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27
Piers and Columns

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27.1 Introduction

Piers provide vertical supports for spans at intermediate points and perform two main functions: transferring superstructure vertical loads to the foundations and resisting horizontal forces acting on the bridge. Although piers are traditionally designed to resist vertical loads, it is becoming more and more common to design piers to resist high lateral loads caused by seismic events. Even in some low seismic areas, designers are paying more attention to the ductility aspect of the design. Piers are predominantly constructed using reinforced concrete. Steel, to a lesser degree, is also used for piers. Steel tubes filled with concrete (composite) columns have gained more attention recently.

This chapter deals only with piers or columns for conventional bridges, such as grade separations, overcrossings, overheads, underpasses, and simple river crossings. Reinforced concrete columns will be discussed in detail while steel and composite columns will be briefly discussed. Substructures for arch, suspension, segmental, cable-stayed, and movable bridges are excluded from this chapter. Chapter 28 discusses the substructures for some of these special types of bridges.

27.2 Structural Types

27.2.1 General

Pier is usually used as a general term for any type of substructure located between horizontal spans and foundations. However, from time to time, it is also used particularly for a solid wall in order to distinguish it from columns or bents. From a structural point of view, a column is a member that resists the lateral force mainly by flexure action whereas a pier is a member that resists the lateral force mainly by a shear mechanism. A pier that consists of multiple columns is often called a *bent*.

There are several ways of defining pier types. One is by its structural connectivity to the superstructure: monolithic or cantilevered. Another is by its sectional shape: solid or hollow; round, octagonal, hexagonal, or rectangular. It can also be distinguished by its framing configuration: single or multiple column bent; hammerhead or pier wall.

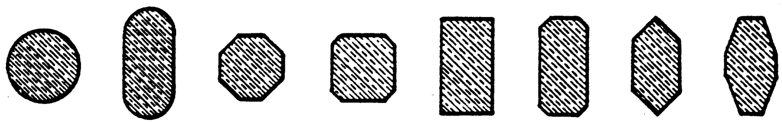


FIGURE 27.1 Typical cross-section shapes of piers for overcrossings or viaducts on land.

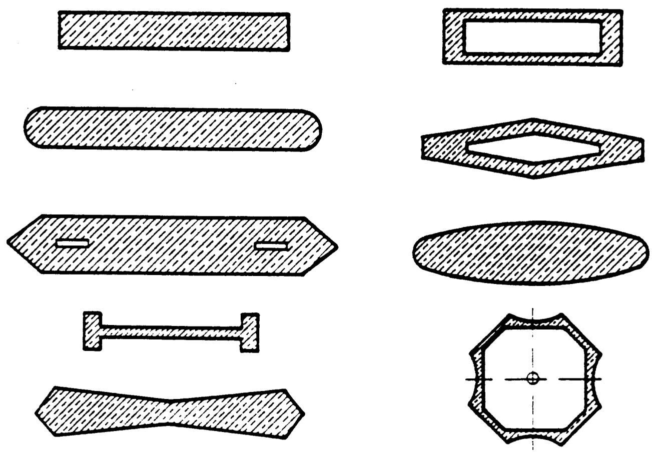


FIGURE 27.2 Typical cross-section shapes of piers for river and waterway crossings.

27.2.2 Selection Criteria

Selection of the type of piers for a bridge should be based on functional, structural, and geometric requirements. Aesthetics is also a very important factor of selection since modern highway bridges are part of a city's landscape. Figure 27.1 shows a collection of typical cross section shapes for overcrossings and viaducts on land and Figure 27.2 shows some typical cross section shapes for piers of river and waterway crossings. Often, pier types are mandated by government agencies or owners. Many state departments of transportation in the United States have their own standard column shapes.

Solid wall piers, as shown in Figures 27.3a and 27.4, are often used at water crossings since they can be constructed to proportions that are both slender and streamlined. These features lend themselves well for providing minimal resistance to flood flows.

Hammerhead piers, as shown in Figure 27.3b, are often found in urban areas where space limitation is a concern. They are used to support steel girder or precast prestressed concrete superstructures. They are aesthetically appealing. They generally occupy less space, thereby providing more room for the traffic underneath. Standards for the use of hammerhead piers are often maintained by individual transportation departments.

A column bent pier consists of a cap beam and supporting columns forming a frame. Column bent piers, as shown in Figure 27.3c and Figure 27.5, can either be used to support a steel girder superstructure or be used as an integral pier where the cast-in-place construction technique is used. The columns can be either circular or rectangular in cross section. They are by far the most popular forms of piers in the modern highway system.

A pile extension pier consists of a drilled shaft as the foundation and the circular column extended from the shaft to form the substructure. An obvious advantage of this type of pier is that it occupies a minimal amount of space. Widening an existing bridge in some instances may require pile extensions because limited space precludes the use of other types of foundations.

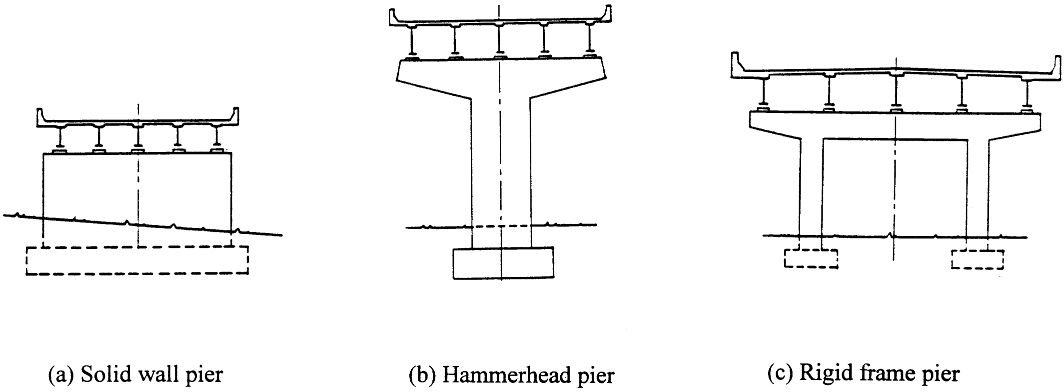


FIGURE 27.3 Typical pier types for steel bridges.

Selections of proper pier type depend upon many factors. First of all, it depends upon the type of superstructure. For example, steel girder superstructures are normally supported by cantilevered piers, whereas the cast-in-place concrete superstructures are normally supported by monolithic bents. Second, it depends upon whether the bridges are over a waterway or not. Pier walls are preferred on river crossings, where debris is a concern and hydraulics dictates it. Multiple pile extension bents are commonly used on slab bridges. Last, the height of piers also dictates the type selection of piers. The taller piers often require hollow cross sections in order to reduce the weight of the substructure. This then reduces the load demands on the costly foundations. Table 27.1 summarizes the general type selection guidelines for different types of bridges.

27.3 Design Loads

Piers are commonly subjected to forces and loads transmitted from the superstructure, and forces acting directly on the substructure. Some of the loads and forces to be resisted by the substructure include:

- Dead loads
- Live loads and impact from the superstructure
- Wind loads on the structure and the live loads
- Centrifugal force from the superstructure
- Longitudinal force from live loads
- Drag forces due to the friction at bearings
- Earth pressure
- Stream flow pressure
- Ice pressure
- Earthquake forces
- Thermal and shrinkage forces
- Ship impact forces
- Force due to prestressing of the superstructure
- Forces due to settlement of foundations

The effect of temperature changes and shrinkage of the superstructure needs to be considered when the superstructure is rigidly connected with the supports. Where expansion bearings are used, forces caused by temperature changes are limited to the frictional resistance of bearings.

