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38

Seismic Design of Reinforced Concrete Bridges

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38.1 Introduction

This chapter provides an overview of the concepts and methods used in modern seismic design of reinforced concrete bridges. Most of the design concepts and equations described in this chapter are based on new research findings developed in the United States. Some background related to current design standards is also provided.

38.1.1 Two-Level Performance-Based Design

Most modern design codes for the seismic design of bridges essentially follow a two-level performance-based design philosophy, although it is not so clearly stated in many cases. The recent document ATC-32

[2] may be the first seismic design guideline based on the two-level performance design. The two level performance criteria adopted in ATC-32 were originally developed by the California Department of Transportation [5].

The first level of design concerns control of the performance of a bridge in earthquake events that have relatively small magnitude but may occur several times during the life of the bridge. The second level of design consideration is to control the performance of a bridge under severe earthquakes that have only a small probability of occurring during the useful life of the bridge. In the recent ATC-32, the first level is defined for functional evaluation, whereas the second level is for safety evaluation of the bridges. In other words, for relatively frequent smaller earthquakes, the bridge should be ensured to maintain its function, whereas the bridge should be designed safe enough to survive the possible severe events.

Performance is defined in terms of the serviceability and the physical damage of the bridge. The following are the recommended service and damage criteria by ATC-32.

1. Service Levels:
   - *Immediate service*: Full access to normal traffic is available almost immediately following the earthquake.
   - *Limited service*: Limited access (e.g., reduced lanes, light emergency traffic) is possible within days of the earthquake. Full service is restorable within months.
2. Damage levels:
   - *Minimal damage*: Essentially elastic performance.
   - *Repairable damage*: Damage that can be repaired with a minimum risk of losing functionality.
   - *Significant damage*: A minimum risk of collapse, but damage that would require closure to repair.

The required performance levels for different levels of design considerations should be set by the owners and the designers based on the importance rank of the bridge. The fundamental task for seismic design of a bridge structure is to ensure a bridge's capability of functioning at the anticipated service levels without exceeding the allowable damage levels. Such a task is realized by providing proper strength and deformation capacities to the structure and its components.

It should also be pointed out that the recent research trend has been directed to the development of more-generalized performance-based design [3,6,8,13].

38.1.2 Elastic vs. Ductile Design

Bridges can certainly be designed to rely primarily on their strength to resist earthquakes, in other words, to perform elastically, in particular for smaller earthquake events where the main concern is to maintain function. However, elastic design for reinforced concrete bridges is uneconomical, sometimes even impossible, when considering safety during large earthquakes. Moreover, due to the uncertain nature of earthquakes, a bridge may be subject to seismic loading that well exceeds its elastic limit or strength and results in significant damage. Modern design philosophy is to allow a structure to perform inelastically to dissipate the energy and maintain appropriate strength during severe earthquake attack. Such an approach can be called ductile design, and the inelastic deformation capacity while maintaining the acceptable strength is called ductility.

The inelastic deformation of a bridge is preferably restricted to well-chosen locations (the plastic hinges) in columns, pier walls, soil behind abutment walls, and wingwalls. Inelastic action of superstructure elements is unexpected and undesirable because that damage to superstructure is difficult and costly to repair and unserviceable.

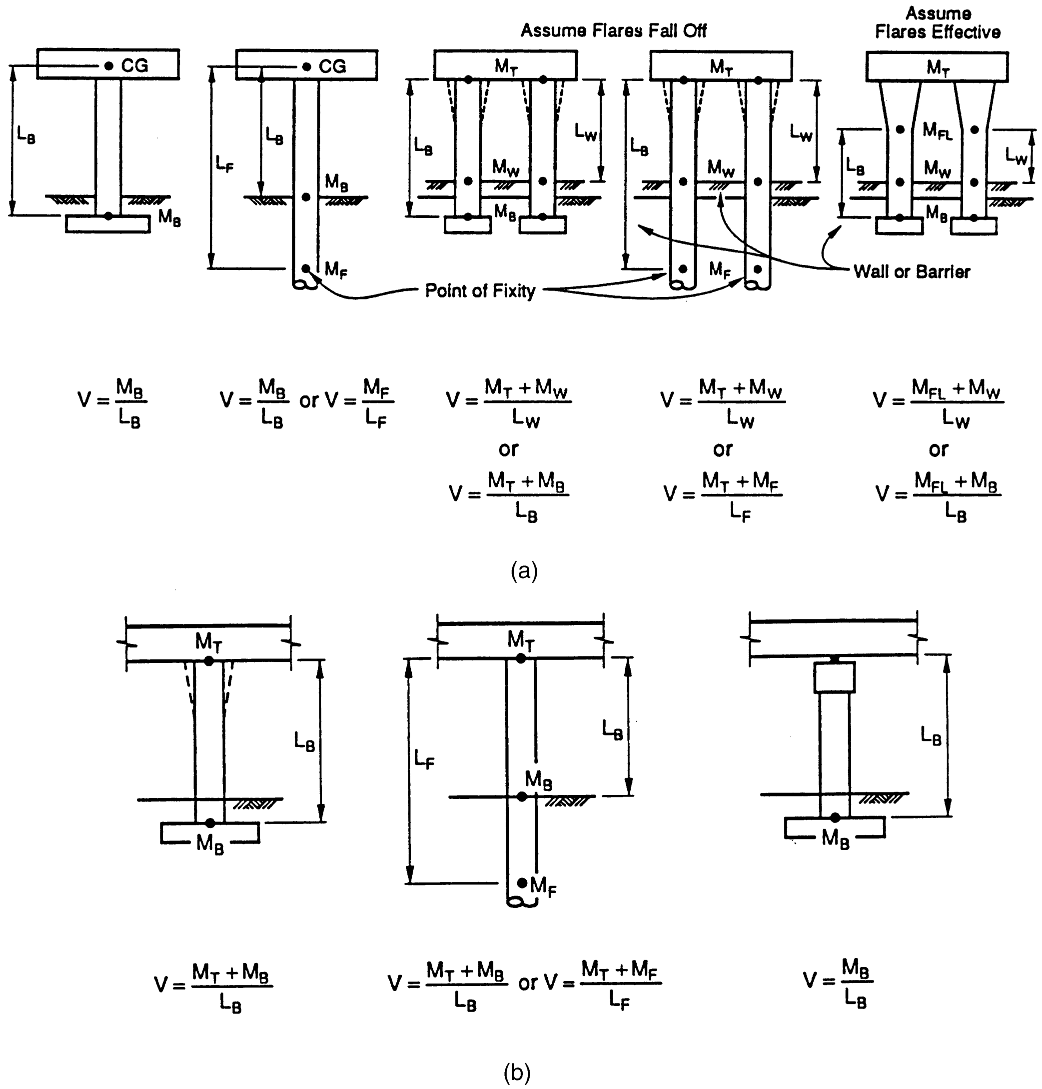


FIGURE 38.1 Potential plastic hinge locations for typical bridge bents: (a) transverse response; (b) longitudinal response. (*Source*: Caltrans, *Bridge Design Specification,* California Department of Transportation, Sacramento, June, 1990.)

38.1.3 Capacity Design Approach

The so-called capacity design has become a widely accepted approach in modern structural design. The main objective of the capacity design approach is to ensure the safety of the bridge during large earthquake attack. For ordinary bridges, it is typically assumed that the performance for lower-level earthquakes is automatically satisfied.

