a variable uniform load which decreases with beam span. For spans larger than 25 ft, the maximum load is 60 psf.

In LRFD Article 3.6.1.6, a load of 0.075 ksf is applied to all sidewalks wider than 2.0 ft and must be considered with the vehicular live load. For bridges carrying only pedestrian and/or bicycle traffic, the load is set at 0.085 ksf.

The above provisions may be excessive where a significant sidewalk loading is unlikely.

7.2.3 Water and Stream Loads

These forces primarily affect substructure elements and are due to water course-related characteristics. Static water pressure is assumed perpendicular to the surface which is retaining the water, while buoyancy is an uplift force acting on all submerged components.

7.2.3.1 Stream Forces

Stream flow pressure affects the design of piers or supports located in water courses. The average pressure of flowing water on a pier is proportional to the square of water velocity, to the drag coefficient for a specific pier geometry and to the projected pier surface exposed to the design flood.

7.2.3.2 Ice Forces

Floating ice sheets and ice floes on streams cause major dynamic (and static) forces to act on piers in cold weather climates. If clearance is low, the superstructure may also be affected, often with severe damage. Usually, the dynamic force on a pier is a function of ice thickness, ice strength, pier width and inclination of the nose to vertical. Both the *Standard Specifications* and the *LRFD Specifications* contain detailed equations and factors for calculation of stream flow and floating ice loads on piers and supports.

7.2.4 Wind Loads

Wind is a dynamic load. However, it is generally approximated as a uniformly distributed static load on the exposed area of a bridge. This area is taken as the combined surfaces of both superstructure and substructure as seen in elevation (orthogonal to the assumed wind direction). AASHTO loads are based on an assumed "base wind velocity" of 100 mph.

7.2.4.1 Wind Forces - Standard Specifications

Wind forces are applied in a transverse and longitudinal direction at the center of gravity of the exposed region of the superstructure. The specifications provide wind loading values for beam bridges based on the angle of attack (skew angle) of wind forces. Conventional slab-on-stringer bridges with span lengths less than or equal to 125 ft can utilize the following basic loading:

Wind Load on Structure	
Transverse Loading	50 psf
Longitudinal Loading	12 psf
Wind Load on Live Load (Vehicle)	
Transverse Loading	100 plf, based on a long row of passenger cars exposed to a 55 mph wind
Longitudinal Loading	40 plf

The transverse and longitudinal loads are applied simultaneously to both the structure and live load. Also, an upward force acting on the deck must be considered.

7.2.4.2 Wind Forces - LRFD Specifications

A more refined analysis is required, although it follows the same general pattern of "wind pressure on structures" and "wind pressure on vehicles." The specifications also require varying the wind load direction to determine extreme force effects, and the consideration of a vertical upward force acting on the deck (especially when checking overturning of the bridge).

7.2.5 Earthquake Loads and Effects

7.2.5.1 Introduction

These temporary natural forces are assumed to act in the horizontal direction and are dependent on the geographic location of the bridge, the structure dead weight (mass), the ground motion (duration and acceleration), the period of the structural system and type of soil. In some cases, a vertical component of acceleration may have to be considered. These factors enter into the seismic analysis which is a simplification of the actual effects of an earthquake. The bridge response assumes the form of an equivalent static load which is applied to the structure to calculate forces and deformations of bridge elements.

For most pretensioned structures, where the superstructure is not integral with the substructure, earthquake forces do not affect beam design, see Chapter 15 for additional information about seismic design of prestressed beam bridges.

7.2.6 Forces Due to Imposed Deformations

These effects include temperature, creep, differential shrinkage and differential settlement. Some general guidelines are offered in the *LRFD Specifications.* Normally, the difference between the base construction temperature and the temperature range limits in a region is used to calculate thermal deformation effects. Nearly all engineers neglect the effect of temperature gradient in pretensioned multi-beam bridges. This practice has been used for over 40 years with good performance. For other types of bridges, judgment and experience should be used in deciding to consider the effects of temperature gradient. Where appropriate, the effects of creep, differential shrinkage and differential settlements should be considered.

7.3 LOAD COMBINATIONS AND DESIGN METHODS

Vehicle live loads may act on a bridge simultaneously with other live loads. The design engineer is responsible to size and reinforce the structural components to safely resist the possible combinations of loads which may act on a bridge. Therefore, the *Standard Specifications* and *LRFD Specifications* contain load combinations, subdivided into various groups, which represent probable simultaneous loadings on the structure. In theory, all structural elements should be designed to resist all groups of loads. In practice, though, many of the groups do not control the design and may be disregarded.

There are two principal methods of design:

1. Service Load Design (Allowable Stress Design)
   In this method, the allowable stress is defined as the material strength (stress) reduced by a suitable factor of safety. The total stress caused by load effects must not exceed this allowable stress. This is expressed in the following relationship:

   $$f_{total} \leq f_{allowable} \qquad \text{(Eq. 7.3-1)}$$

2. Strength Design (Load Factor Design)
   In this method, the general relationship is defined as follows:

Provided Strength ≥ Required Strength
OR
Factored Resistance ≥ Factored Moment, Shear or Axial Force (Eq. 7.3-2)

The nominal resistance of a member, R_n, is computed using procedures given in the specifications. This value is then modified by a resistance factor, ϕ, appropriate for the specific conditions of design to obtain the provided strength. The load effects, Q_i, are usually calculated using conventional elastic analysis procedures. These are then modified by the specified load factors, γ_i, to obtain the required strength. In a concise form, Equation 7.3-2 can be expressed as follows:

$$\phi R_n \geq \sum \gamma_i Q_i \qquad \text{(Eq. 7.3-3)}$$

where Q_i is the load effect.

7.3.1 Standard Specifications

Group loading combinations for Service Load Design and Load Factor Design are given by: [STD Art 3.22.1]

$$\begin{aligned}\text{Group}(N) = \gamma\ [&\beta_D D + \beta_L (L + I) + \beta_C CF + \beta_E E \\ &+ \beta_B B + \beta_S SF + \beta_W W + \beta_{WL} WL \\ &+ \beta_L LF + \beta_R (R + S + T) + \beta_{EQ} EQ + \beta_{ICE} ICE]\end{aligned} \qquad \text{[STD Eq. 3-10]}$$

where

N = group number
γ = load factor, see **Table 7.3.1-1**
β = coefficient, see **Table 7.3.1-1**
D = dead load
L = live load
I = live load impact
E = earth pressure
B = buoyancy
W = wind load on structure
LF = longitudinal force from live load
CF = centrifugal force
R = rib shortening
S = shrinkage
T = temperature
EQ = earthquake
SF = stream flow pressure
ICE = ice pressure
WL = wind load on live load