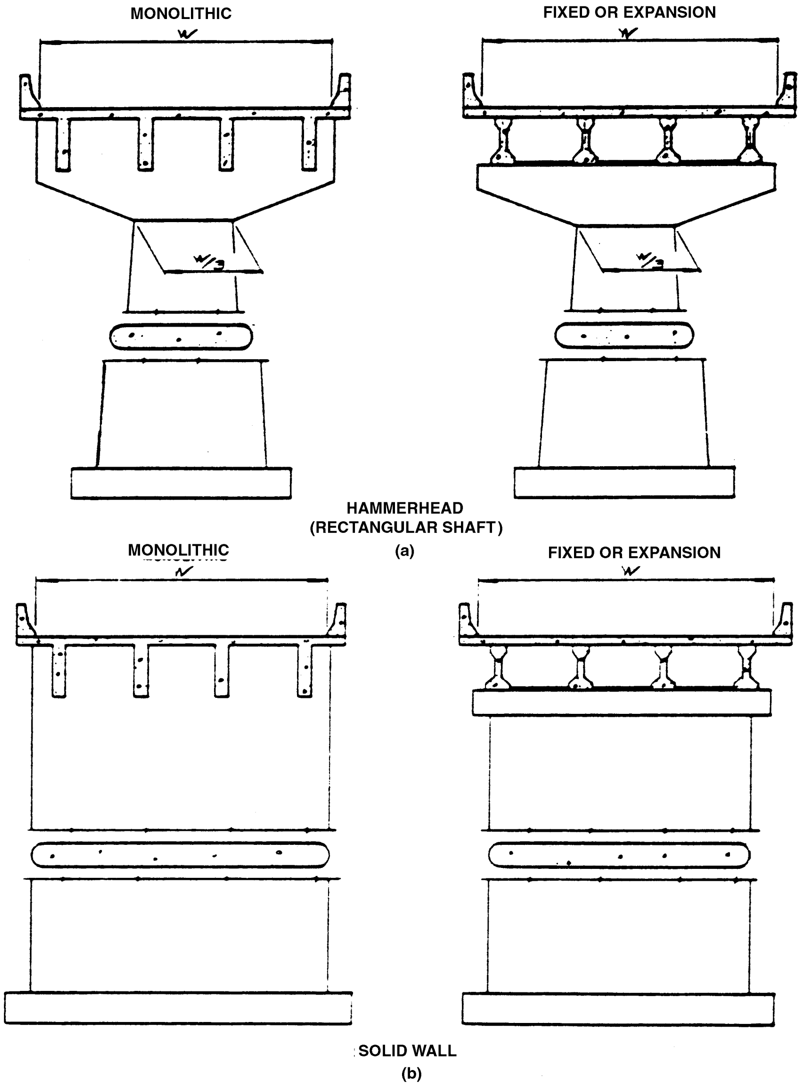


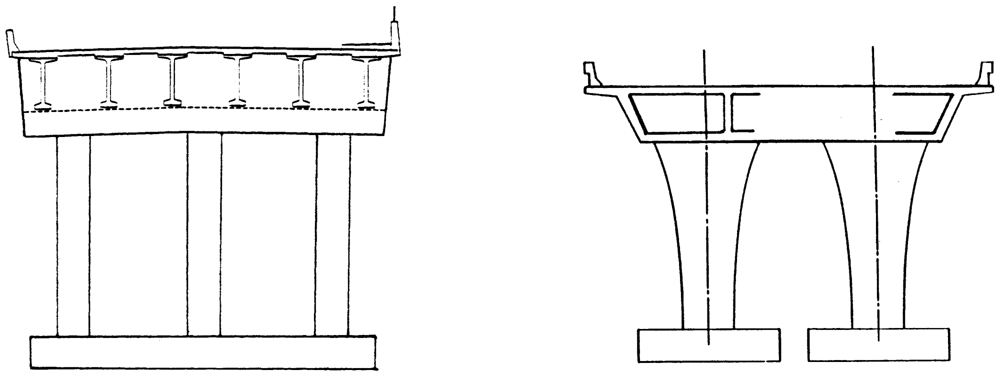
FIGURE 27.4 Typical pier types and configurations for river and waterway crossings.

Readers should refer to Chapters 5 and 6 for more details about various loads and load combinations and Part IV about earthquake loads. In the following, however, two load cases, live loads and thermal forces, will be discussed in detail because they are two of the most common loads on the piers, but are often applied incorrectly.

27.3.1 Live Loads

Bridge live loads are the loads specified or approved by the contracting agencies and owners. They are usually specified in the design codes such as AASHTO LRFD Bridge Design Specifications [1]. There are other special loading conditions peculiar to the type or location of the bridge structure which should be specified in the contracting documents.

Live-load reactions obtained from the design of individual members of the superstructure should not be used directly for substructure design. These reactions are based upon maximum conditions



(a) Bent for precast girders

(b) Bent for cast-in-place girders

FIGURE 27.5 Typical pier types for concrete bridges.

TABLE 27.1 General Guidelines for Selecting Pier Types

		Applicable Pier Types
		Steel Superstructure
Over water	Tall piers	Pier walls or hammerheads (T-piers) (Figures 27.3a and b); hollow cross sections for most cases; cantilevered; could use combined hammerheads with pier wall base and step tapered shaft
	Short piers	Pier walls or hammerheads (T-piers) (Figures 27.3a and b); solid cross sections; cantilevered
On land	Tall piers	Hammerheads (T-piers) and possibly rigid frames (multiple column bents)(Figures 27.3b and c); hollow cross sections for single shaft and solid cross sections for rigid frames; cantilevered
	Short piers	Hammerheads and rigid frames (Figures 27.3b and c); solid cross sections; cantilevered
		Precast Prestressed Concrete Superstructure
Over water	Tall piers	Pier walls or hammerheads (Figure 27.4); hollow cross sections for most cases; cantilevered; could use combined hammerheads with pier wall base and step-tapered shaft
	Short piers	Pier walls or hammerheads; solid cross sections; cantilevered
On land	Tall piers	Hammerheads and possibly rigid frames (multiple column bents); hollow cross sections for single shafts and solid cross sections for rigid frames; cantilevered
	Short piers	Hammerheads and rigid frames (multiple column bents) (Figure 27.5a); solid cross sections; cantilevered
		Cast-in-Place Concrete Superstructure
Over water	Tall piers	Single shaft pier (Figure 27.4); superstructure will likely cast by traveled forms with balanced cantilevered construction method; hollow cross sections; monolithic; fixed at bottom
	Short piers	Pier walls (Figure 27.4); solid cross sections; monolithic; fixed at bottom
On land	Tall piers	Single or multiple column bents; solid cross sections for most cases, monolithic; fixed at bottom
	Short piers	Single or multiple column bents (Figure 27.5b); solid cross sections; monolithic; pinned at bottom

for one beam and make no allowance for distribution of live loads across the roadway. Use of these maximum loadings would result in a pier design with an unrealistically severe loading condition and uneconomical sections.

For substructure design, a maximum design traffic lane reaction using either the standard truck load or standard lane load should be used. Design traffic lanes are determined according to AASHTO