The procedure of capacity design involves the following steps to control the locations of inelastic action in a structure:

1. Choose the desirable mechanisms that can dissipate the most energy and identify plastic hinge locations. For bridge structures, the plastic hinges are commonly considered in columns. Figure 38.1 shows potential plastic hinge locations for typical bridge bents.

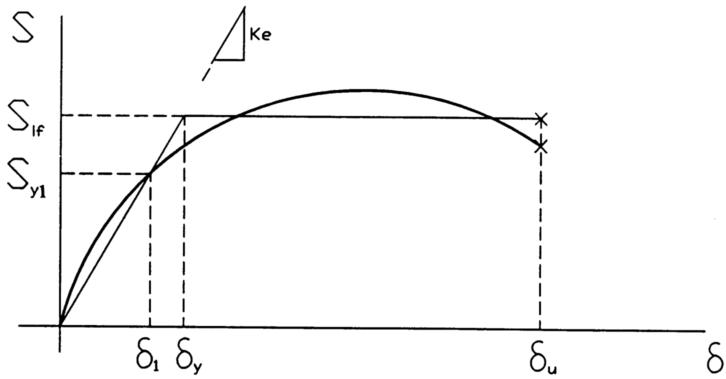


FIGURE 38.2 Idealization of column behavior.

2. Proportion structures for design loads and detail plastic hinge for ductility.
3. Design and detail to prevent undesirable failure patterns, such as shear failure or joint failure. The design demand should be based on plastic moment capacity calculated considering actual proportions and expected material overstrengths.

38.2 Typical Column Performance

38.2.1 Characteristics of Column Performance

Strictly speaking, elastic or plastic behaviors are defined for ideal elastoplastic materials. In design, the actual behavior of reinforced concrete structural components is approximated by an idealized bilinear relationship, as shown in Figure 38.2. In such bilinear characterization, the following mechanical quantities have to be defined.

Stiffness

For seismic design, the initial stiffness of concrete members calculated on the basis of full section geometry and material elasticity has little meaning, since cracking of concrete can be easily induced even under minor seismic excitation. Unless for bridges or bridge members that are expected to respond essentially elastically to design earthquakes, the effective stiffness based on cracked section is instead more useful. For example, the effective stiffness, K_e, is usually based on the cracked section corresponding to the first yield of longitudinal reinforcement,

$$K_e = S_{y1}/\delta_1 \tag{38.1}$$

where, S_{y1} and δ_1 are the force and the deformation of the member corresponding to the first yield of longitudinal reinforcement, respectively.

Strength

Ideal strength S_i represents the most feasible approximation of the "yield" strength of a member predicted using measured material properties. However, for design, such "yield" strength is conservatively assessed using *nominal strength* S_n predicted based on nominal material properties. The *ultimate* or *overstrength* represents the maximum feasible capacities of a member or a section and is predicted by taking account of all possible factors that may contribute to strength exceeding S_i or S_n. The factors include realistic values of steel yield strength, strength enhancement due to strain hardening, concrete strength increase due to confinement, strain rate, as well as actual aging, etc.

Deformation

In modern seismic design, deformation has the same importance as strength since deformation is directly related to physical damage of a structure or a structural member. Significant deformation limits are onset of cracking, onset of yielding of extreme tension reinforcement, cover concrete spalling, concrete compression crushing, or rupture of reinforcement. For structures that are expected to perform inelastically in severe earthquake, cracking is unimportant for safety design; however, it can be used as a limit for elastic performance. The first yield of tension reinforcement marks a significant change in stiffness and can be used to define the elastic stiffness for simple bilinear approximation of structural behavior, as expressed in Eq. (38.1). If the stiffness is defined by Eq. (38.1), then the yield deformation for the approximate elastoplastic or bilinear behavior can be defined as

$$\delta_y = S_{if}/S_{y1}\delta_1 \tag{38.2}$$

where, S_{y1} and δ_1 are the force and the deformation of the member corresponding to the first yield of longitudinal reinforcement, respectively; S_{if} is the idealized flexural strength for the elastoplastic behavior.

Meanwhile, the ductility factor, μ, is defined as the index of inelastic deformation beyond the yield deformation, given by

$$\mu = \delta/\delta_y \tag{38.3}$$

where δ is the deformation under consideration and δ_y is the yield deformation.

The limit of the bilinear behavior is set by an ultimate ductility factor or deformation, corresponding to certain physical events, that are typically corresponded by a significant degradation of load-carrying capacity. For unconfined member sections, the onset of cover concrete spalling is typically considered the failure. Rupture of either transverse reinforcement or longitudinal reinforcement and the crushing of confined concrete typically initiate a total failure of the member.

38.2.2 Experimentally Observed Performance

Figure 38.3a shows the lateral force–displacement hysteretic relationship obtained from cyclic testing of a well-confined column [10,11]. The envelope of the hysteresis loops can be either conservatively approximated with an elastoplastic bilinear behavior with V_{if} as the yield strength and the stiffness defined corresponding to the first yield of longitudinal steel. The envelope of the hysteresis loops can also be well simulated using a bilinear behavior with the second linear portion account for the overstrength due to strain hardening. Final failure of this column was caused by the rupture of longitudinal reinforcement at the critical sections near the column ends.

The ductile behavior shown in Figure 38.3a can be achieved by following the capacity design approach with ensuring that a flexural deformation mode to dominate the behavior and other nonductile deformation mode be prevented. As a contrary example to ductile behavior, Figure 38.3b shows a typical poor behavior that is undesirable for seismic design, where the column failed in a brittle manner due to the sudden loss of its shear strength before developing yielding, V_{if}. Bond failure of reinforcement lap splices can also result in rapid degradation of load-carrying capacity of a column.

An intermediate case between the above two extreme behaviors is shown in Figure 38.3c, where the behavior is somewhat premature for full ductility due to the fact that the tested column failed in shear upon cyclic loading after developing its yield strength but at a smaller ductility level than that shown in Figure 38.3a. Such premature behavior is also not desirable.