For Service Load Design, the percentage of the basic unit stress for the various groups is given in **Table 7.3.1-1**. In the design of pretensioned flexural elements in the superstructure, such as stringers or beams, the design is governed by the Group I loading combination which may be stated as:

$$f_D + f_{(L+I)} \leq f_{allowable} \qquad \text{(Eq. 7.3.1-2)}$$

where

f_D = the sum of dead load bending stresses

$f_{(L+I)}$ = live load plus impact bending stress

Table 7.3.1-1
Table of Coefficients γ and β–Standard Specifications

Col. No.		1	2	3	3A	4	5	6	7	8	9	10	11	12	13	14	
GROUP		γ	β FACTORS													%	
			D	$(L+I)_n$	$(L+I)_p$	CF	E	B	SF	W	WL	LF	R+S+T	EQ	ICE		
SERVICE LOAD	I	1.0	1	1	0	1	β_E	1	1	0	0	0	0	0	0	100	
	IA	1.0	1	2	0	0	0	0	0	0	0	0	0	0	0	150	
	IB	1.0	1	0	1	1	β_E	1	1	0	0	0	0	0	0	**	
	II	1.0	1	0	0	0	1	1	1	1	0	0	0	0	0	125	
	III	1.0	1	1	0	1	β_E	1	1	0.3	1	1	0	0	0	125	
	IV	1.0	1	1	0	1	β_E	1	1	0	0	0	1	0	0	125	
	V	1.0	1	0	0	0	1	1	1	1	0	0	1	0	0	140	
	VI	1.0	1	1	0	1	β_E	1	1	0.3	1	1	1	0	0	140	
	VII	1.0	1	0	0	0	1	1	1	0	0	0	0	1	0	133	
	VIII	1.0	1	1	0	1	1	1	1	0	0	0	0	0	1	140	
	IX	1.0	1	0	0	0	1	1	1	1	0	0	0	0	1	150	
	X	1.0	1	1	0	0	β_E	0	0	0	0	0	0	0	0	100	Culvert
LOAD FACTOR DESIGN	I	1.3	β_D	1.67*	0	1.0	β_E	1	1	0	0	0	0	0	0	Not Applicable	
	IA	1.3	β_D	2.20	0	0	0	0	0	0	0	0	0	0	0		
	IB	1.3	β_D	0	1	1.0	β_E	1	1	0	0	0	0	0	0		
	II	1.3	β_D	0	0	0	β_E	1	1	1	0	0	0	0	0		
	III	1.3	β_D	1	0	1	β_E	1	1	0.3	1	1	0	0	0		
	IV	1.3	β_D	1	0	1	β_E	1	1	0	0	0	1	0	0		
	V	1.25	β_D	0	0	0	β_E	1	1	1	0	0	1	0	0		
	VI	1.25	β_D	1	0	1	β_E	1	1	0.3	1	1	1	0	0		
	VII	1.3	β_D	0	0	0	β_E	1	1	0	0	0	0	1	0		
	VIII	1.3	β_D	1	0	1	β_E	1	1	0	0	0	0	0	1		
	IX	1.20	β_D	0	0	0	β_E	1	1	1	0	0	0	0	1		Culvert

$(L+I)_n$ - Live load plus impact for AASHTO Highway H or HS loading

$(L+I)_p$ - Live load plus impact consistent with overload criteria of the operation agency

* and ** - Refer to *Standard Specifications* for explanation

For Load Factor Design of pretensioned stringers or beams, the section design is also governed by Group I requirements:

Provided Strength ≥ 1.3[D + 1.67(L+I)] (Eq. 7.3.1-3)

One exception is the case of an outside roadway beam when the combination of sidewalk live load and traffic live load (plus impact) may govern the design. Then the load factor 1.67 may be replaced by 1.25, provided the section capacity is not less than that required for traffic live load only using $\beta_L = 1.67$.

In many states, structures are required to be analyzed for an overload that is selected by the particular transportation department. This load is then applied in Group IB as defined in **Table 7.3.1-1**, which may or may not control the design.

7.3.2 LRFD Specifications

The total factored load, Q, is given by:

$$Q = \eta \Sigma \gamma_i q_i \qquad \text{(Eq. 7.3.2-1)}$$

where

η = variable load modifier which depends on ductility, redundancy and operational importance. Its value is often set by state DOTs

q_i = specified loads

γ_i = load factors specified in **Tables 7.3.2-1** and **7.3.2-2**

Table 7.3.2-1
Load Combinations and Load Factors, LRFD Specifications [LRFD Table 3.4.1-1]

Load Combination / Limit State	DC DD DW EH EV ES EL	LL IM CE BR PL LS	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These at a Time			
										EQ	IC	CT	CV
STRENGTH-I	γ_p	1.75	1.00	–	–	1.00	0.50/1.20	γ_{TG}	γ_{SE}	–	–	–	–
STRENGTH-II	γ_p	1.35	1.00	–	–	1.00	0.50/1.20	γ_{TG}	γ_{SE}	–	–	–	–
STRENGTH-III	γ_p	–	1.00	1.40	–	1.00	0.50/1.20	γ_{TG}	γ_{SE}	–	–	–	–
STRENGTH-IV EH, EV, ES, DW	γ_p	–	1.00	–	–	1.00	0.50/1.20	–	–	–	–	–	–
DC ONLY	1.5												
STRENGTH-V	γ_p	1.35	1.00	0.40	0.40	1.00	0.50/1.20	γ_{TG}	γ_{SE}	–	–	–	–
EXTREME EVENT-I	γ_p	γ_{EQ}	1.00	–	–	1.00	–	–	–	1.00	–	–	–
EXTREME EVENT-II	γ_p	0.50	1.00	–	–	1.00	–	–	–	–	1.00	1.00	1.00
SERVICE-I	1.00	1.00	1.00	0.30	0.30	1.00	1.00/1.20	γ_{TG}	γ_{SE}	–	–	–	–
SERVICE-II	1.00	1.30	1.00	–	–	1.00	1.00/1.20	–	–	–	–	–	–
SERVICE-III	1.00	0.80	1.00	–	–	1.00	1.00/1.20	γ_{TG}	γ_{SE}	–	–	–	–
SERVICE-IV	1.00	–	1.00	0.70	–	1.00	1.00	–	1.00	–	–	–	–
FATIGUE-LL, IM & CE ONLY	–	0.75	–	–	–	–	–	–	–	–	–	–	–

For notes on γ_p, γ_{EQ}, γ_{TG} and γ_{SE}, refer to *LRFD Specifications*

Table 7.3.2-2
Load Factors for Permanent Loads, γ_p, LRFD Specifications
[LRFD Table 3.4.1-2]

Type of Load	Load Factor	
	Maximum	Minimum
DC: Component and Attachments	1.25	0.90
DD: Downdrag	1.80	0.45
DW: Wearing Surfaces and Utilities	1.50	0.65
EH: Horizontal Earth Pressure		
• Active	1.50	0.90
• At-Rest	1.35	0.90
EL: Locked-in Erection Stresses	1.00	1.00
EV: Vertical Earth Pressure		
• Overall Stability	1.00	N/A
• Retaining Walls and Abutments	1.35	1.00
• Rigid Buried Structure	1.30	0.90
• Rigid Frames	1.35	0.90
• Flexible Buried Structures other than Metal Box Culverts	1.95	0.90
• Flexible Metal Box Culverts	1.50	0.90
ES: Earth Surcharge	1.50	0.75