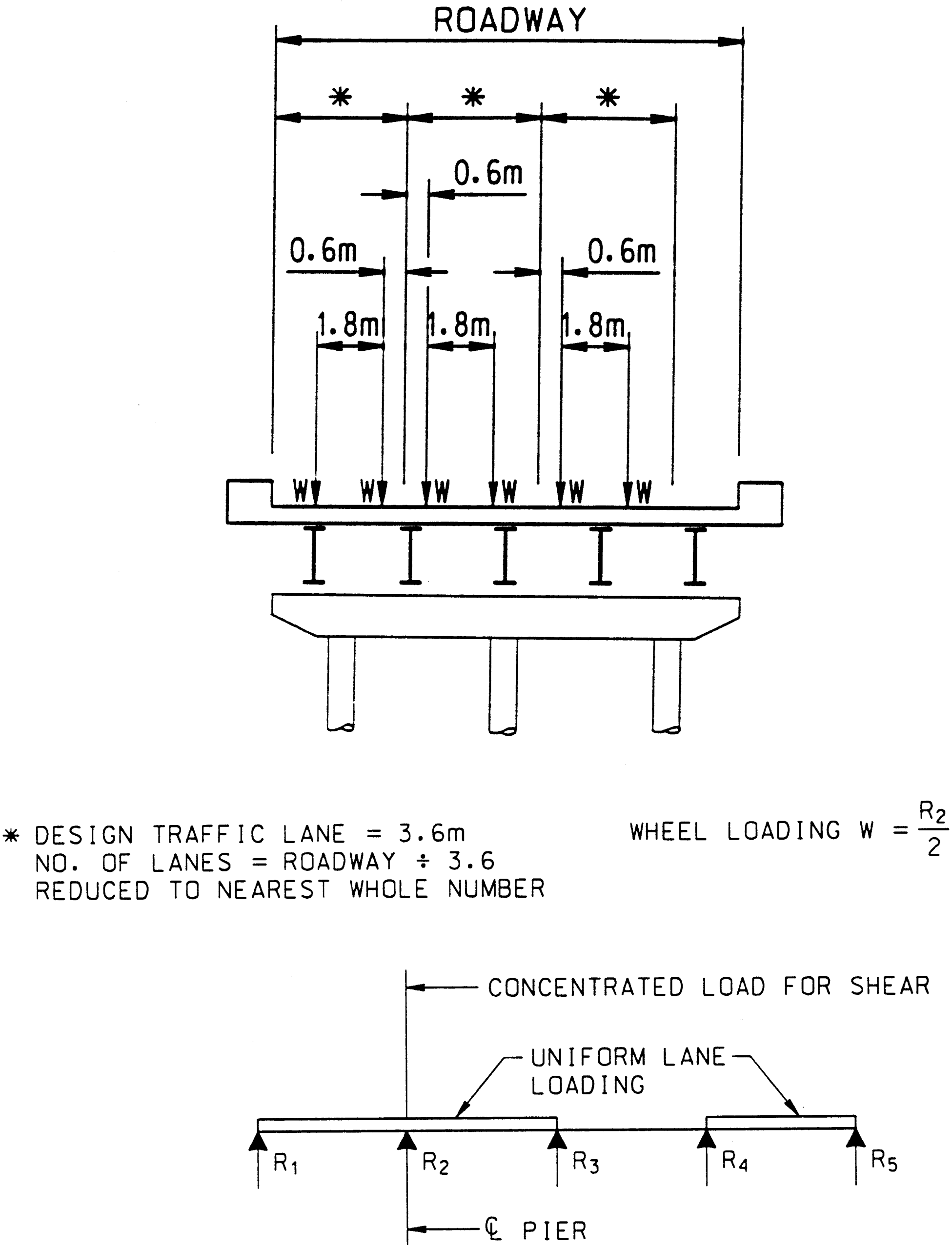


FIGURE 27.6 Wheel load arrangement to produce maximum positive moment.

LRFD [1] Section 3.6. For the calculation of the actual beam reactions on the piers, the maximum lane reaction can be applied within the design traffic lanes as wheel loads, and then distributed to the beams assuming the slab between beams to be simply supported. (Figure 27.6). Wheel loads can be positioned anywhere within the design traffic lane with a minimum distance between lane boundary and wheel load of 0.61 m (2 ft).

The design traffic lanes and the live load within the lanes should be arranged to produce beam reactions that result in maximum loads on the piers. AASHTO LRFD Section 3.6.1.1.2 provides load reduction factors due to multiple loaded lanes.

TABLE 27.2 Dynamic Load Allowance, IM

Component	IM
Deck joints — all limit states	75%
All other components	
• Fatigue and fracture limit state	15%
• All other limit states	33%

Live-load reactions will be increased due to impact effect. AASHTO LRFD [1] refers to this as the *dynamic load allowance, IM*. and is listed here as in Table 27.2.

27.3.2 Thermal Forces

Forces on piers due to thermal movements, shrinkage, and prestressing can become large on short, stiff bents of prestressed concrete bridges with integral bents. Piers should be checked against these forces. Design codes or specifications normally specify the design temperature range. Some codes even specify temperature distribution along the depth of the superstructure member.

The first step in determining the thermal forces on the substructures for a bridge with integral bents is to determine the point of no movement. After this point is determined, the relative displacement of any point along the superstructure to this point is simply equal to the distance to this point times the temperature range and times the coefficient of expansion. With known displacement at the top and known boundary conditions at the top and bottom, the forces on the pier due to the temperature change can be calculated by using the displacement times the stiffness of the pier.

The determination of the point of no movement is best demonstrated by the following example, which is adopted from Memo to Designers issued by California Department of Transportations [2]:

Example 27.1

A 225.55-m (740-foot)-long and 23.77-m (78-foot) wide concrete box-girder superstructure is supported by five two-column bents. The size of the column is 1.52 m (5 ft) in diameter and the heights vary between 10.67 m (35 ft) and 12.80 m (42 ft). Other assumptions are listed in the calculations. The calculation is done through a table. Please refer Figure 27.7 for the calculation for determining the point of no movement.

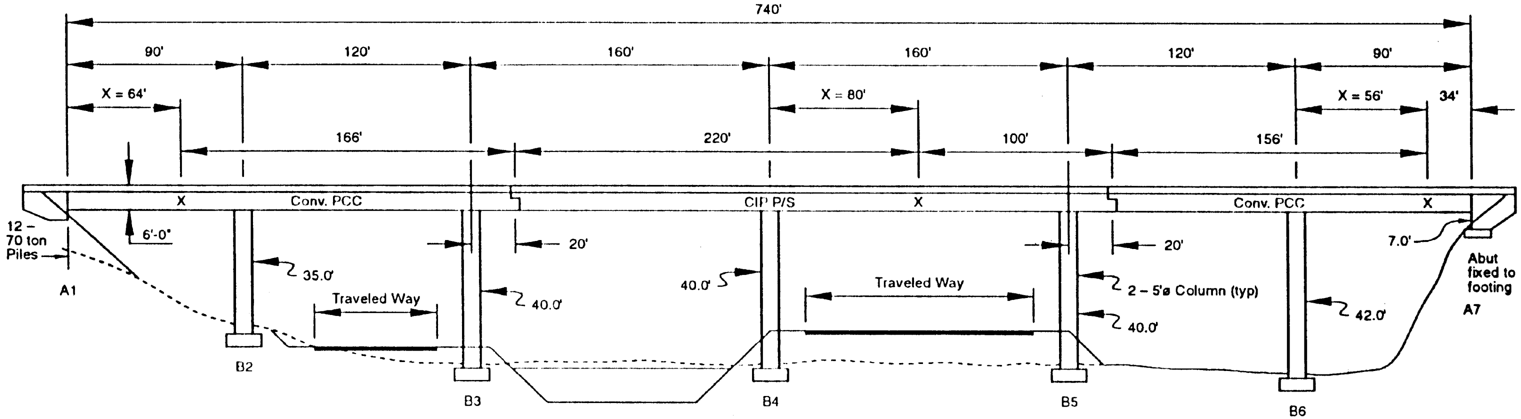
27.4 Design Criteria

27.4.1 Overview

Like the design of any structural component, the design of a pier or column is performed to fulfill strength and serviceability requirements. A pier should be designed to withstand the overturning, sliding forces applied from superstructure as well as the forces applied to substructures. It also needs to be designed so that during an extreme event it will prevent the collapse of the structure but may sustain some damage.

A pier as a structure component is subjected to combined forces of axial, bending, and shear. For a pier, the bending strength is dependent upon the axial force. In the plastic hinge zone of a pier, the shear strength is also influenced by bending. To complicate the behavior even more, the bending moment will be magnified by the axial force due to the P-Δ effect.

In current design practice, the bridge designers are becoming increasingly aware of the adverse effects of earthquake. Therefore, ductility consideration has become a very important factor for bridge design. Failure due to scouring is also a common cause of failure of bridges. In order to prevent this type of failure, the bridge designers need to work closely with the hydraulic engineers to determine adequate depths for the piers and provide proper protection measures.