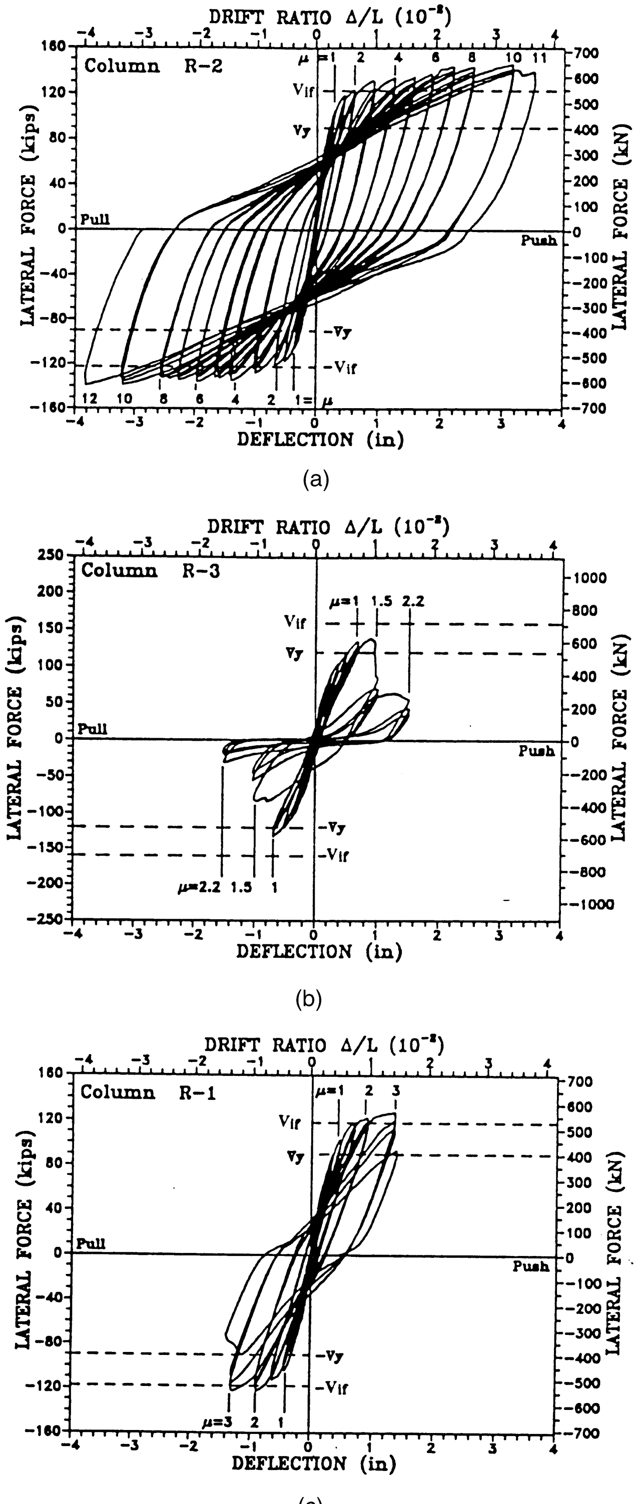


FIGURE 38.3 Typical experimental behaviors for (a) well-confined column; (b) column failed in brittle shear; (c) column with limited ductility. (*Source*: Priestley, M. J. N. et al., *ACI Struct. J.*, 91C52, 537–551, 1994. With permission.)

38.3 Flexural Design of Columns

38.3.1 Earthquake Load

For ordinary, regular bridges, the simple force design based on equivalent static analysis can be used to determine the moment demands on columns. Seismic load is assumed as an equivalent static horizontal force applied to individual bridge or frames, i.e.,

$$F_{eq} = ma_g \tag{38.4}$$

where m is the mass; a_g is the design peak acceleration depended on the period of the structure. In the Caltrans BDS [4] and the ATC-32 [2], the peak ground acceleration a_g is calculated as 5% damped elastic acceleration response spectrum at the site, expressed as ARS, which is the ratio of peak ground acceleration and the gravity acceleration g. Thus the equivalent elastic force is

$$F_{eq} = mg(\text{ARS}) = W(\text{ARS}) \tag{38.5}$$

where W is the dead load of bridge or frame.

Recognizing the reduction of earthquake force effects on inelastically responding structures, the elastic load is typically reduced by a period-dependent factor. Using the Caltrans BDS expression, the design force is found:

$$F_d = W(\text{ARS})/Z \tag{38.6}$$

This is the seismic demand for calculating the required moment capacity, whereas the capability of inelastic response (ductility) is ensured by following a capacity design approach and proper detailing of plastic hinges. Figure 38.4a and b shows the Z factor required by current Caltrans BDS and modified Z factor by ATC-32, respectively. The design seismic forces are applied to the structure with other loads to compute the member forces. A similar approach is recommended by the AASHTO-LRFD specifications.

The equivalent static analysis method is best suited for structures with well-balanced spans and supporting elements of approximately equal stiffness. For these structures, response is primarily in a single mode and the lateral force distribution is simply defined. For unbalanced systems, or systems in which vertical accelerations may be significant, more-advanced methods of analysis such as elastic or inelastic dynamic analysis should be used.

38.3.2 Fundamental Design Equation

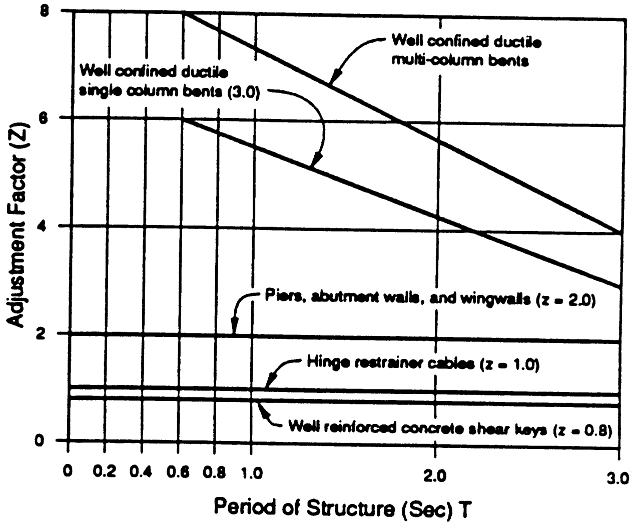
The fundamental design equation is based on the following:

$$\phi R_n \geq R_u \tag{38.7}$$

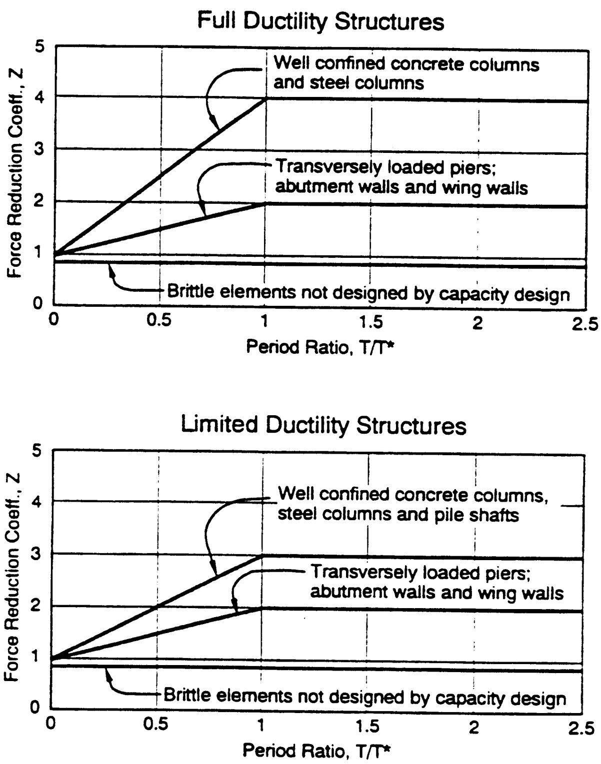
where R_u is the strength demand; R_n is the nominal strength; and ϕ is the strength reduction factor.

38.3.3 Design Flexural Strength

Flexural strength of a member or a section depends on the section shape and dimension, amount and configuration of longitudinal reinforcement, strengths of steel and concrete, axial load magnitude, lateral confinement, etc. In most North American codes, the design flexural strength is conservatively calculated based on nominal moment capacity M_n following the ACI code recommendations [1]. The ACI approach is based on the following assumptions:



(a)



(b)

FIGURE 38.4 Force reduction coefficient Z (a) Caltrans BDS 1990; (b) ATC-32.