Components (and connections) of a bridge structure must satisfy the applicable combinations of factored extreme force effects as specified at each of the limit states. The following load designations are used:

- Permanent Loads

DD = downdrag

DC = dead load of structural components and nonstructural attachments

DW = dead load of wearing surfaces and utilities

EL = accumulated locked-in force effects resulting from the construction process, including the secondary forces from post-tensioning

EH = horizontal earth pressure load

ES = earth surcharge load

EV = vertical pressure from dead load of earth fill

- Transient Loads

BR = vehicular braking force

CE = vehicular centrifugal force

CR = creep

CT = vehicular collision force

CV = vessel collision force

EQ = earthquake

FR = friction

IC = ice load

IM = vehicular dynamic load allowance

LL = vehicular live load

LS = live load surcharge

PL = pedestrian live load

SE = settlement

SH = shrinkage

TG = temperature gradient

TU = uniform temperature

WA = water load and stream pressure

WL = wind on live load

WS = wind load on structure

As has always been the case, the owner or designer may determine that not all of the loads in a given load combination apply to the situation being investigated. The various applicable load factors are in **Tables 7.3.2-1** and **7.3.2-2**. The minimum load factors are especially important in the negative moment regions of continuous beams.

The factors must be selected to produce the total extreme factored force effect. For each load combination, both positive and negative extremes must be investigated. In load combinations where one force effect decreases the effect of another, the minimum value is applied to the load reducing the force effect. For permanent force effects, the load factor (maximum or minimum) which produces the more critical combination is selected from **Table 7.3.2-2**.

The design of pretensioned superstructure beams using the *LRFD Specifications* usually consists of satisfying the requirements of Service I, Service III and Strength I load combinations. Use of the new larger vehicular live load for working stress design of prestressed concrete members would result in over-conservative designs. Also, since no significant cracking has been observed in existing bridges that were designed for the relatively lower loads of the *Standard Specifications,* the Service III load combination was introduced. Service III specifies a load factor of 0.80 to reduce the effect of live load at the service limit state. This combination is only applicable when checking allowable tensile stresses in prestressed concrete superstructure members. Service I is used when checking compressive stresses only. The load combination Strength I is used for design at the strength limit state. Other load combinations for the strength and extreme event limit states are not considered here, but may be required by specific agencies or DOTs—such as Strength II combination for permit vehicles.

The various load combinations applicable to prestressed beams and substructures (Service IV) and shown in **Table 7.3.2-1** are described below.

STRENGTH I - Basic load combination relating to the normal vehicular use of the bridge without wind.

STRENGTH II - Load combination relating to the use of the bridge by permit vehicles without wind. If a permit vehicle is traveling unescorted, or if control is not provided by the escorts, the other lanes may be assumed to be occupied by the vehicular live load herein specified. For bridges longer than the permit vehicle, addition of the lane load, preceding and following the permit load in its lane, should be considered.

SERVICE I - Load combination relating to the normal operational use of the bridge with 55 mph wind. All loads are taken at their nominal values and extreme load conditions are excluded. Compression in prestressed concrete components is investigated using this load combination.

SERVICE III - Load combination relating only to prestressed concrete superstructures with the primary objective of crack control. Tensile stress in prestressed concrete superstructure members is investigated using this load combination.

SERVICE IV - Load combination relating only to tension in prestressed concrete substructures with the primary objective of crack control. Tensile stress in prestressed concrete substructure members is investigated using this load combination.

FATIGUE - Fatigue and fracture load combination relating to gravitational vehicular live load and dynamic response. Consequently BR, LS and PL loads need not be considered. The load factor is applied to a single design truck.

7.4 LIVE LOAD DISTRIBUTION - STANDARD SPECIFICATIONS

7.4.1 Introduction and Background

The following sections present several approximate formulas for live load distribution factors taken from the *Standard Specifications.* A wheel load is defined as one half of a full lane (or truck) load. These procedures may be used in lieu of refined methods, such as the finite element or grillage analysis (see Section 7.6). They utilize the concept of a wheel load distribution factor, g, for bending moment and shear in interior beams given by:

$g = S/D$, or (Eq. 7.4.1-1)

$g =$ function of: number of lanes and beams and S/L (Eq. 7.4.1-2)

where

g = a factor used to multiply the total longitudinal response of the bridge due to a single longitudinal line of wheel loads in order to determine the maximum response of a single beam.

S = center-to-center beam spacing, ft

D = a constant that varies with bridge type and geometry, ft

L = span length, ft

The live load bending moment for each interior beam, is determined by applying to the beam, the fraction of a wheel load as determined from the applicable equation. No longitudinal distribution of loads is assumed. Except for the case of multi-beam decks, the live load moment for exterior beams is determined by applying to the beam the reaction of the wheel load obtained by assuming the flooring to act as a simple span between the beams.

The approximate equations described below are suitable for the design of normal (non-skewed) bridge decks. There are no guidelines for adjustments in the case of skews. Designers should be aware that a major shortcoming of the current specifications is that the piecemeal changes that have taken place over the last four decades have led to inconsistencies and general conservatism in the load distribution criteria.

7.4.2 Distribution Factors for I-Beams and Bulb-Tees

[STD Arts. 3.23.2.2 and 3.23.2.3.1.2]

When a bridge is designed for two or more traffic lanes and the beam spacing, $S \leq 14$ ft, the distribution factor for interior beams is determined by:

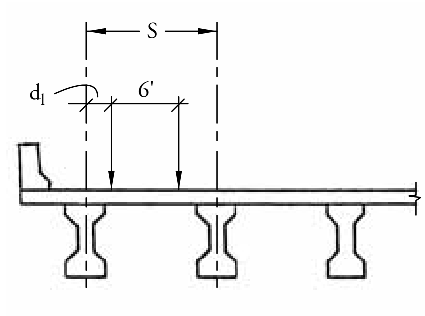
$g = S/5.5$ (Eq. 7.4.2-1)

If a bridge is narrow and designed for only one traffic lane then:

$g = S/7.0$ (Eq. 7.4.2-2)

Eq. (7.4.2-1) is credited to Newmark and has not changed until the introduction of the *LRFD Specifications.* Although composite double tee decks are not specifically listed in STD Table 3.23.1, it has been a common practice to use this equation with S equal to the stem or web spacing.