	A1		B2		B3			B4		B5			B6		A7		
$I\ (Ft)^4$	1.38		61.36		61.36			61.36		61.36			61.36		102		
L (Ft)	5.50		35.0		40.0		Sum	40.0		40.0		Sum	42.0		7.0		Sum
P (kips) @ 1" side sway	1200	+	618	+	415	=	2,233	415	+	415	=	830	359	Will slide +	600	=	959
D (distance from 1st member of frame)	0		90		210			0		160			0		90		
P × D / 100	0	+	556	+	872	=	1,428	0	+	664	=	664	0	+	540	=	540
$X = \frac{\Sigma(P \times D)/100}{\Sigma P}(100) = \frac{1{,}428}{2{,}233}(100) = \underline{64'}$								$\frac{664}{830}(100) = \underline{80'}$					$\frac{540}{959}(100) = \underline{56'}$				

Notes:
- Width of Structure = 78'
- Diameter of Column = 5'-0"
- K/Pile @ 1" deflection = 100 kips
- Point of No Movement = X
- Refer to Properties/Piles Table

Assumptions:
1. Super str. inf. rigid
2. Columns fixed top and bottom
3. Abutment footing will slide @ a force equal to D.W.
4. E (piles) = 4×10^6 psi
   E (columns) = 3×10^6 psi

Fixed/Fixed Condition

$$P\ (\text{Col.}) = 12EI\frac{\Delta}{L^3}$$

$$@\ 1"\ \text{defl.} = \frac{432\,I}{\left(\frac{L}{10}\right)^3}$$

Pinned/Fixed Condition

$$P\ (\text{Col.}) = 3EI\frac{\Delta}{L^3}$$

$$@\ 1"\ \text{defl.} = \frac{108\,I}{\left(\frac{L}{10}\right)^3}$$

D.W. Abut 7 = 600 k (assume linear up to 1" deflection)

$$I\ (\text{abut}) = \frac{78}{12}(2.5)^3 = 102$$

FIGURE 27.7 Calculation of points of no movement.

27.4.2 Slenderness and Second-Order Effect

The design of compression members must be based on forces and moments determined from an analysis of the structure. Small deflection theory is usually adequate for the analysis of beam-type members. For compression members, however, the second-order effect must be considered. According to AASHTO LRFD [1], the second-order effect is defined as follows:

> The presence of compressive axial forces amplify both out-of-straightness of a component and the deformation due to non-tangential loads acting thereon, therefore increasing the eccentricity of the axial force with respect to the centerline of the component. The synergistic effect of this interaction is the apparent softening of the component, i.e., a loss of stiffness.

To assess this effect accurately, a properly formulated large deflection nonlinear analysis can be performed. Discussions on this subject can be found in References [3,4] and Chapter 36. However, it is impractical to expect practicing engineers to perform this type of sophisticated analysis on a regular basis. The moment magnification procedure given in AASHTO LRFD [1] is an approximate process which was selected as a compromise between accuracy and ease of use. Therefore, the AASHTO LRFD moment magnification procedure is outlined in the following.

When the cross section dimensions of a compression member are small in comparison to its length, the member is said to be slender. Whether or not a member can be considered slender is dependent on the magnitude of the slenderness ratio of the member. The slenderness ratio of a compression member is defined as, KL_u/r, where K is the effective length factor for compression members; L_u is the unsupported length of compression member; r is the radius of gyration = $\sqrt{I/A}$; I is the moment of inertia; and A is the cross-sectional area.

When a compression member is braced against side sway, the effective length factor, $K = 1.0$ can be used. However, a lower value of K can be used if further analysis demonstrates that a lower value is applicable. L_u is defined as the clear distance between slabs, girders, or other members which is capable of providing lateral support for the compression member. If haunches are present, then, the unsupported length is taken from the lower extremity of the haunch in the plane considered (AASHTO LRFD 5.7.4.3). For a detailed discussion of the K-factor, please refer to Chapter 52.

For a compression member braced against side sway, the effects of slenderness can be ignored as long as the following condition is met (AASHTO LRFD 5.7.4.3):

$$\frac{KL_u}{r} < 34 - \left(\frac{12 M_{1b}}{M_{2b}}\right) \tag{27.1}$$

where

M_{1b} = smaller end moment on compression member — positive if member is bent in single curvature, negative if member is bent in double curvature

M_{2b} = larger end moment on compression member — always positive

For an unbraced compression member, the effects of slenderness can be ignored as long as the following condition is met (AASHTO LRFD 5.7.4.3):

$$\frac{KL_u}{r} < 22 \tag{27.2}$$

If the slenderness ratio exceeds the above-specified limits, the effects can be approximated through the use of the moment magnification method. If the slenderness ratio KL_u/r exceeds 100, however, a more-detailed second-order nonlinear analysis [Chapter 36] will be required. Any detailed analysis should consider the influence of axial loads and variable moment of inertia on member stiffness and forces, and the effects of the duration of the loads.

The factored moments may be increased to reflect effects of deformations as follows:

$$M_c = \delta_b M_{2b} + \delta_s M_{2s} \tag{27.3}$$

where

M_{2b} = moment on compression member due to factored gravity loads that result in no appreciable side sway calculated by conventional first-order elastic frame analysis, always positive

M_{2s} = moment on compression member due to lateral or gravity loads that result in side sway, Δ, greater than $L_u/1500$, calculated by conventional first-order elastic frame analysis, always positive

The moment magnification factors are defined as follows:

$$\delta_b = \frac{C_m}{1 - \dfrac{P_u}{\phi P_c}} \geq 1.0 \tag{27.4}$$

$$\delta_s = \frac{1}{1 - \dfrac{\sum P_u}{\phi \sum P_c}} \geq 1.0 \tag{27.5}$$

where

P_u = factored axial load

P_c = Euler buckling load, which is determined as follows:

$$P_c = \frac{\pi^2 EI}{\left(KL_u\right)^2} \tag{27.6}$$

C_m, a factor which relates the actual moment diagram to an equivalent uniform moment diagram, is typically taken as 1.0. However, in the case where the member is braced against side sway and without transverse loads between supports, it may be taken by the following expression:

$$C_m = 0.60 + 0.40\left(\frac{M_{1b}}{M_{2b}}\right) \tag{27.7}$$

The value resulting from Eq. (27.7), however, is not to be less than 0.40.

To compute the flexural rigidity EI for concrete columns, AASHTO offers two possible solutions, with the first being:

$$EI = \frac{\dfrac{E_c I_g}{5} + E_s I_s}{1 + \beta_d} \tag{27.8}$$

and the second, more-conservative solution being:

$$EI = \frac{\dfrac{E_c I_g}{2.5}}{1 + \beta_d} \tag{27.9}$$

where E_c is the elastic modulus of concrete, I_g is the gross moment inertia, E_s is the elastic modules of reinforcement, I_s is the moment inertia of reinforcement about centroidal axis, and β is the ratio of maximum dead-load moment to maximum total-load moment and is always positive. It is an approximation of the effects of creep, so that when larger moments are induced by loads sustained over a long period of time, the creep deformation and associated curvature will also be increased.

27.4.3 Concrete Piers and Columns

27.4.3.1 Combined Axial and Flexural Strength

A critical aspect of the design of bridge piers is the design of compression members. We will use AASHTO LRFD Bridge Design Specifications [1] as the reference source. The following discussion provides an overview of some of the major criteria governing the design of compression members.

Under the Strength Limit State Design, the factored resistance is determined with the product of nominal resistance, P_n, and the resistance factor, ϕ. Two different values of ϕ are used for the nominal resistance P_n. Thus, the factored axial load resistance ϕP_n is obtained using $\phi = 0.75$ for columns with spiral and tie confinement reinforcement. The specifications also allows for the value ϕ to be linearly increased from the value stipulated for compression members to the value specified for flexure which is equal to 0.9 as the design axial load ϕP_n decreases from $0.10 f_c' A_g$ to zero.

Interaction Diagrams

Flexural resistance of a concrete member is dependent upon the axial force acting on the member. Interaction diagrams are usually used as aids for the design of the compression members. Interaction diagrams for columns are usually created assuming a series of strain distributions, and computing the corresponding values of P and M. Once enough points have been computed, the results are plotted to produce an interaction diagram.