1. A plane section remains plane even after deformation. This implies that strains in longitudinal reinforcement and concrete are directly proportional to the distance from the neutral axis.
2. The section reaches the capacity when compression strain of the extreme concrete fiber reaches its maximum usable strain that is assumed to be 0.003.
3. The stress in reinforcement is calculated as the following function of the steel strain

$$f_s = -f_y \quad \text{for} \quad \varepsilon < -\varepsilon_y \tag{38.8a}$$

$$f_s = E_s \varepsilon \quad \text{for} \quad -\varepsilon_y \le \varepsilon \le \varepsilon_y \tag{38.8b}$$

$$f_s = f_y \quad \text{for} \quad \varepsilon > \varepsilon_y \tag{38.8c}$$

   where ε_y and f_y are the yield strain and specified strength of steel, respectively; E_s is the elastic modulus of steel.
4. Tensile stress in concrete is ignored.
5. Concrete compressive stress and strain relationship can be assumed to be rectangular, trapezoidal, parabolic, or any other shape that results in prediction of strength in substantial agreement with test results. This is satisfied by an equivalent rectangular concrete stress block with an average stress of 0.85 f_c', and a depth of $\beta_1 c$, where c is the distance from the extreme compression fiber to the neutral axis, and

$$0.65 \le \beta_1 = 0.85 - 0.05 \frac{f_c' - 28}{7} \le 0.85 \quad [\, f_c' \text{ in } MPa] \tag{38.9}$$

In calculating the moment capacity, the equilibrium conditions in axial direction and bending must be used. By using the equilibrium condition that the applied axial load is balanced by the resultant axial forces of concrete and reinforcement, the depth of the concrete compression zone can be calculated. Then the moment capacity can be calculated by integrating the moment contributions of concrete and steel.

The nominal moment capacity, M_n, reduced by a strength reduction factor ϕ (typically 0.9 for flexural) is compared with the required strength, M_u, to determine the feasibility of longitudinal reinforcement, section dimension, and adequacy of material strength.

Overstrength

The calculation of the nominal strength, M_n, is based on specified minimum material strength. The actual values of steel yield strength and concrete strength may be substantially higher than the specified strengths. These and other factors such as strain hardening in longitudinal reinforcement and lateral confinement result in the actual strength of a member perhaps being considerably higher than the nominal strength. Such overstrength must be considered in calculating ultimate seismic demands for shear and joint designs.

38.3.4 Moment–Curvature Analysis

Flexural design of columns can also be carried out more realistically based on moment–curvature analysis, where the effects of lateral confinement on the concrete compression stress–strain relationship and the strain hardening of longitudinal reinforcement are considered. The typical assumptions used in the moment–curvature analysis are as follows:

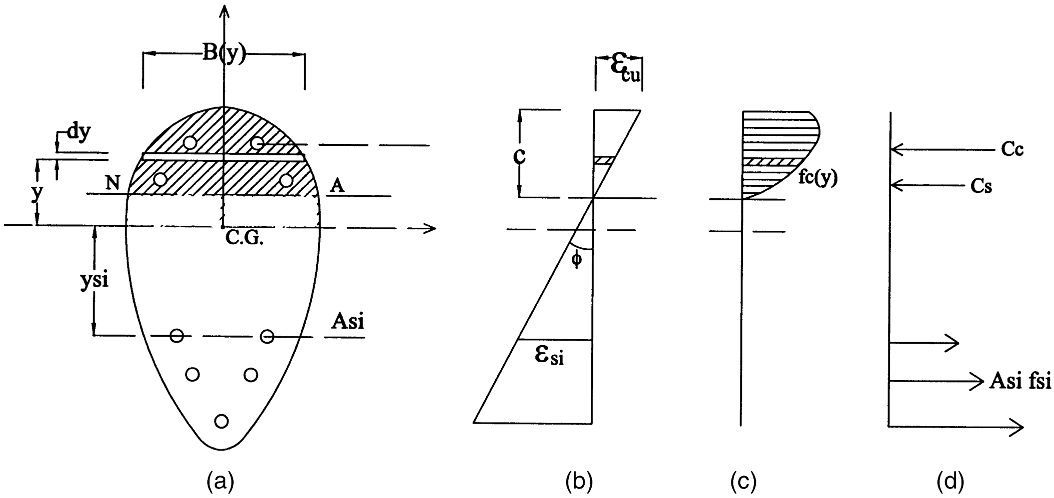


FIGURE 38.5 Moment curvature analysis: (a) generalized section; (b) strain distribution; (c) concrete stress distribution; (d) rebar forces.

1. A plane section remains plane even after deformation. This implies that strains in longitudinal reinforcement and concrete are directly proportional to the distance from the neutral axis.
2. The stress–strain relationship of reinforcement is known and can be expressed as a general function, $f_s = F_s(\varepsilon_s)$.
3. The stress–strain relationship of concrete is known and can be expressed as a general function, $f_c = F_c(\varepsilon_c)$. The tensile stress of concrete is typically ignored for seismic analysis but can be considered if the uncracked section response needs to be analyzed. The compression stress–strain relationship of concrete should be able to consider the effects of confined concrete (for example, Mander et al. [7]).
4. The resulting axial force and moment of concrete and reinforcement are in equilibrium with the applied external axial load and moment.

The procedure for moment–curvature analysis is demonstrated using a general section shown in Figure 38.5a. The distributions of strains and stresses in the cracked section corresponding to an arbitrary curvature, ϕ, are shown in Figure 38.5b, c, and d, respectively.

Corresponding to the arbitrary curvature, ϕ, the strains of concrete and steel at an arbitrary position with a distance of y to the centroid of the section can be calculated as

$$\varepsilon = \phi(y - y_c + c) \tag{38.10}$$

where y_c is the distance of the centroid to the extreme compression fiber and c is the depth of compression zone. Then the corresponding stresses can be determined using the known stress–strain relationships for concrete and steel.

Based on the equilibrium conditions, the following two equations can be established,

$$P = \sum A_{si} f_{si} + \int_A B(y) f_c(y) dy \tag{38.11}$$

$$M = \sum A_{si} f_{si} y_{si} + \int_A B(y) f_c(y) y dy \tag{38.12}$$

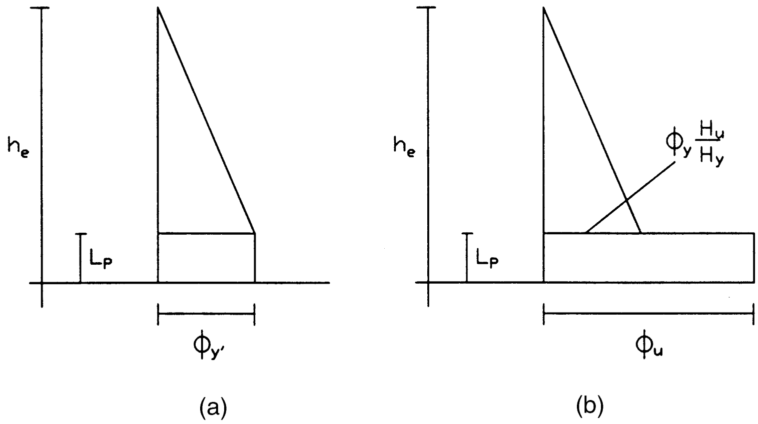


FIGURE 38.6 Idealized curvature distributions: (a) corresponding to first yield of reinforcement; (b) at ultimate flexural failure.

Using the axial equilibrium condition, the depth of the compression zone, *c*, corresponding to curvature, ϕ, can be determined, and then the corresponding moment, *M*, can be calculated. The actual computation of moment–curvature relationships is typically done by computer programs (for example, "SC-Push" [15]).