Figure 7.4.2-1
Distribution Factor for Exterior Beam



For exterior beams, the distribution factor is determined using the requirements of STD Article 3.23.2.3.1.2. These provisions, which are often called the "Lever Rule," are best explained by an example, as shown in **Figure 7.4.2-1**. The bridge deck is modeled as a simple span with an overhang. The fraction of a wheel load carried by exterior beams is determined by summing moments about the center of the first interior beam.

$$g = \{(S - d_1)/S\} + \{(S - d_1 - 6)/S\} \qquad \text{(Eq. 7.4.2-3)}$$

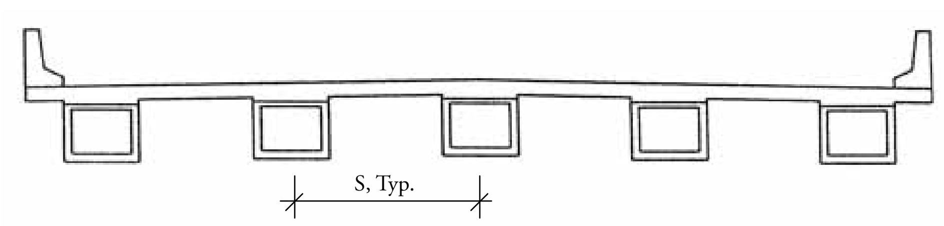
If the overhang is wide enough to accommodate a wheel position outside of the center of the exterior beam, then d_1 is negative in Eq. (7.4.2-3).

7.4.3 Distribution Factors for Spread Box Beams

[STD Art. 3.28]

The live load bending moment for each interior beam in a spread box beam superstructure is computed by applying to the beam the fraction of a wheel load **Figure 7.4.3-1** determined by the following equation:

Figure 7.4.3-1
Typical Cross-Section of a Spread Box Beam Bridge Deck



$$g = \frac{2N_L}{N_B} + k\left(\frac{S}{L}\right) \qquad \text{[STD Eq. 3-33]}$$

where

[STD Eq. 3-34]

$k = 0.07\,W - N_L(0.10N_L - 0.26) - 0.20N_B - 0.12$

N_L = number of design traffic lanes

N_B = number of beams ($4 \le N_B \le 10$)

S = beam spacing, ft ($6.57 \le S \le 11.00$)

L = span length, ft

W = roadway width between curbs, ft ($32 \le W \le 66$)

These two equations are based on a statistical correlation with the results of finite element analyses covering some 300 cases (Motarjemi, 1969). However, no multi-lane reduction factor was considered. If a spread box beam bridge is designed for a two-lane roadway, there is probably little advantage in using a refined analysis method.

For exterior beams, the lever rule discussed in the preceding section is used.

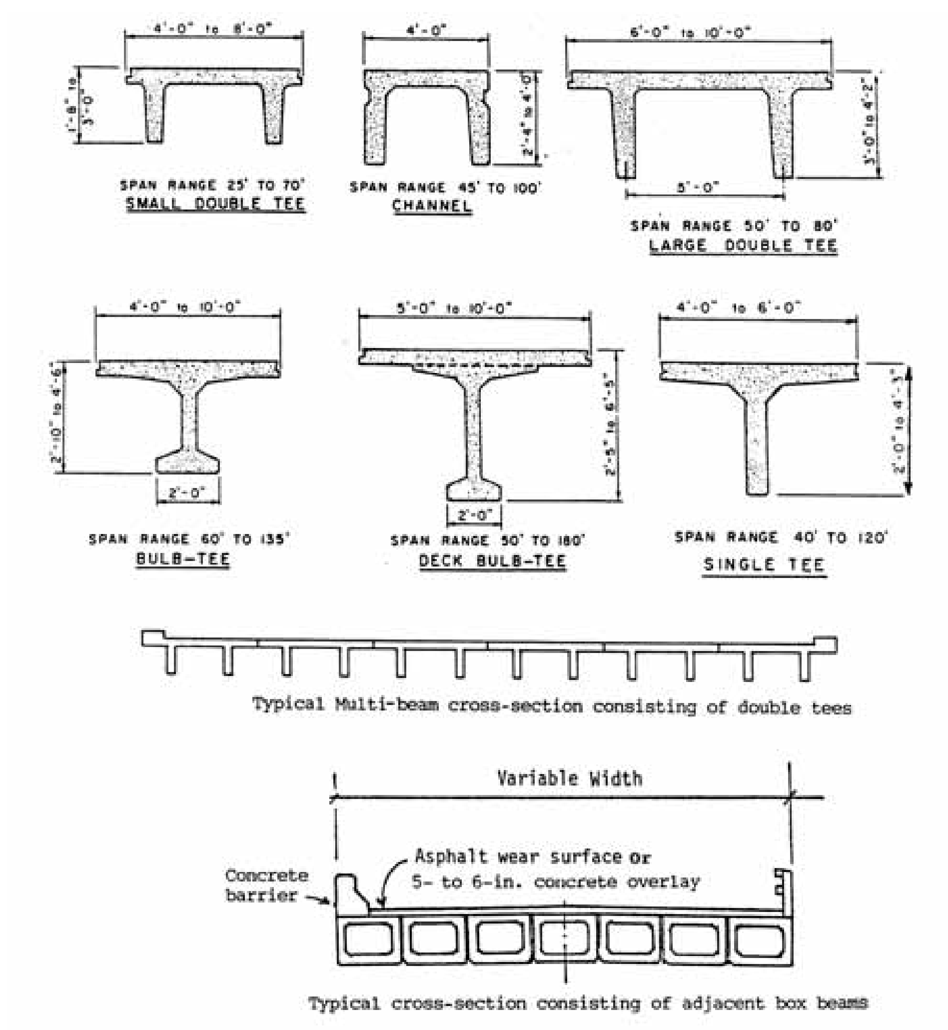
7.4.4 Distribution Factors for Adjacent Box Beams and Multi-Beam Decks

[STD Art. 3.23.4]

A multi-beam bridge deck consists of precast or prestressed concrete beams that are placed side-by-side on the supports. Adjacent box beams, channels, double tees, deck bulb-tees and solid or hollow slabs (**Figure 7.4.4-1**) fall under this category. A structural concrete or asphalt overlay may be required by state or local practice.

In general, the interaction between beams is developed by continuous longitudinal shear keys used in combination with metal tie plates or lateral bolting or prestressing. Full-depth rigid end diaphragms for channel, single tee, or multi-stem beams are required by the specifications. However, midspan diaphragms often are not required by local practice. It has been traditional in some states to use steel cross frames or K-braces in lieu of cast-in-place end diaphragms.