Figure 27.8 shows a series of strain distributions and the resulting points on the interaction diagram. In an actual design, however, a few points on the diagrams can be easily obtained and can define the diagram rather closely.

- Pure Compression:

The factored axial resistance for pure compression, ϕP_n, may be computed by:

For members with spiral reinforcement:

$$P_r = \phi P_n = \phi 0.85 P_o = \phi 0.85 \left[0.85 f_c' \left(A_g - A_{st} \right) + A_{st} f_y \right] \tag{27.10}$$

For members with tie reinforcement:

$$P_r = \phi P_n = \phi 0.80 P_o = \phi 0.80 \left[0.85 f_c' \left(A_g - A_{st} \right) + A_{st} f_y \right] \tag{27.11}$$

For design, pure compression strength is a hypothetical condition since almost always there will be moments present due to various reasons. For this reason, AASHTO LRFD 5.7.4.4 limits the nominal axial load resistance of compression members to 85 and 80% of the axial resistance at zero eccentricity, P_o, for spiral and tied columns, respectively.

- Pure Flexure:

The section in this case is only subjected to bending moment and without any axial force. The factored flexural resistance, M_r, may be computed by

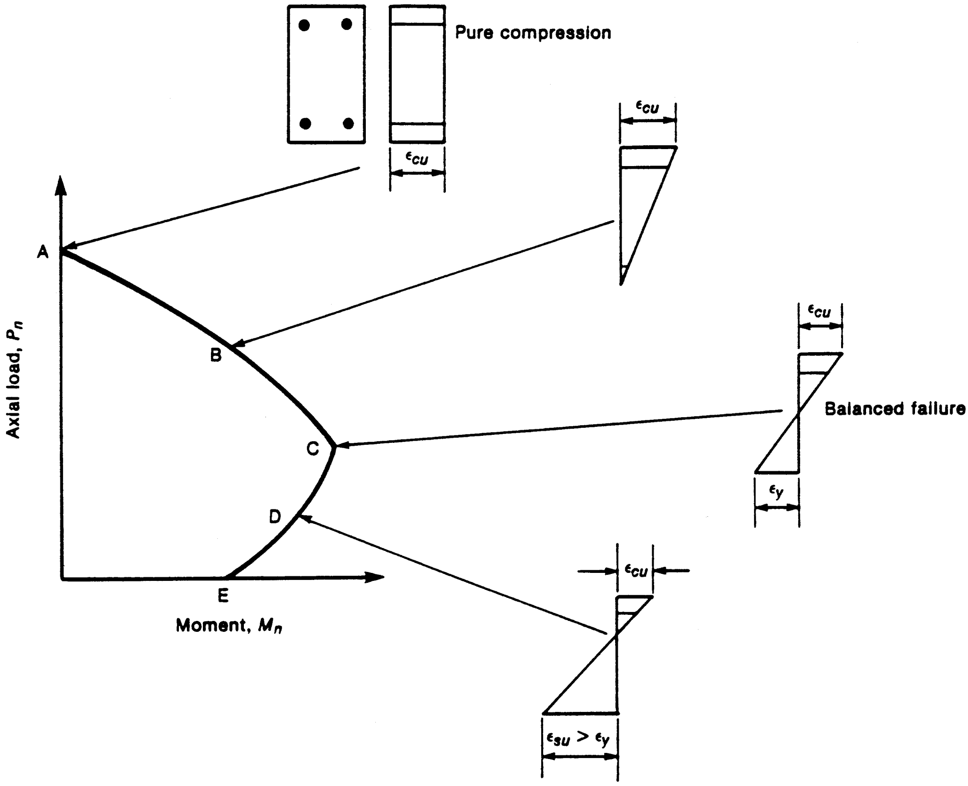


FIGURE 27.8 Strain distributions corresponding to points on interaction diagram.

$$M_r = \phi M_n = \phi\left[A_s f_y d\left(1 - 0.6\rho\frac{f_y}{f_c'}\right)\right] \\ = \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right] \tag{27.12}$$

where

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

- **Balanced Strain Conditions**:

Balanced strain conditions correspond to the strain distribution where the extreme concrete strain reaches 0.003 and the strain in reinforcement reaches yield at the same time. At this condition, the section has the highest moment capacity. For a rectangular section with reinforcement in one face, or located in two faces at approximately the same distance from the axis of bending, the balanced factored axial resistance, P_r, and balanced factored flexural resistance, M_r, may be computed by

$$P_r = \phi P_b = \phi\left[0.85 f_c' b a_b + A_s' f_s' - A_s f_y\right] \tag{27.13}$$

and

$$M_r = \phi M_b = \phi\left[0.85 f_c' b a_b \left(d - d'' - a_b/2\right) + A_s' f_s'\left(d - d' - d''\right) + A_s f_y d''\right] \qquad (27.14)$$

where

$$a_b = \left(\frac{600}{600 + f_y}\right)\beta_1 d$$

and

$$f_s' = 600\left[1 - \left(\frac{d'}{d}\right)\left(600 + \frac{f_y}{600}\right)\right] \le f_y$$

where f_y is in MPa.

Biaxial Bending

AASHTO LRFD 5.7.4.5 stipulates that the design strength of noncircular members subjected to biaxial bending may be computed, in lieu of a general section analysis based on stress and strain compatibility, by one of the following approximate expressions:

$$\frac{1}{P_{rxy}} = \frac{1}{P_{rx}} + \frac{1}{P_{ry}} - \frac{1}{P_o} \qquad (27.15)$$

when the factored axial load, $P_u \ge 0.10\phi f_c' A_g$

$$\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \le 1 \qquad (27.16)$$

when the factored axial load, $P_u < 0.10\phi f_c' A_g$

where

P_{rxy} = factored axial resistance in biaxial flexure

P_{rx}, P_{ry} = factored axial resistance corresponding to M_{rx}, M_{ry}

M_{ux}, M_{uy} = factored applied moment about the x-axis, y-axis

M_{rx}, M_{ry} = uniaxial factored flexural resistance of a section about the x-axis and y-axis corresponding to the eccentricity produced by the applied factored axial load and moment, and

P_o = $0.85 f_c'(A_g - A_s) + A_s f_y$

27.4.3.2 Shear Strength

Under the normal load conditions, the shear seldom governs the design of the column for conventional bridges since the lateral loads are usually small compared with the vertical loads. However, in a seismic design, the shear is very important. In recent years, the research effort on shear strength evaluation for columns has been increased remarkably. AASHTO LRFD provides a general shear equation that applies for both beams and columns. The concrete shear capacity component and the angle of inclination of diagonal compressive stresses are functions of the shear stress on the concrete and the strain in the reinforcement on the flexural tension side of the member. It is rather involved and hard to use.

Alternatively, the equations recommended by ATC-32 [5] can be used with acceptable accuracy. The recommendations are listed as follows.

Except for the end regions of ductile columns, the nominal shear strength provided by concrete, V_c, for members subjected to flexure and axial compression should be computed by

$$V_c = 0.165\left(1+(3.45)\left(10^{-6}\right)\frac{N_u}{A_g}\right)\sqrt{f_c'}A_e \quad \text{(MPa)} \tag{27.17}$$

If the axial force is in tension, the V_c should be computed by

$$V_c = 0.165\left(1+(1.38)\left(10^{-5}\right)\frac{N_u}{A_g}\right)\sqrt{f_c'}A_e \quad \text{(MPa)} \tag{27.18}$$

(note that N_u is negative for tension),

where
A_g = gross section area of the column (mm^2)
A_e = effective section area, can be taken as $0.8A_g$ (mm^2)
N_u = axial force applied to the column (N)
f_c' = compressive strength of concrete (MPa)

For end regions where the flexural ductility is normally high, the shear capacity should be reduced. ATC-32 [5] offers the following equations to address this interaction.