38.3.5 Transverse Reinforcement Design

In most codes, the ductility of the columns is ensured by proper detailing of transverse confinement steel. Transverse reinforcement can also be determined by a trial-and-error procedure to satisfy the required displacement member ductility levels. The lateral displacement of a member can be calculated by an integration of curvature and rotational angle along the member. This typically requires the assumption of curvature distributions along the member. Figure 38.6 shows the idealized curvature distributions at the first yield of longitudinal reinforcement and the ultimate condition for a column.

Yield Conditions: The horizontal force at the yield condition of the bilinear approximation is taken as the ideal capacity, H_{if}. Assuming linear elastic behavior up to conditions at first yield of the longitudinal reinforcement, the displacement of the column top at first yield due to flexure alone is

$$\Delta'_{yf} = \int_0^{h_e} \frac{\phi'_y}{h_e} (h_e - y)^2 dy \quad [f_y \text{ in } MPa] \tag{38.13}$$

where, ϕ'_y is the curvature at first yield and h_e is the effective height of the column. Allowing for strain penetration of the longitudinal reinforcement into the footing, the effective column height can be taken as

$$h_e = h + 0.022 f_y d_{bl} \tag{38.14}$$

where *h* is the column height measured from one end of a column to the point of inflexion, d_{bl} is the longitudinal reinforcement nominal diameter, and f_y is the nominal yield strength of rebar. The flexural component of yield displacement corresponding to the yield force, H_{if}, for the bilinear approximation can be found by extrapolating the first yield displacement to the ideal flexural strength, giving

$$\Delta_{yf} = \Delta'_{yf} H_{if} / H_y \tag{38.15}$$

where H_y is the horizontal force corresponding to first yield.

Ultimate Conditions: Ultimate conditions for the bilinear approximation can be taken to be the horizontal force, H_u, and displacement, Δ_{uf}, corresponding to development of an ultimate curvature, ϕ_u, based on moment-curvature analysis of the critical sections considering reinforcement strain-hardening and concrete confinement as appropriate. Ultimate curvature corresponds to the more critical ultimate concrete compression strain [7] based on an energy-balance approach, and maximum longitudinal reinforcement tensile strain, taken as $0.7\varepsilon_{su}$, where ε_{su} is the steel strain corresponding to maximum steel stress. The "ultimate" displacement is thus

$$\Delta_{uf} = \theta_p (h - 0.5L_p) + \Delta_y \frac{H_u}{H_{if}} \tag{38.16}$$

where the plastic rotation, θ_p, can be estimated as

$$\theta_p = \left(\phi_u - \phi'_y \frac{H_u}{H_y} \right) L_p \tag{38.17}$$

In Eqs. (38.16) and (38.17), L_p is the equivalent plastic hinge length appropriate to a bilinear approximation of response, given by

$$L_p = 0.08h + 0.022 f_y d_{bl} \quad [f_y \text{ in } MPa] \tag{38.18}$$

Based on the displacement check, the required transverse reinforcement within the plastic hinge region of a column to satisfy the required displacement ductility demand can be determined using a trial-and-error procedure.

Similarly, transverse reinforcement in the potential plastic hinge region of a column can be determined based on curvature ductility requirements. The ATC-32 document [2] suggests the following minimum requirement for the volumetric ratio, ρ_s, of spiral or circular hoop reinforcement within plastic hinge regions of columns based on a curvature ductility factor of 13.0 or larger.

$$\rho_s = 0.16 \frac{f'_{ce}}{f_{ye}} \left[0.5 + \frac{1.25P_e}{f'_{ce} A_g} \right] + 0.13(\rho_l - 0.01) \tag{38.19}$$

where f'_{ce} is the expected concrete compression strength taken as $1.3\, f'_c$; f_{ye} is the expected steel yield strength taken as $1.1 f_y$; P_e is axial load; A_g is cross-sectional area of column; and ρ_l is longitudinal reinforcement ratio. For transverse reinforcement outside the potential plastic hinge region, the volumetric ratio can be reduced to 50% of the amount given by Eq. (38.19). The length of the plastic hinge region should be the greater of (1) the section dimension in the direction considered or (2) the length of the column portion over which the moment exceeds 80% of the moment at the critical section. However, for columns with axial load ratio $P_e / f'_{ce} A_g > 0.3$, this length should be increased by 50%. Requirements for cross ties and hoops in rectilinear sections can also be found in ATC-32 [2].

38.4 Shear Design of Columns

38.4.1 Fundamental Design Equation

As discussed previously, shear failure of columns is the most dangerous failure pattern that typically can result in the collapse of a bridge. Thus, design to prevent shear failure is of particular importance. The general design equation for shear strength can be described as

$$\phi V_n > V_u \quad (38.20)$$

where V_u is the ultimate shear demands; V_n is the nominal shear resistant; and ϕ is the strength reduction factor for shear strength.

The calculation of the ultimate shear demand, V_u, has to be based on the equilibrium of the internal forces corresponding to the maximum flexural capacity, which is calculated taking into consideration all factors for overstrength. Figure 38.1 also includes equations for determining V_u for typical bridge bents.

38.4.2 Current Code Shear Strength Equation

Following the ACI code approach [1], the shear strength of axially loaded members is empirically expressed as

$$V_n = V_c + V_s \quad (38.21)$$

In the two-term additive equations, V_c is the shear strength contribution by concrete shear resisting mechanism and V_s is the shear strength contribution by the truss mechanism provided by shear reinforcement. Concrete shear contribution, V_c, is calculated as

$$V_c = 0.166\sqrt{f_c'}\left(1 + \frac{P}{13.8A_g}\right)A_e \quad (38.22)$$

where for columns the effective shear area A_e can be taken as 80% of the cross-sectional area, A_g; P is the applied axial compression force. Note that $\sqrt{f_c'} \le 0.69$ MPa.

The contribution of truss mechanism is taken as

$$V_s = \frac{A_v f_y d}{s} \quad (38.23)$$

where A_v is the total transverse steel area within spacing s; f_y is yield strength of transverse steel; and d is the effective depth of the section.

Comparisons with existing test data indicate that actual shear strengths of columns often exceeds the design shear strength based on the ACI approach, in many cases by more than 100%.

38.4.3 Refined Shear Strength Equations

A refined shear strength equation that agrees significantly well with tests was proposed by Priestley et al. [12] in the following three-term additive expression:

$$V_n = V_c + V_s + V_a \quad (38.24)$$

where V_c and V_s are shear strength contributions by concrete shear resisting mechanism and the truss mechanism, respectively; the additional term, V_a, represents the shear resistance by the arch mechanism, provided mainly by axial compression.

$$V_c = k\sqrt{f_c'}A_e \quad (38.25)$$

and k depends on the displacement ductility factor μ_Δ, which reduces from 0.29 in MPa units (3.5 in psi) for $\mu_\Delta \leq 2.0$ to 0.1 in MPa units (1.2 in psi) for $\mu_\Delta \geq 4.0$; A_e is taken as $0.8A_g$. The shear strength contribution by truss mechanism for circular columns is given by,

$$V_s = \frac{\pi}{2}\frac{A_{sp} f_y (d-c)}{s}\cot\theta \tag{38.26}$$

where A_{sp} is cross-sectional area of spiral or hoop reinforcement; d is the effective depth of the section; c is the depth of compression zone at the critical section; s is spacing of spiral or hoop; and θ is the angle of truss mechanism, taken as 30°. In general, the truss mechanism angle θ should be considered a variable for different column conditions. Note that $(d - x)\cot\theta$ in Eq. (38.26) essentially represents the length of the critical shear crack that intersects with the critical section at the position of neutral axis, as shown in Figure 38.7a [17]. Equation (38.26) is approximate for circular columns with circular hoops or spirals as shear reinforcement. As shown in Figure 38.7b, the shear contribution of circular hoops intersected by a shear crack can be calculated by integrating the components of their hoop tension forces, $A_{sp}f_s$, in the direction of the applied shear, given by [18],

$$V_s = 2\int_{c-d/2} \frac{2A_{sp} f_s}{sd}\cot\theta\sqrt{(d/2)^2 - x^2}\,dx \tag{38.27}$$