Figure 7.4.4-1
Adjacent Box Beam and Multi-Beam Stemmed Sections with Approximate Geometries and Span Ranges



The live load distribution factor for interior or exterior beams is given by:

$g = S/D$ [STD Eq. 3-11]

where

S = width of precast member, ft

$D = (5.75 - 0.5N_L) + 0.7N_L(1 - 0.2C)^2$ [STD Eq. 3-12] where

N_L = number of traffic lanes

C = K(W/L) for W/L < 1

= K for W/L ≥ 1 [STD Eq. 3-13]

where

W = overall (edge-to-edge) width of bridge measured perpendicular to the longitudinal beams, ft

L = span length measured parallel to longitudinal beams, ft; for beams with cast-in-place end diaphragms, use the length between diaphragms

$K = [(1 + \mu)I/J]^{0.5}$ (Eq. 7.4.4-1)

where

I = moment of inertia

J = St. Venant torsional constant

μ = Poisson's ratio for beams

For preliminary design, approximate values of K are provided in the LRFD Specification.

7.5 SIMPLIFIED DISTRIBUTION METHODS—LRFD SPECIFICATIONS

The following sections will focus on precast, prestressed concrete bridges using box, I-, bulb-tee or multi-stem beam cross sections. The majority of the live load distribution formulas in the *LRFD Specifications* are entirely new and are based on an NCHRP project (Zokaie, 1991). However, as with any new technology, revisions and clarifications are inevitable.

7.5.1 Background

Advanced computer technology and refined procedures of analysis—such as the finite element method—constitute the basis for development of the approximate formulas given in the *LRFD Specifications.* First, a large database of more than 800 actual bridges was randomly compiled from various states to achieve national representation. Then average bridges were obtained for each slab and beam category. Finally, refined analyses were implemented on selected bridges from each group.

Approximate formulas were developed to capture the variation of load distribution factors with each of the dominant geometric and material parameters. It was assumed that the effect of each parameter could be modeled by an exponential function of the form ax^b, where 'x' is the value of the given parameter (span, spacing, box depth, etc.) and 'b' is an exponent to be defined. The final distribution factor is given in the following general format which is based on a multiple regression analysis:

$D.F. = A + B(x)^b(y)^c(z)^d. . . .$ (Eq. 7.5.1-1)

Although the multiple exponential procedure worked well in many cases, it is inherently conservative in general because of several assumptions made during its development, such as:

- midspan diaphragms were disregarded thereby increasing moments in interior beams and reducing moments in exterior beams
- multi-lane presence factors were higher than the final factors used in the *LRFD Specifications* (See Sec. 7.2.2.1.2)
- the width of the concrete parapet (1'-6" or 1'-9") was often neglected, thereby increasing the load factors for the first two beams.

Furthermore, in order to assure conservative results, the constants in the formulas were adjusted so that the ratio of the average value computed using the approximate method to the accurate distribution factor was always larger than 1.0.

7.5.1.1 Introduction

LRFD Article 4.6.2.2 presents approximate live load distribution factors that may be used when a refined method is not used. Different structure types are identified descriptively and graphically in **LRFD Table 4.6.2.2.1-1** to assist the designer in using the correct distribution factor for the structure being designed. There are 12 structure types included in the table, eight of which utilize precast concrete.

Longitudinal joints connecting adjacent members are shown for five of the types of structures. If adjacent beams are "sufficiently connected to act as a unit," they may be considered to act monolithically. Those types without composite structural concrete topping may require transverse post-tensioning. (See Section 7.5.5.)

The live load distribution factors for beam-slab bridges presented in the *LRFD Specifications* are significantly different from those used in the *Standard Specifications.* The differences between the two specifications include:

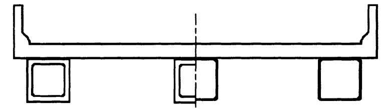
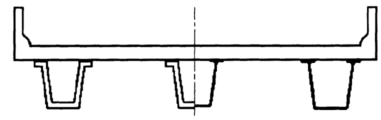
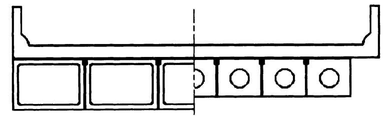
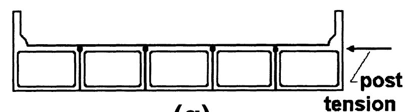
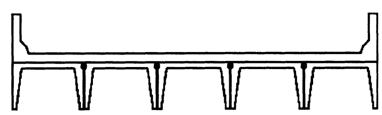
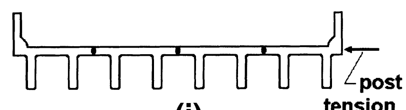
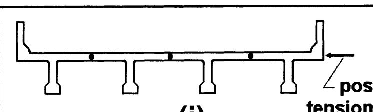
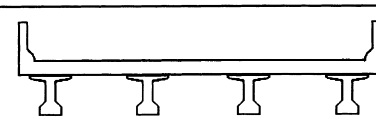
- There are now eight types of distribution factors for different types of structures and connections between beams, four of which apply to precast concrete sections.
- Separate distribution factors are provided for moment and shear in interior beams.
- Distribution factors for moment and shear in exterior beams are computed either by modifying the distribution factor for interior beams or by using the lever rule.
- Where rigid intermediate diaphragms are provided, the load on the exterior beams must also be checked assuming that the cross section remains straight, deflecting and rotating as a rigid body.
- The effect of multiple lane loading is included in the distribution factors. Therefore, multiple presence factors should not be used unless a refined analysis method is used or the lever arm procedure is required.
- For skewed bridges, distribution factors for moment and shear are adjusted using factors given for different structure types in appropriate tables.

The following general conditions must be satisfied for the approximate distribution factor equations to be used:

- the width of deck is constant
- the number of beams is not less than three, four or five depending on the case
- beams are parallel and have approximately the same stiffness
- unless otherwise specified, the roadway part of the overhang, d_e, does not exceed 3.0 ft
- curvature in plan is less than the specified limit
- the cross-section is consistent with one of the cross-sections shown in **Figure 7.5.1-1**
- for beams, other than box beams, used in multi-beam decks with shear keys:
  - deep, rigid end diaphragms are required
  - if the stem spacing of stemmed beams is less than 4.0 ft or more than 10.0 ft, a refined analysis is to be used

All formulas in the tables in the *LRFD Specifications* provide the live load distribution per lane. Where roadway width is larger than 20 ft, the formulas for "Two or More Design Lanes Loaded" must be used for the following limit states: Strength I, Service I and Service III. For the Strength II limit state, the same distribution factor may be used. However, results can be overly conservative if the permit load is heavy. To circumvent this

Figure 7.5.1-1
Common Deck Superstructures
[LRFD Table 4.6.2.2.1-1]

SUPPORTING COMPONENTS	TYPE OF DECK	TYPICAL CROSS-SECTION
Closed Steel or Precast Concrete Boxes	Cast-in-place concrete slab	 (b)
Open Steel or Precast Concrete Boxes	Cast-in-place concrete slab, precast concrete deck slab	 (c)
Precast Solid, Voided or Cellular Concrete Boxes with Shear Keys	Cast-in-place concrete overlay	 (f)
Precast Solid, Voided or Cellular Concrete Box with Shear Keys and with or without Transverse Post-Tensioning	Integral concrete	 (g)
Precast Concrete Channel Sections with Shear Keys	Cast-in-place concrete overlay	 (h)
Precast Concrete Double Tee Section with Shear Keys and with or without Transverse Post-Tensioning	Integral concrete	 (i)
Precast Concrete Tee Section with Shear Keys and with or without Transverse Post-Tensioning	Integral concrete	 (j)
Precast Concrete I or Bulb-Tee Sections	Cast-in-place concrete, precast concrete	 (k)

situation, where it controls the design, the engineer can use a refined method as discussed in Section 7.6. Finally, when checking for fatigue, the formulas for "One Design Lane Loaded" must be used. In the following sections, two loaded lanes will be assumed.