With the end region of columns extending a distance from the critical section or sections not less than $1.5D$ for circular columns or $1.5h$ for rectangular columns, the nominal shear strength provided by concrete subjected to flexure and axial compression should be computed by

$$V_c = 0.165\left(0.5+(6.9)\left(10^{-6}\right)\frac{N_u}{A_g}\right)\sqrt{f_c'}A_e \quad \text{(MPa)} \tag{27.19}$$

When axial load is tension, V_c can be calculated as

$$V_c = 0.165\left(1+(1.38)\left(10^{-5}\right)\frac{N_u}{A_g}\right)\sqrt{f_c'}A_e \quad \text{(MPa)} \tag{27.18}$$

Again, N_u should be negative in this case.

The nominal shear contribution from reinforcement is given by

$$V_s = \frac{A_v f_{yh} d}{s} \quad \text{(MPa)} \tag{27.20}$$

for tied rectangular sections, and by

$$V_s = \frac{\pi}{2}\frac{A_h f_{yh} D'}{s} \tag{27.21}$$

for spirally reinforced circular sections. In these equations, A_v is the total area of shear reinforcement parallel to the applied shear force, A_h is the area of a single hoop, f_{yh} is the yield stress of horizontal reinforcement, D' is the diameter of a circular hoop, and s is the spacing of horizontal reinforcement.

27.4.3.3 Ductility of Columns

The AASHTO LRFD [1] introduces the term *ductility* and requires that a structural system of bridge be designed to ensure the development of significant and visible inelastic deformations prior to failure.

The term *ductility* defines the ability of a structure and selected structural components to deform beyond elastic limits without excessive strength or stiffness degradation. In mathematical terms, the ductility μ is defined by the ratio of the total imposed displacement Δ at any instant to that at the onset of yield Δ_y. This is a measure of the ability for a structure, or a component of a structure, to absorb energy. The goal of seismic design is to limit the estimated maximum ductility demand to the ductility capacity of the structure during a seismic event.

For concrete columns, the confinement of concrete must be provided to ensure a ductile column. AASHTO LRFD [1] specifies the following minimum ratio of spiral reinforcement to total volume of concrete core, measured out-to-out of spirals:

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f_c'}{f_{yh}} \tag{27.22}$$

The transverse reinforcement for confinement at the plastic hinges shall be determined as follows:

$$\rho_s = 0.16\frac{f_c'}{f_y}\left(0.5 + \frac{1.25P_u}{A_g f_c'}\right) \tag{27.23}$$

for which

$$\left(0.5 + \frac{1.25P_u}{A_g f_c'}\right) \geq 1.0$$

The total cross-sectional area (A_{sh}) of rectangular hoop (stirrup) reinforcement for a rectangular column shall be either

$$A_{sh} = 0.30ah_c\frac{f_c'}{f_{yh}}\left(\frac{A_g}{A_c} - 1\right) \tag{27.24}$$

or,

$$A_{sh} = 0.12ah_c\frac{f_c'}{f_y}\left(0.5 + \frac{1.25P_u}{A_g f_c'}\right) \tag{27.25}$$

whichever is greater,

where

a = vertical spacing of hoops (stirrups) with a maximum of 100 mm (mm)
A_c = area of column core measured to the outside of the transverse spiral reinforcement (mm^2)
A_g = gross area of column (mm^2)
A_{sh} = total cross-sectional area of hoop (stirrup) reinforcement (mm^2)
f_c' = specified compressive strength of concrete (Pa)
f_{yh} = yield strength of hoop or spiral reinforcement (Pa)
h_c = core dimension of tied column in the direction under consideration (mm)
ρ_s = ratio of volume of spiral reinforcement to total volume of concrete core (out-to-out of spiral)
P_u = factored axial load (MN)

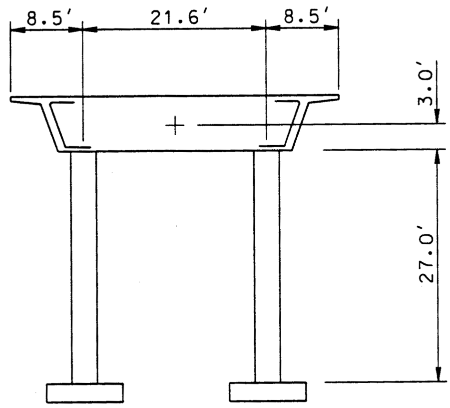


FIGURE 27.9 Example 27.2 — typical section.

TABLE 27.3 Column Group Loads — Service

	Dead Load	Live Load + Impact: Case 1 Trans $M_{y\text{-max}}$	Case 2 Long $M_{x\text{-max}}$	Case 3 Axial N-max	Wind	Wind on LL	Long Force	Centrifugal Force-M_y	Temp.
M_y (k-ft)	220	75	15	32	532	153	208	127	180
M_x (k-ft)	148	67	599	131	192	86	295	2	0
P (k)	1108	173	131	280	44	17	12	23	0

TABLE 27.4 Unreduced Seismic Loads (ARS)

	Case 1 Max. Transverse	Case 2 Max. Longitudinal
M_y — Trans (k-ft)	4855	3286
M_x — Long (k-ft)	3126	3334
P — Axial (k)	–282	–220

Example 27.2 Design of a Two-Column Bent

Design the columns of a two-span overcrossing. The typical section of the structure is shown in Figure 27.9. The concrete box girder is supported by a two-column bent and is subjected to HS20 loading. The columns are pinned at the bottom of the columns. Therefore, only the loads at the top of columns are given here. Table 27.3 lists all the forces due to live load plus impact. Table 27.4 lists the forces due to seismic loads. Note that a load reduction factor of 5.0 will be assumed for the columns.

Material Data

f'_c = 4.0 ksi (27.6 MPa) $\quad E_c$ = 3605 ksi (24855 MPa)

E_s = 29000 ksi (199946 MPa) $\quad f_y$ = 60 ksi (414 MPa)

Try a column size of 4 ft (1.22 m) in diameter. Provide 26-#9 (26-#30) longitudinal reinforcement. The reinforcement ratio is 1.44%.

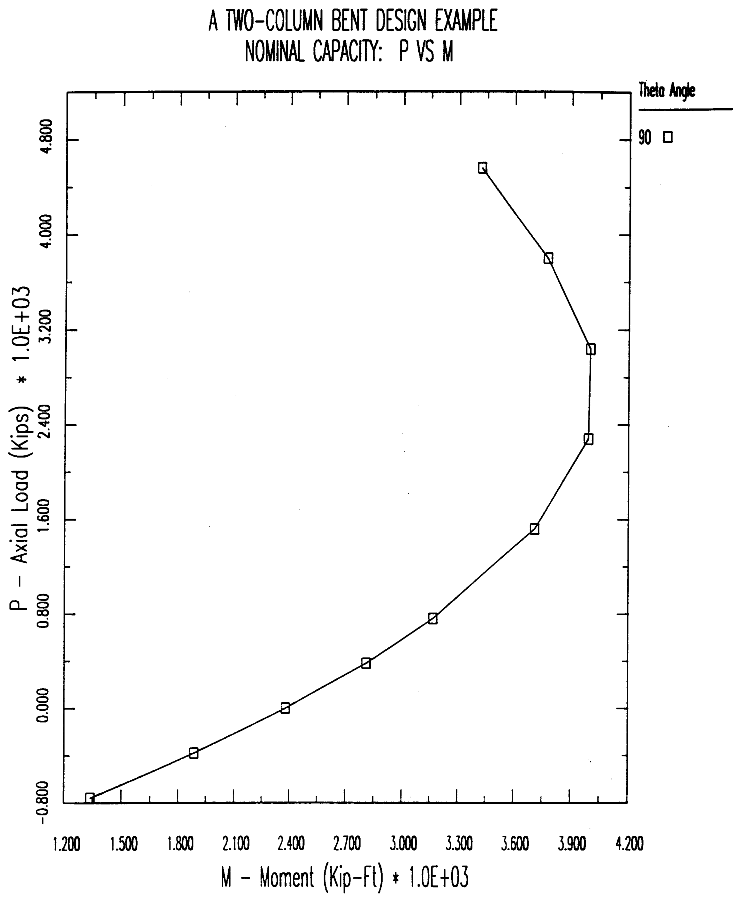


FIGURE 27.10 Example 27.2 — interaction diagram.