By further assuming that all the hoops intersected by the shear crack develop yield strength, f_y, the integration results in

$$V_s = \frac{A_{sp} f_y d}{s}\cot\theta\left[\left(1-\frac{2c}{d}\right)\sqrt{\frac{c}{d}\left(1-\frac{c}{d}\right)} + \frac{1}{2}\left(1-\frac{2c}{d}\right)^2 \arcsin\left(1-\frac{2c}{d}\right) + \frac{\pi}{4}\right] \tag{38.28}$$

The shear strength enhancement by axial load is considered to result from an inclined compression strut, given by

$$V_a = P\tan\alpha = \frac{D-x}{2D(M/VD)}P \tag{38.29}$$

where D is section depth or diameter; x is the compression zone depth which can be determined from flexural analysis; and (M/VD) is the shear aspect ratio.

38.5 Moment-Resisting Connection Between Column and Beam

Connections are key elements that maintain the integrity of overall structures; thus, they should be designed carefully to ensure the full transfer of seismic forces and moments. Because of their importance, complexity, and difficulty of repair if damaged, connections are typically provided with a higher degree of safety and conservativeness than column or beam members. Current Caltrans BDS and AASHTO-LRFD do not provide specific design requirements for joints, except requiring the lateral reinforcement for columns to be extended into column/footing or column/cap beam joints. A new design approach recently developed by Priestley and adopted in the ATC-32 design guidelines is summarized below.

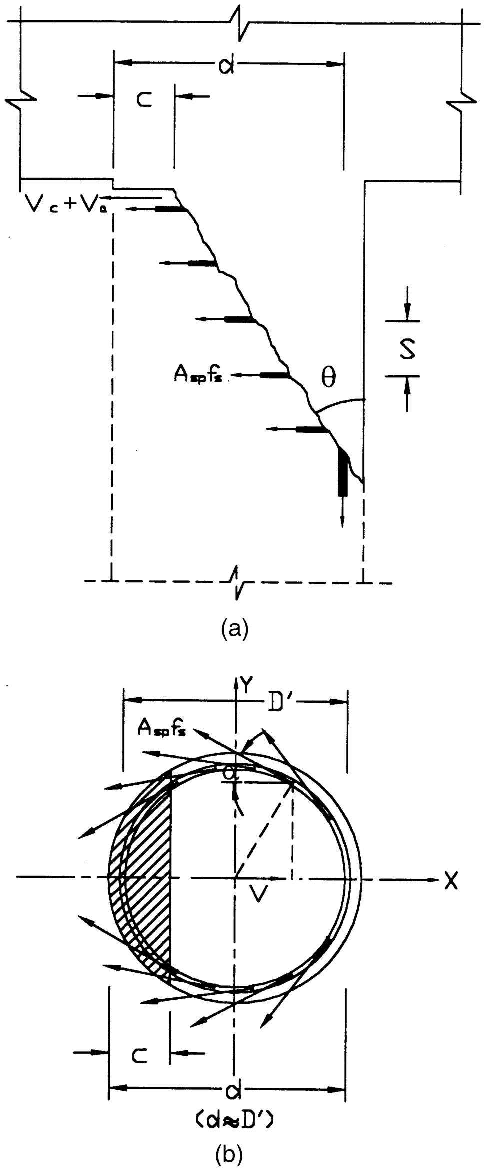


FIGURE 38.7 Shear resisting mechanism of circular hoops or spirals: (a) critical shear section; (b) stresses in hoops or spirals intersected with shear crack. (*Source*: Xiao et al. 1998. With permission.)

38.5.1 Design Forces

In moment-resisting frame structures, the force transfer typically results in sudden changes (magnitude and direction) of moments at connections. Because of the relatively small dimensions of joints, such sudden moment changes cause significant shear forces. Thus, joint shear design is the major concern of the column and beam connection, as well as that the longitudinal reinforcements of beams and columns are to be properly anchored or continued through the joint to transmit the moment. For seismic design, joint shear forces can be calculated based on the equilibrium condition using forces generated by the maximum plastic moment acting on the boundary of connections. In the following section, calculations for joint shear forces in the most common connections in bridge structures are discussed [13].

38.5.2 Design of Uncracked Joints

Joints can be conservatively designed based on elastic theory for not permitting cracks. In this approach, the principal tensile stress within a connection is calculated and compared with allowable tensile strength. The principal tensile stress, p_t, can be calculated as

$$p_t = \frac{f_h + f_v}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{hv}^2} \tag{38.30}$$

where f_h and f_v are the average axial stresses in the horizontal and vertical directions within the connection and v_{hv} is the average shear stress. In a typical joint, f_v is provided by the column axial force P_e. An average stress at the midheight of the joint should be used, assuming a 45° spread away from the boundaries of the column in all directions. The horizontal axial stress f_h is based on the mean axial force at the center of the joint, including effects of prestress, if present.

The joint shear stress, v_{hv}, can be estimated as

$$v_{hv} = \frac{M_p}{h_b h_c b_{je}} \tag{38.31}$$

where M_p is the maximum plastic moment, h_b is the beam depth, h_c is the column lateral dimension in the direction considered, and b_{je} is the effective joint width, found using a 45° spread from the column boundaries, as shown in Figure 38.8.

Based on theoretical consideration [13] and experimental observation , it is assumed that onset of diagonal cracking can be induced if the principal tensile stress exceed 0.29 $\sqrt{f_c'}$ MPa (3.5 $\sqrt{f_c'}$ in psi units). When the principal tensile stress is less than $p_t = 0.29\sqrt{f_c'}$ MPa, the minimum amount of horizontal joint shear reinforcement capable of transferring 50% of the cracking stress resolved to the horizontal direction should be provided.

On the other hand, the principal compression stress, p_c, calculated based on the following equation should not exceed $p_c = 0.25\sqrt{f_c'}$, to prevent possible joint crushing.

$$p_c = \frac{f_h + f_v}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{hv}^2} \tag{38.32}$$

38.5.3 Reinforcement for Joint Force Transfer

Diagonal cracks are likely to be induced if the principal tensile stress exceed 0.29 $\sqrt{f_c'}$ MPa; thus, additional vertical and horizontal joint reinforcement is needed to ensure the force transfer within the joint. Unlike the joints in building structures where more rigorous dimension restraints exist, the joints in bridges can be reinforced with vertical and horizontal bars outside the zone where the column and beam longitudinal bars anchored to reduce the joint congestion. Figure 38.9 shows the force-resisting mechanism in a typical knee joint.