Specific limitations for each equation are given in the tables. These must also be satisfied before the equations can be used.

Where bridges meet the specified conditions, permanent superimposed loads, such as parapets and wearing surface, may be distributed equally between all beams in the bridge.

The live load distribution factors specified herein may also be used for permit and rating vehicles whose overall width is comparable to the width of the design truck.

7.5.2 Approximate Distribution Formulas for Moments (Two Lanes Loaded)

[LRFD Art. 4.6.2.2]
[LRFD Table 4.6.2.2.2b-1]
[LRFD Table 4.6.2.2.2d-1]

The following notation is used in the distribution factor equations:

A = area of stringer, or beam, in.^{2}

b = width of beam, in.

C = stiffness parameter = $K(W/L)$

d = depth of beam, in.

d_e = distance between the center of exterior beam and interior edge of curb or traffic barrier, ft

D = width of distribution per lane, ft

e = correction factor

g = distribution factor

J = St. Venant torsional constant, in.^{4}

K = a non-dimensional constant

K_g = longitudinal stiffness parameter, in.^{4}

L = span of beam, ft

N_b = number of beams

N_L = number of design lanes

S = spacing of beams or webs, ft

t_s = depth of concrete slab, in.

W = edge-to-edge width of bridge, ft

θ = skew angle, deg

μ = Poisson's ratio, usually assumed equal to 0.20

The longitudinal stiffness parameter, K_g, is taken as:

$$K_g = n(I + Ae_g^2) \qquad \text{[LRFD Eq. 4.6.2.2.1-1]}$$

where

n = modular ratio between beam and deck materials, generally ≥ 1

I = moment of inertia of beam, in.^{4}

e_g = distance between the centers of gravity of the beam and deck, in.

7.5.2.1 I-Beam, Bulb-Tee, or Single or Double Tee Beams with Transverse Post-Tensioning

The applicable live load distribution factor equation for interior beams [**Figure 7.5.1-1, types (i), (j)** and (**k**)] is:

$$g = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^{\,3}}\right)^{0.1} \qquad \text{(Eq. 7.5.2.1-1)}$$

The only practical conditions affecting applicability of this equation are that N_b must be equal to or larger than 4 and $10{,}000 \le K_g \le 7{,}000{,}000$. The latter limit may be exceeded in the case of I-beams that are 96 in. deep or more. For preliminary design, the engineer may assume that $(K_g/12.0Lt_s^{3})^{0.1} \doteq 1.10$, which is an average value obtained from a large database.

The equation for exterior beams without midspan diaphragms is:

$$g = eg_{interior} \qquad \text{(Eq. 7.5.2.1-2)}$$

where $e = 0.77 + (d_e/9.1) \ge 1.0$ (Eq. 7.5.2.1-2a)

If rigid midspan diaphragms are used in the cross-section, an additional check is required using an interim, conservative procedure for I- and bulb-tee beam sections and applying the related multiple presence factor, m:

$$g \ge R = \frac{N_L}{N_b} + \frac{X_{ext}\sum^{N_L} e}{\sum^{N_b} x^2} \qquad \text{(Eq. 7.5.2.1-3)}$$

[LRFD Eq. C4.6.2.2.2d-1]

where

- R = reaction on exterior beam in terms of lanes
- N_L = number of loaded lanes under consideration
- N_b = number of beams
- e = eccentricity of a lane from the center of gravity of the pattern of beams, ft
- x = horizontal distance from the center of gravity of the pattern of beams to each beam, ft
- X_{ext} = horizontal distance from the center of gravity of the pattern of beams to the exterior beam, ft

7.5.2.2 Open or Closed Precast Spread Box Beams with Cast-In-Place Deck

The live load flexural moment for interior beams [**Figure 7.5.1-1, types (b)** and (**c**)] may be determined by applying the following lane fraction:

$$g = \left(\frac{S}{6.3}\right)^{0.6}\left(\frac{Sd}{12.0L^2}\right)^{0.125} \qquad \text{(Eq. 7.5.2.2-1)}$$

where d = precast beam depth.

This formula is subject to two practical limitations: $N_b \ge 3$ and $6.0 \le S \le 18.0$ ft. The other geometric conditions are usually met.

The corresponding formula for exterior beams is:

$$g = eg_{interior} \qquad \text{(Eq. 7.5.2.2-2)}$$

where $e = 0.97 + (d_e/28.5)$ (Eq. 7.5.2.2-2a)

Equation (7.5.2.1-3) must also be checked in the case of rigid midspan diaphragms.

7.5.2.3 Adjacent Box Beams with Cast-In-Place Overlay or Transverse Post-Tensioning

The applicable distribution factor equation for interior beams [**Figure 7.5.1-1, types (f)** and **(g)**], is given by:

$$g = k\left(\frac{b}{305}\right)^{0.6}\left(\frac{b}{12.0L}\right)^{0.2}\left(\frac{I}{J}\right)^{0.06} \quad \text{(Eq. 7.5.2.3-1)}$$

where $k = 2.5(N_b)^{-0.2} \geq 1.5$ (Eq. 7.5.2.3-1a)

In a preliminary design situation one may assume $(I/J)^{0.06} = 1.0$. These equations are limited to box beam widths not exceeding 5.0 ft and to span lengths $L \leq 120$ ft.

The bending moment for exterior beams is determined by applying the following lane fraction:

$$g = eg_{interior} \quad \text{(Eq. 7.5.2.3-2)}$$

where $e = 1.04 + (d_e/25)$, $d_e \leq 2.0$ (Eq. 7.5.2.3-2a)

7.5.2.4 Channel Sections, or Box or Tee Sections Connected by "Hinges" at Interface

For interior beams, [**Figure 7.5.1-1, types (g)**, **(h)**, **(i)** and **(j)**], the applicable formula for the distribution factor, regardless of the number of loaded lanes, is:

$$g = S/D \quad \text{(Eq. 7.5.2.4-1)}$$

where

$D = 11.5 - N_L + 1.4N_L(1 - 0.2C)^2$ when $C \leq 5$ (Eq. 7.5.2.4-1a)

$D = 11.5 - N_L$ when $C > 5$ (Eq. 7.5.2.4-1b)

where

$C = K(W/L) \leq K$ (Eq. 7.5.2.4-1c)

where $K = [(1 + \mu)I/J]^{0.5}$ (Eq. 7.5.2.4-1d)

LRFD Table 4.6.2.2.2b-1 suggests values of K for preliminary design.