Section Properties

A_g = 12.51 ft^2 (1.16 m^2) $\quad$ A_{st} = 26.0 in^2 (16774 mm^2)

I_{xc} = I_{yc} = 12.46 ft^4 (0.1075 m^4) $\quad$ I_{xs} = I_{ys} = 0.2712 ft^4 (0.0023 m^4)

The analysis follows the procedure discussed in Section 27.4.3.1. The moment and axial force interaction diagram is generated and is shown in Figure 27.10.

Following the procedure outlined in Section 27.4.2, the moment magnification factors for each load group can be calculated and the results are shown in Table 27.5.

In which:

$$K_y = K_x = 2.10$$

$$K_y L/R = K_x\ L/R = 2.1 \times 27.0/(1.0) = 57$$

where R = radio of gyration = $r/2$ for a circular section.

$$22 < KL/R < 100 \quad \therefore \ \underline{\text{Second-order effect should be considered}}.$$

TABLE 27.5 Moment Magnification and Buckling Calculations

Load Group	P (k) Case	Moment Magnification			Cracked Transformed Section		Critical Buckling		Axial Load
		Trans. M_{agy}	Long M_{agx}	Comb. M_{ag}	E^*I_y (k-ft^2)	E^*I_x (k-ft^2)	Trans. P_{cy} (k)	Long P_{cx} (k)	P(k)
I	1	1.571	1.640	1.587	1,738,699	1,619,399	5338	4972	1455
I	2	1.661	1.367	1.384	1,488,966	2,205,948	4571	6772	1364
I	3	2.765	2.059	2.364	1,392,713	1,728,396	4276	5306	2047
II		1.337	1.385	1.344	1,962,171	1,776,045	6024	5452	1137
III	1	1.406	1.403	1.405	2,046,281	2,056,470	6282	6313	1360
III	2	1.396	**1.344**	1.361	1,999,624	**2,212,829**	6139	**6793**	1305
III	3	1.738	1.671	1.708	1,901,005	2,011,763	5836	6176	1859
IV	1	1.437	1.611	1.455	1,864,312	1,494,630	5723	4588	1306
IV	2	1.448	1.349	1.377	1,755,985	2,098,586	5391	6443	1251
IV	3	1.920	1.978	1.936	1,635,757	1,585,579	5022	4868	1805
V		1.303	1.365	1.310	2,042,411	1,776,045	6270	5452	1094
VI	1	1.370	1.382	1.373	2,101,830	2,056,470	6453	6313	1308
VI	2	1.358	1.327	1.340	2,068,404	2,212,829	6350	6793	1256
VI	3	1.645	1.629	1.640	1,980,146	2,011,763	6079	6176	1788
VII	1	1.243	1.245	1.244	2,048,312	2,036,805	6288	6253	826
VII	2	1.296	1.275	1.286	1,940,100	2,053,651	5956	6305	888

Note: Column assumed to be unbraced against side sway.

The calculations for Loading Group III and Case 2 will be demonstrated in the following:

Bending in the longitudinal direction: M_x

$$\text{Factored load} = 1.3[\beta_D D + (L + I) + CF + 0.3W + WL + LF]$$

$\beta_D = 0.75$ when checking columns for maximum moment or maximum eccentricities and associated axial load. β_d in Eq. (27.8) = max dead-load moment, M_{DL}/max total moment, M_t.

$$M_{DL} = 148 \times 0.75 = 111 \text{ k-ft (151 kN·m)}$$

$$M_t = 0.75 \times 148 + 599 + 0.3 \times 192 + 86 + 295 + 2 = 1151 \text{ k-ft (1561 kN·m)}$$

$$\beta_d = 111/1151 = 0.0964$$

$$EI_x = \frac{\frac{E_c I_g}{5} + E_s I_s}{1+\beta_d} = \frac{\frac{3605\times144\times12.46}{5} + 29,000\times144\times0.2712}{1+0.0964} = 2,212,829 \text{ k-ft}^2$$

$$P_{cx} = \frac{\pi^2 EI_x}{(KL_u)^2} = \frac{\pi^2 \times 2,212,829}{(2.1\times27)^2} = 6793 \text{ kips (30,229 kN)}$$

$$C_m = 1.0 \text{ for frame braced against side sway}$$

$$\delta_s = \frac{1}{1-\frac{\sum P_u}{\phi\sum P_c}} = \frac{1}{1-\frac{1305}{0.75\times6793}} = 1.344$$

The magnified factored moment = 1.344 × 1.3 × 1151 = 2011 k-ft (2728 kN·m)

TABLE 27.6 Comparison of Factored Loads to Factored Capacity of the Column

Group	Case	Applied Factored Forces (k-ft)				Capacity (k-ft)		Ratio M_u/M	Status
		Trans. M_y	Long M_x	Comb. M	Axial P (k)	ϕM_n	ϕ		
I	1	852	475	975	1455	2924	0.75	3.00	OK
I	2	566	1972	2051	1364	2889	0.75	1.41	OK
I	3	1065	981	1448	2047	3029	0.75	2.09	OK
II		1211	546	1328	1137	2780	0.75	2.09	OK
III	1	1622	1125	1974	1360	2886	0.75	1.46	OK
III	2	1402	**2011**	2449	1305	2861	0.75	1.17	OK
III	3	1798	1558	2379	1859	3018	0.75	1.27	OK
IV	1	1022	373	1088	1306	2865	0.75	2.63	OK
IV	2	813	1245	1487	1251	2837	0.75	1.91	OK
IV	3	1136	717	1343	1805	3012	0.75	2.24	OK
V		1429	517	1519	1094	2754	0.75	1.81	OK
VI	1	1829	1065	2116	1308	2864	0.75	1.35	OK
VI	2	1617	1905	2499	1256	2842	0.75	1.14	OK
VI	3	2007	1461	2482	1788	3008	0.75	1.21	OK
VII	1	1481	963	1766	826	2372	0.67	1.34	OK
VII	2	1136	1039	1540	888	2364	0.65	1.54	OK

Notes:
1. Applied factored moments are magnified for slenderness in accordance with AASHTO LRFD.
2. The seismic forces are reduced by the load reduction factor $R = 5.0$.

$L = 27.00$ ft, $f'_c = 4.00$ ksi, $F_y = 60.0$ ksi, $A_{st} = 26.00$ in.^{2}

The analysis results with the comparison of applied moments to capacities are summarized in Table 27.6.

Column lateral reinforcement is calculated for two cases: (1) for applied shear and (2) for confinement. Typically, the confinement requirement governs. Apply Eq. 27.22 or Eq. 27.23 to calculate the confinement reinforcement. For seismic analysis, the unreduced seismic shear forces should be compared with the shear forces due to plastic hinging of columns. The smaller should be used. The plastic hinging analysis procedure is discussed elsewhere in this handbook and will not be repeated here.

The lateral reinforcement for both columns are shown as follows.

For left column:

V_u = 148 kips (659 kN) (shear due to plastic hinging governs)

ϕV_n = 167 kips (743 kN) $\therefore$ No lateral reinforcement is required for shear.

Reinforcement for confinement = $\rho_s = 0.0057$ $\therefore$ Provide #4 at 3 in. (#15 at 76 mm)

For right column:

V_u = 180 kips (801 kN) (shear due to plastic hinging governs)

ϕV_n = 167 kips (734 kN)

ϕV_s = 13 kips (58 kN) (does not govern)

Reinforcement for confinement $= \rho_s = 0.00623$ $\therefore$ Provide #4 at 2.9 in. (#15 at 74 mm)

Summary of design:

4 ft (1.22 m) diameter of column with 26-#9 (26-#30) for main reinforcement and #4 at 2.9 in. (#15 at 74 mm) for spiral confinement.

FIGURE 27.11 Typical cross sections of composite columns.