Vertical Reinforcement

On each side of the column or pier wall, the beam that is subjected to moments and shear will have vertical stirrups, with a total area $A_{jv} = 0.16A_{st}$, where A_{st} is the total area of column reinforcement anchored in the joint. The vertical stirrups shall be located within a distance $0.5D$ or $0.5h$ from the column or pier wall face, and distributed over a width not exceeding $2D$. As shown in Figure 38.9, reinforcement A_{jv} is required to provide the tie force T_s resisting the vertical component of strut $D2$. It is also clear from Figure 38.9, all the longitudinal bars contributing to the beam flexural strength should be clamped by the vertical stirrups.

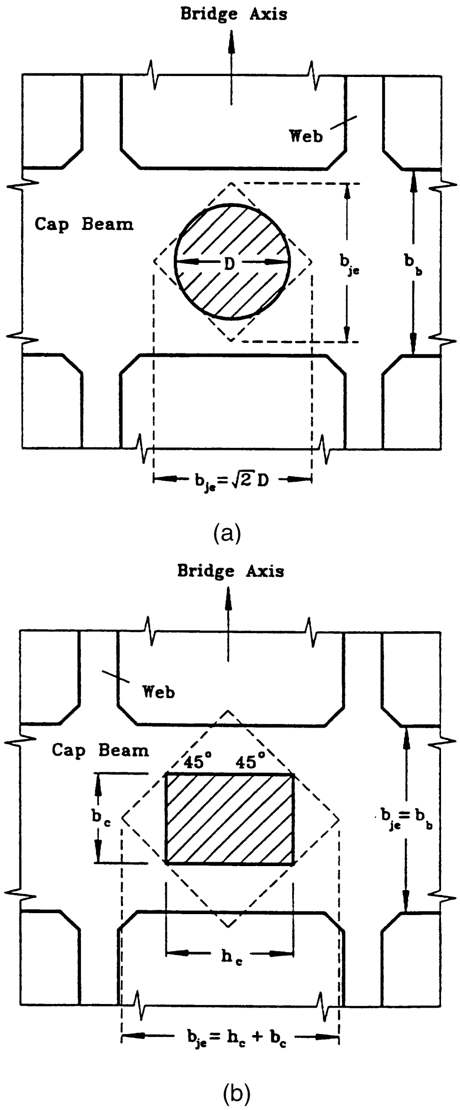


FIGURE 38.8 Effective joint width for joint shear stress calculations: (a) circular column; (b) rectangular column (Priestley et al. 1996; ATC-32, 1996).

Horizontal Reinforcement

Additional cap-beam bottom reinforcement is required to provide the horizontal resistance of the strut D2, shown in Figure 38.9. The suggested details are shown in Figure 38.9. This reinforcement may be omitted in prestressed or partially prestressed cap beams if the prestressed design force is increased by the amount needed to provide an equivalent increase in cap beam moment capacity to that provided by this reinforcement.

Hoop or Spiral Reinforcement

The hoop or spiral reinforcement is required to provide adequate confinement of the joint, and to resist the net outward thrust of struts *D*1 and *D*2 shown in Figure 38.9. The suggested volumetric ratio of column joint hoop or spiral reinforcement to be carried into the joint is

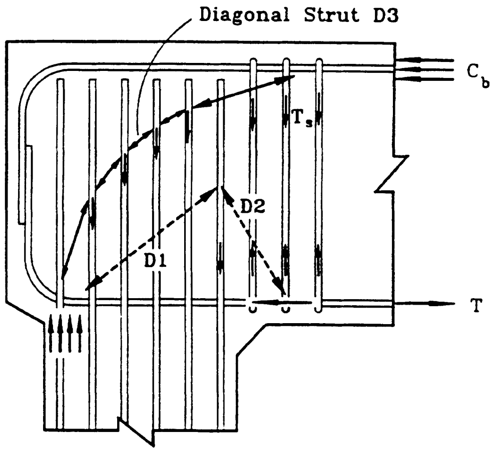


FIGURE 38.9 Joint force transfer. (*Source*: ATC-32 [2]).

$$\rho_s \geq \frac{0.4A_{st}}{l_{ac}^2} \tag{38.33}$$

where l_{ac} is the anchorage length of the column longitudinal reinforcement in the joint.

38.6 Column Footing Design

Bridge footing designs in 1950s to early 1970s were typically based on elastic analysis under relatively low lateral seismic input compared with current design provisions. As a consequence, footings in many older bridges are inadequate for resisting the actual earthquake force input corresponding to column plastic moment capacity. Seismic design for bridge structures has been significantly improved since the 1971 San Fernando earthquake. For bridge footings, a capacity design approach has been adopted by using the column ultimate flexural moment, shear, and axial force as the input to determine the required flexural and shear strength as well as the pile capacity of the footing. However, the designs for flexure and shear of footings are essentially based on a one-way beam model, which lacks experimental verification. The use of a full footing width for the design is nonconservative. In addition, despite the requirement of extending the column transverse reinforcement into the footing, there is a lack of rational consideration of column/footing joint shear in current design [16]. Based on large-scale model tests, Xiao et al. have recommended the following improved design for bridge column footings [16,18].

38.6.1 Seismic Demand

The footings are considered as under the action of column forces, due to the superimposed loads, resisted by an upward pressure exerted by the foundation materials and distributed over the area of the footing. When piles are used under footings, the upward reaction of the foundation is considered as a series of concentrated loads applied at the pile centers. For seismic design, the maximum probable moment, M_p, calculated based on actual strength with consideration of strain hardening of steel and enhancement due to confinement, with the associated axial and lateral loads are applied at the column base as the seismic inputs to the footing. Note that per current Caltrans BDS the maximum probable moment can be taken as 1.3 times the nominal moment capacity of the column, M_n, if the axial load of the column is below its balanced load, P_b. The numbers of piles and the internal forces in the footing are then determined from the seismic input. The internal moment and shear force of the footing can be determined based on the equilibrium conditions of the applied forces at the column base corresponding to the maximum moment capacity and the pile reaction forces.

38.6.2 Flexural Design

For flexural reinforcement design, the footing critical section is taken at the face of the column, pier wall, or at the edge of hinge. In case of columns that are not square or rectangular, the critical sections are taken at the side of the concentric square of equivalent area. The flexural reinforcements near top or the bottom of the footing to resist positive and negative critical moments should be calculated and placed based on the following effective footing width [16].

$$B_{feff} = B_c + 2d_f \quad \text{for rectangular columns} \tag{38.34a}$$

$$B_{feff} = D_c + 2d_f \quad \text{for circular columns} \tag{38.34b}$$

where B_{feff} is the footing effective width; B_c is the rectangular column width; d_f is the effective footing depth; and D_c is the circular column diameter. The minimum reinforcement must satisfy minimum flexural reinforcement requirements. The top reinforcement must also satisfy the requirement for shrinkage and temperature.

38.6.3 Shear Design

For shear, reinforcement of the footing should be designed against the critical shear force at the column face. As with flexure, the effective width of the footing should be used. The minimum shear reinforcement for column footings is vertical No. 5 (nominal diameter = ⅝ in. or 15.9 mm) at 12 in. (305 mm) spacing in each direction in a band between d_f, of the footing from the column surface and 6 in. (152 mm) maximum from the column reinforcement. Shear reinforcement must be hooked around the top and bottom flexure reinforcement in the footing. Inverted J stirrups with a 180° hook at the top and a 90° hook at the bottom are commonly used in California.