The specified procedure for exterior beams is simply the 'Lever Rule' in conjunction with the multiple presence factor, m (see Section 7.2.2.1.2). However, this presents some interpretation problems regarding how many lanes should be loaded (say 2, 3 or 4 lanes if roadway width is 48 ft or more). Until this question is resolved, it is prudent to at least assign the same live load distribution factor for exterior beams as for interior beams, which is the approach used in the *Standard Specifications.* Furthermore, LRFD Article 2.5.2.7 requires that, in general, the load carrying capacity of an exterior beam be not less than the one for an interior beam.

7.5.3 Approximate Distribution Formulas for Shear (Two Lanes Loaded)

The live load shear for interior and exterior beams is determined by applying the lane fractions specified for the categories below. The shear distribution factors are normally higher than the moment factors for the same cross-section and span.

7.5.3.1 I-Beam, Bulb-Tee, or Single or Double Tee Beams with Transverse Post-Tensioning

The applicable live load distribution factor equation for interior beams, [**Figure 7.5.1-1, types (i)**, **(j)** and **(k)**], is:

$$g = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^{2.0} \quad \text{(Eq. 7.5.3.1-1)}$$

The only practical limitation on its applicability is $N_b \geq 4$.

The corresponding equation for exterior beams without midspan diaphragm is:

$g = eg_{interior}$ (Eq. 7.5.3.1-2)

where $e = 0.6 + (d_e/10)$ (Eq. 7.5.3.1-2a)

If rigid midspan diaphragms are present, then the conservative approach in Eq. (7.5.2.1-3) must be used.

7.5.3.2 Open or Closed Spread Box Beams with Cast-In-Place Deck

The live load shear for interior beams [**Figure 7.5.1-1, types (b)** and **(c)**], may be determined by applying the following lane fraction:

$$g = \left(\frac{S}{7.4}\right)^{0.8}\left(\frac{d}{12.0L}\right)^{0.1} \quad \text{(Eq. 7.5.3.2-1)}$$

The formula is subject to two practical limits: $N_b \geq 3$ and $6.0 \leq S \leq 18.0$ ft. The other conditions are generally satisfied.

The related equation for exterior beams is:

$g = eg_{interior}$ (Eq. 7.5.3.2-2)

where $e = 0.8 + (d_e/10)$ (Eq. 7.5.3.2-2a)

Equation (7.5.2.1-3) must also be checked in case of rigid midspan diaphragms.

7.5.3.3 Adjacent Box Beams in Multi-Beam Decks

The applicable distribution factor equation for interior beams [**Figure 7.5.1-1, types (f)** and **(g)**], is:

$$g = \left(\frac{b}{156}\right)^{0.4}\left(\frac{b}{12.0L}\right)^{0.1}\left(\frac{I}{J}\right)^{0.05} \quad \text{(Eq. 7.5.3.3-1)}$$

These equations are limited to box widths not exceeding 5.0 ft, to span lengths $L \leq 120$ ft and to I or $J \leq 610{,}000$ in^4. The latter value may be exceeded if depth exceeds 66 in.

The shear for exterior beams is determined by applying the following lane fraction:

$g = eg_{interior}$ (Eq. 7.5.3.3-2)

where $e = 1.02 + (d_e/50)$, $d_e \leq 2.0$ (Eq. 7.5.3.3-2a)

7.5.3.4 Channel Sections or Tee Sections Connected by "Hinges" at Interface

For interior or exterior beams [**Figure 7.5.1-1, types (h), (i)** and **(j)**], the 'Lever Rule' in conjunction with the multiple presence factor, m, is specified.

7.5.4 Correction Factors For Skews

Skewed beam layout is generally dictated by complex highway intersections and/or by the lack of space in urban areas. When the skew angle of a bridge is small, say, less than 20°, it is often considered safe to ignore the angle of skew and to analyze the bridge as a zero-skew bridge whose span is equal to the skew span. This approach is generally conservative for moments in the beams, and slightly unsafe (<5%) for slab-on-beam decks for longitudinal shears.

LRFD Table 4.6.2.2.2e-1, lists reduction multipliers for moments in longitudinal beams. Also listed in LRFD Table 4.6.2.2.3c-1 are correction factors (> 1.0) applicable to the distribution factors for support shears at the obtuse corner of exterior beams. The commentary reminds the designer to check the possibility of uplift at the acute corners of large skews. Unfortunately, reliable multipliers and correction factors are missing for some bridge cross-sections.

7.5.4.1 Multipliers for Moments in Longitudinal Beams

Bending moments in interior and exterior beams on skewed supports may be reduced using the following multipliers: [LRFD Table 4.6.2.2.2e-1]

a) I-Beam, Bulb-Tee, Single or Double Tee Beams with Transverse Post-Tensioning [**Figure 7.5.1-1, types (i), (j) and (k)**]:

Use: $1 - c_1 (\tan\theta)^{1.5}$ (Eq. 7.5.4.1-1)

$$\text{where } c_1 = 0.25\left(\frac{K_g}{12.0Lt_s^{\,3}}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5} \qquad \text{(Eq. 7.5.4.1-1a)}$$

Set $c_1 = 0$ when $\theta < 30°$

Set $\theta = 60°$ when $\theta > 60°$

b) Spread Box Beams, Adjacent Box Beams with Concrete Overlays or Transverse Post-Tensioning, and Double Tees in Multi-Beam Decks [**Figure 7.5.1-1, types (b), (c), (f) and (g)**]:

Use: $1.05 - 0.25 \tan\theta \le 1.0$ (Eq. 7.5.4.1-2)

Set $\theta = 60°$ if $\theta > 60°$

7.5.4.2 Multipliers for Support Shear at Obtuse Corners of Exterior Beams

Shears in exterior beams on the obtuse corner of the bridge may be reduced using the following multipliers: [LRFD Table 4.6.2.2.3c-1]

a) I-Beam, Bulb-Tee, Single or Double Tee Beams with Transverse Post-Tensioning [**Figure 7.5.1-1, types (i), (j) and (k)**]:

$$\text{Use: } 1.0 + 0.20\left(\frac{12.0Lt_s^{\,3}}{K_g}\right)^{0.3}\tan\theta \qquad \text{(Eq. 7.5.4.2-1)}$$

This formula is valid for $\theta < 60°$.

b) Spread Box Beams [**Figure 7.5.1-1, types (b) and (c)**]:

$$\text{Use: } 1.0 + \left\{\left(\frac{Ld}{12.0}\right)^{0.5}\left(\frac{\tan\theta}{6S}\right)\right\} \qquad \text{(Eq. 7.5.4.2-2)}$$

Two practical limits apply, $\theta < 60°$ and $N_b \ge 3$.

c) Adjacent Box Beams with Cast-In-Place Overlay or Transverse Post-Tensioning [**Figure 7.5.1-1, types (f) and (g)**]:

$$\text{Use: } 1.0 + \left\{\frac{12.0L\,(\tan\theta)^{0.5}}{90\,d}\right\} \qquad \text{(Eq. 7.5.4.2-3)}$$

7.5.5 Lateral Bolting or Post-Tensioning Requirements

The following discussion concerns apparent inconsistencies in provisions of the *LRFD Specifications* related to the transverse connection between adjacent members.