27.4.4 Steel and Composite Columns

Steel columns are not as commonly used as concrete columns. Nevertheless, they are viable solutions for some special occasions, e.g., in space-restricted areas. Steel pipes or tubes filled with concrete known as composite columns (Figure 27.11) offer the most efficient use of the two basic materials. Steel at the perimeter of the cross section provides stiffness and triaxial confinement, and the concrete core resists compression and prohibits local elastic buckling of the steel encasement. The toughness and ductility of composite columns makes them the preferred column type for earthquake-resistant structures in Japan. In China, the composite columns were first used in Beijing subway stations as early as 1963. Over the years, the composite columns have been used extensively in building structures as well as in bridges [6–9].

In this section, the design provisions of AASHTO LRFD [1] for steel and composite columns are summarized.

Compressive Resistance

For prismatic members with at least one plane of symmetry and subjected to either axial compression or combined axial compression and flexure about an axis of symmetry, the factored resistance of components in compression, P_r, is calculated as

$$P_r = \phi_c P_n$$

where

P_n = nominal compressive resistance

ϕ_c = resistance factor for compression = 0.90

The nominal compressive resistance of a steel or composite column should be determined as

$$P_n = \begin{cases} 0.66^{\lambda} F_e A_s & \text{if} \quad \lambda \le 2.25 \\ \dfrac{0.88 F_e A_s}{\lambda} & \text{if} \quad \lambda > 2.25 \end{cases} \tag{27.26}$$

in which

For steel columns:

$$\lambda = \left(\frac{KL}{r_s} \pi \right)^2 \frac{F_y}{E_e} \tag{27.27}$$

For composite column:

$$\lambda = \left(\frac{KL}{r_s}\pi\right)^2 \frac{F_e}{E_e} \tag{27.28}$$

$$F_e = F_y + C_1 F_{yr}\left(\frac{A_r}{A_s}\right) + C_2 f_c\left(\frac{A_c}{A_s}\right) \tag{27.29}$$

$$Ee = E\left[1 + \left(\frac{C_3}{n}\right)\left(\frac{A_c}{A_s}\right)\right] \tag{27.30}$$

where

A_s = cross-sectional area of the steel section (mm^2)
A_c = cross-sectional area of the concrete (mm^2)
A_r = total cross-sectional area of the longitudinal reinforcement (mm^2)
F_y = specified minimum yield strength of steel section (MPa)
F_{yr} = specified minimum yield strength of the longitudinal reinforcement (MPa)
f'_c = specified minimum 28-day compressive strength of the concrete (MPa)
E = modules of elasticity of the steel (MPa)
L = unbraced length of the column (mm)
K = effective length factor
n = modular ratio of the concrete
r_s = radius of gyration of the steel section in the plane of bending, but not less than 0.3 times the width of the composite member in the plane of bending for composite columns, and, for filled tubes,

$$C_1 = 1.0; \quad C_2 = 0.85; \quad C_3 = 0.40$$

In order to use the above equation, the following limiting width/thickness ratios for axial compression of steel members of any shape must be satisfied:

$$\frac{b}{t} \le k\sqrt{\frac{E}{F_y}} \tag{27.31}$$

where

k = plate buckling coefficient as specified in Table 27.7
b = width of plate as specified in Table 27.7
t = plate thickness (mm)

Wall thickness of steel or composite tubes should satisfy:

For circular tubes:

$$\frac{D}{t} \le 2.8\sqrt{\frac{E}{F_y}}$$

TABLE 27.7 Limiting Width-to-Thickness Ratios

	k	b
		Plates Supported along One Edge
Flanges and projecting leg or plates	0.56	Half-flange width of I-section Full-flange width of channels Distance between free edge and first line of bolts or welds in plates Full-width of an outstanding leg for pairs of angles on continuous contact
Stems of rolled tees	0.75	Full-depth of tee
Other projecting elements	0.45	Full-width of outstanding leg for single-angle strut or double-angle strut with separator Full projecting width for others
		Plates Supported along Two Edges
Box flanges and cover plates	1.40	Clear distance between webs minus inside corner radius on each side for box flanges Distance between lines of welds or bolts for flange cover plates
Webs and other plates elements	1.49	Clear distance between flanges minus fillet radii for webs of rolled beams Clear distance between edge supports for all others
Perforated cover plates	1.86	Clear distance between edge supports

For rectangular tubes:

$$\frac{b}{t} \le 1.7\sqrt{\frac{E}{F_y}}$$

where
D = diameter of tube (mm)
b = width of face (mm)
t = thickness of tube (mm)

Flexural Resistance

The factored flexural resistance, M_r, should be determined as

$$M_r = \phi_f M_n \tag{27.32}$$

where
M_n = nominal flexural resistance
ϕ_f = resistance factor for flexure, $\phi_f = 1.0$

The nominal flexural resistance of concrete-filled pipes that satisfy the limitation

$$\frac{D}{t} \le 2.8\sqrt{\frac{E}{F_y}}$$

may be determined:

$$\text{If } \frac{D}{t} < 2.0\sqrt{\frac{E}{F_y}}, \text{ then } M_n = M_{ps} \tag{27.33}$$

$$\text{If } 2.0\sqrt{\frac{E}{F_y}} < \frac{D}{t} \leq 8.8\sqrt{\frac{E}{F_y}}, \text{ then } M_n = M_{yc} \tag{27.34}$$

where

M_{ps} = plastic moment of the steel section

M_{yc} = yield moment of the composite section

Combined Axial Compression and Flexure

The axial compressive load, P_u, and concurrent moments, M_{ux} and M_{uy}, calculated for the factored loadings for both steel and composite columns should satisfy the following relationship:

$$\text{If } \frac{P_u}{P_r} < 0.2, \text{ then } \frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \tag{27.35}$$

$$\text{If } \frac{P_u}{P_r} \geq 0.2, \text{ then } \frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \tag{27.36}$$

where

P_r = factored compressive resistance

M_{rx}, M_{ry} = factored flexural resistances about x and y axis, respectively

M_{ux}, M_{uy} = factored flexural moments about the x and y axis, respectively

References

1. AASHTO, *LRFD Bridge Design Specifications,* 1st ed., American Association of State Highway and Transportation Officials, Washington, D.C., 1994.
2. Caltrans, Bridge Memo to Designers (7-10), California Department of Transportation, Sacramento, 1994.
3. White, D. W. and Hajjar, J. F., Application of second-order elastic analysis in LRFD: research to practice, *Eng. J.,* 28(4), 133, 1994.
4. Galambos, T. V., Ed., *Guide to Stability Design for Metal Structures*, 4th ed., the Structural Stability Research Council, John Wiley & Sons, New York, 1988.
5. ATC, Improved Seismic Design Criteria for California Bridges: Provisional Recommendations, Applied Technology Council, Report ATC-32, Redwood City, CA, 1996.
6. Cai, S.-H., Chinese standard for concrete-filled tube columns, in *Composite Construction in Steel and Concrete II,* Proc. of an Engineering Foundation Conference, Samuel Easterling, W. and Kim Roddis, W. M., Eds, Potosi, MO, 1992, 143.
7. Cai, S.-H., Ultimate strength of concrete-filled tube columns, in *Composite Construction in Steel and Concrete,* Proc. of an Engineering Foundation Conference, Dale Buckner, C. and Viest, I. M., Eds, Henniker, NH, 1987, 703.
8. Zhong, S.-T., New concept and development of research on concrete-filled steel tube (CFST) members, in *Proc. 2nd Int. Symp. on Civil Infrastructure Systems*, 1996.
9. CECS 28:90, *Specifications for the Design and Construction of Concrete-Filled Steel Tubular Structures,* China Planning Press, Beijing [in Chinese], 1990.
10. AISC, *Load and Research Factor Design Specification for Structural Steel Buildings and Commentary,* 2nd ed., American Institute of Steel Construction, Chicago, IL, 1993.
11. Galambos, T. V. and Chapuis, J., LRFD Criteria for Composite Columns and Beam Columns, Revised Draft, Washington University, Department of Civil Engineering, St. Louis, MO, December 1990.