7.5.5.1 Monolithic Behavior

As noted earlier, the *LRFD Specifications* indicate that adjacent beams connected by longitudinal joints may be considered to act monolithically if they are “sufficiently connected to act as a unit.” The *LRFD Specifications* also note that transverse post-tensioning provides the best connection between adjacent beams to achieve monolithic behavior but that a reinforced structural concrete overlay may also be used.

7.5.5.2 Minimum Post-Tensioning Requirement

LRFD Commentary Article C4.6.2.2.1 *recommends* a minimum transverse post-tensioning stress of 0.250 ksi to make the beams act as a unit. However, in LRFD Article 5.14.1.2.8, this same level of effective stress is *required* for the connection between adjacent members if transverse post-tensioning is used. Excessively large post-tensioning forces will be required to achieve this level of prestress across the depth of typical shear keys. There is no support in the literature or current practice for requiring this high level of prestress.

7.5.5.3 Concrete Overlay Alternative

LRFD Article 5.14.4.3.3.f gives requirements for a structural concrete topping that can also be used to achieve monolithic action, according to LRFD Commentary Article C4.6.2.2.1.

7.6 REFINED ANALYSIS METHODS

7.6.1 Introduction and Background

LRFD Article 4.6.3 allows the use of refined methods of analysis for lateral load distribution in lieu of the tabulated simplified equations. Although the simplified equations are based on a statistical approach, they are often quite conservative.

7.6.2 The Economic Perspective

The refined methods most often used to study the behavior of bridges are the grillage analysis and the finite element methods. The finite element analysis (FEA) requires the fewest simplifying assumptions in accounting for the greatest number of variables which govern the structural response of the bridge deck. However, input preparation time, and derivation of overall forces for the composite beam are usually quite tedious. On the other hand, data preparation for the grillage method is simpler and integration of stresses is not needed.

7.6.2.1 Moment Reductions

Analyses by Aswad and Chen (1994) have shown that using the FEA may result in a reduction of the lateral load distribution factor for moments by at least 18% for interior I-beams when compared to the simplified LRFD approach. The analysis for exterior I-beams and spread box beams showed a smaller but non-negligible reduction.

7.6.2.2 Stretching Span Capability

Detailed prestress designs by Aswad (1994) have shown that the percentage reduction in strands and release strength for interior beams is roughly one-half of the reduction in the distribution factor. For instance, a 22% reduction of midspan moment will result in about 11% less strands and less required release strength, or may allow a 4 to 5% increase in span length without having to use a deeper section. Clearly, there is a significant incentive for both the owner and the industry to use refined methods in many future projects. This is especially significant for beams with higher span-to-depth ratios.

7.6.3 St. Venant Torsional Constant, J

An important step in the FEA method is the computation of the torsional constant, J, for the basic precast beam. The torsional constant of a thin-walled, hollow box section, is given by the familiar formula from standard textbooks (Hambly, 1976):

$$J = 4A_0^2/\Sigma(s/t) \qquad \text{(Eq. 7.6.3-1)}$$

where

- A_0 = the area enclosed by centerlines of elements (walls)
- s = the length of a side element
- t = the thickness of that element

Table 7.6.3-1 Torsional Constant J for AASHTO I-Beams

Shape	J value, in.^{4}
Type I	4,745
Type II	7,793
Type III	17,044
Type IV	32,924
Type V	35,433
Type VI	36,071

For I-beams, the engineer should use rational methods such as those given in the report by Eby (1973). The use of formulas for open, thin sections is not appropriate. A list of St. Venant torsional constants for AASHTO I-beams is shown in **Table 7.6.3-1**.

7.6.4 Related Publications

The following reports by Lehigh University are recommended:

- For I-beams — Reports by Wegmuller (1973) and Zellin (1976)
- For spread box beams — Reports by Lin (1968), Guilford (1968), VanHorn (1969), Motarjemi (1969) and Chen (1970).

7.6.5 Modeling Guidelines

The following guidelines are suggested for refined analysis methods:

- A minimum of 9 nodes per beam span is preferred
- Aspect ratio of finite elements and grid panels should not exceed 5.0 (Note: this ratio should be reduced to 2.0 ± for better accuracy)
- Nodal loads shall be statically equivalent to the actual point load being applied
- For FEA, relative vertical distances should be maintained between various elements
- For grillage analysis, composite properties should be used
- St. Venant torsional constant, J, is to be determined rationally
- For grillage analysis, only one-half of the effective flange width of the flexural section, before transformation, should be used in computing J. In finite element analysis, an element should have membrane capability with sufficient discretization. Therefore, a shell element is ideal for modeling the cast-in-place slab.

7.6.6 Finite Element Study for Moment Distribution Factors

A parametric study for distribution factors was conducted by Chen and Aswad (1996) using FEA and the ADINA (1991) software. The number of beam elements per span was 16. There were two 4-noded shell elements between adjacent beam lines.

The study covered 10 different I-beam superstructures with spans, L, varying between 90 and 140 ft, and spacings, S, between 8 and 10 ft. The number of beam lines was 5, 6 or 7 while the total slab width (out-to-out) was either 48 or 60 ft. The midspan diaphragm is separated from the cast-in-place deck slab by a 6-in. gap.

The investigation also covered six various superstructures with a spacing, S, of either 8'-3" or 10'-6" and spans, L, varying between 60 and 100 ft. There were either 4 or 5 beam lines. The total slab width was either 39'-6" or 41'-0" which corresponds to 3 design lanes.

The following paragraphs summarize the findings of the study:

1. Refined methods of analysis may reduce the midspan moment by 18 to 23% in the case of interior I-beams, and by 4% to 12% for exterior I-beams when compared to the LRFD simplified method.

2. The same FEA may reduce the midspan moment by 6 to 12% for spread box beams. However, the reduction may reach 30% for exterior beams when midspan diaphragms are used. This is so because the *LRFD Specifications* have an interim formula that may result in an exaggerated midspan moment due to the assumption of infinitely rigid diaphragm.
3. The approximate equations for computing distribution factors are generally quite conservative when the span-to-depth ratios approach the upper limits of the span capability.

Based on this study, it is recommended that finite element or grillage analysis be used for the design of bridges with high span-to-depth ratios because they allow a significant reduction in the required release strength or, alternatively, a stretching of the span capability.

7.7 REFERENCES

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