38.6.4 Joint Shear Cracking Check

If the footing is sufficiently large, then an uncracked joint may be designed by keeping the principal tensile stress in the joint region below the allowable cracking strength. Average principal tensile stress in the joint region can be calculated from the equivalent joint shear stress, v_{jv}, and the average vertical stress, f_a, by use of Mohr's circle for stress as

$$f_t = -f_a / 2 + \sqrt{(f_a / 2)^2 + {v_{jv}}^2} \tag{38.35}$$

It is suggested that f_a be based on the average effective axial compressive stress at middepth of the footing:

$$f_a = \frac{W_t}{A_{\text{eff}}} \tag{38.36}$$

where the effective area, A_{eff}, over which the total axial load W_t at the column base is distributed, is found from a 45-degree spread of the zone of influence as

$$A_{\text{eff}} = (B_c + d_f)(D_c + d_f) \quad \text{for rectangular column} \tag{38.37a}$$

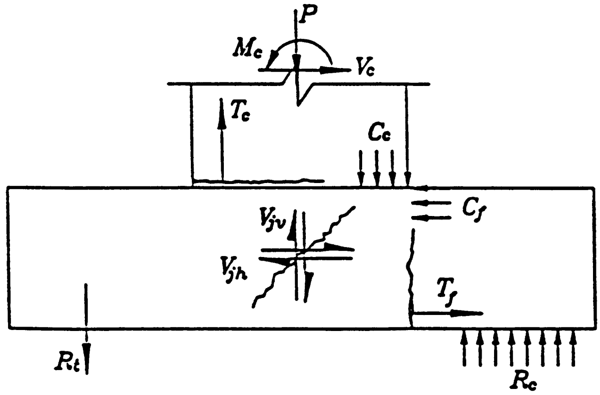


FIGURE 38.10 Shear Force in Column/Footing Joint. (*Source*: Xiao, Y. et al., *ACI Struct. J.*, 93(1), 79–94, 1996. With permission.)

$$A_{\text{eff}} = \pi(D_c + d_f)^2 / 4 \qquad \text{for circular column} \tag{38.37b}$$

where D_c is the overall section depth of rectangular column or the diameter of circular column; B_c is the section width of rectangular column; and d_f is the effective depth of the footing. As illustrated in Figure 38.10, the vertical joint shear V_{jv} can be assessed by subtracting the footing shear force due to the hold-down force, R_t, in the tensile piles and the footing self-weight, W_{fl}, outside the column tension stress resultant, from the total tensile force in the critical section of the column:

$$V_{jv} = T_c - (R_t + W_{fl}) \tag{38.38}$$

Considering an effective joint width, b_{jeff}, the average joint shear stress, v_{jv}, can be calculated as follows:

$$v_{jv} = \frac{V_{jv}}{b_{\text{jeff}} d_f} \tag{38.39}$$

where the effective joint width, b_{jeff}, can be assumed as the values given by Eq. (38.39), which is obtained based on the St. Venant 45-degree spread of influence between the tension and compression resultants in column critical section.

$$b_{\text{jeff}} = B_c + D_c \qquad \text{for rectangular column} \tag{38.40a}$$

$$b_{\text{jeff}} = \sqrt{2} D_c \qquad \text{for circular column} \tag{38.40b}$$

Joint shear distress is expected when the principal tensile stress given by Eq. (38.35) induced in the footing exceeds the direct tension strength of the concrete, which may conservatively be taken as 0.29 $\sqrt{f_c'}$ MPa (or 3.5 $\sqrt{f_c'}$ psi) [9], where f_c' is the concrete cylinder compressive strength. The minimum joint shear reinforcement should be provided even when the principal tensile stress is less than the tensile strength. This should be satisfied by simply extending the column transverse confinement into the footing.

38.6.5 Design of Joint Shear Reinforcement

When a footing cannot be prevented from joint shear cracking, additional vertical stirrups should be added around the column. For a typical column/footing designed to current standards, the assumed strut-and-tie model is shown in Figure 38.11a. The force inputs to the footing corresponding to the ultimate moment of the column critical section are the resultant tensile force, T_c, resultant compressive force, C_c, and the shear force, V_c, as shown in Figure 38.11a. The resultant tensile force, T_c, is resisted by two struts, $C1$ and $C2$, inside the column/footing joint region and a strut $C3$, outside the joint. Strut $C1$, is balanced by a horizontal tie, $T1$, provided by the transverse reinforcement of the column inside the footing and the compression zone of the critical section. Struts $C2$, and $C3$, are balanced horizontally at the intersection with the resultant tensile force, T_c. The internal strut $C2$, is supported at the compression zone of the column critical section. The external strut $C3$, transfers the forces to the ties, $T2$, $T3$, provided by the stirrups outside the joint and the top reinforcement, respectively. The forces are further transferred to the tensile piles through struts and ties, $C4$, $C5$, $C6$, and $T4$, $T5$, $T6$, $T7$. In the compression side of the footing, the resultant compressive force, C_c, and the shear force, V_c, are resisted mainly by compression struts, $C9$, $C10$, which are supported on the compression piles. It should be pointed out that the numbers and shapes of the struts and ties may vary for different column/footings.

As shown in Figure 38.11a, the column resultant tensile force, T_c, in the column/footing joint is essentially resisted by a redundant system. The joint shear design is to ensure that the tensile force, T_c, is resisted sufficiently by the internal strut and ties. The resisting system reaches its capacity, R_{ju}, when the ties, $T1$, $T2$, develop yielding. Although tie, $T3$, may also yield, it is not likely to dominate the capacity of the resisting system, since it may be assisted by a membrane mechanism near the footing face.

Assuming the inclination angles of $C1$, $C2$, and $C3$, are 45°, then the resistance can be expressed as

$$R_{ju} = (C1 + C2 + C3)\sin 45° = T1 + 2T2 \tag{38.41}$$

and at steel yielding,

$$R_{ju} = \frac{\pi}{2} n A_{sp} f_y + 2A_{\text{jeff}} \rho_{vs} f_y \tag{38.42}$$

where n is the number of layers of the column transverse reinforcement inside the footing; A_{sp} is the cross-sectional area of a hoop or spiral bar; A_{jeff} is the effective area in which the vertical stirrups are effective to resist the resultant tensile force, T_c; ρ_{vs} is the area ratio of the footing vertical stirrups.

The effective area, A_{jeff}, can be defined based on a three-dimensional crack with 45° slope around the column longitudinal bars in tension. The projection of the crack to the footing surface is shown by the shaded area in Figure 38.11b. The depth of the crack or the distance of the boundary of the shaded area in Figure 38.11b to the nearest longitudinal bar yielded in tension is assumed to be equal to the anchorage depth, d_{af}. The depth of the crack reduces from d_{af} to zero linearly if the strain of the rebar reduces from the yield strain, ε_y, to zero. Thus, A_{jeff} can be calculated as follows,

$$A_{\text{jeff}} = d_{af}(d_{af} + r_c)\arccos\left[\left(1 + \frac{\varepsilon_y}{\varepsilon_c}\right)\frac{x_n}{r_c} - 1\right] + d_{af} r_c \arccos\left(\frac{x_n}{r_c} - 1\right) \tag{38.43}$$

where d_{af} is the depth of the column longitudinal reinforcement inside footing; r_c is the radius of the centroidal circle of the longitudinal reinforcement and can be simply taken as the radius of the column section if the cover concrete is ignored; ε_y is the yield strain of the longitudinal bars; ε_c is

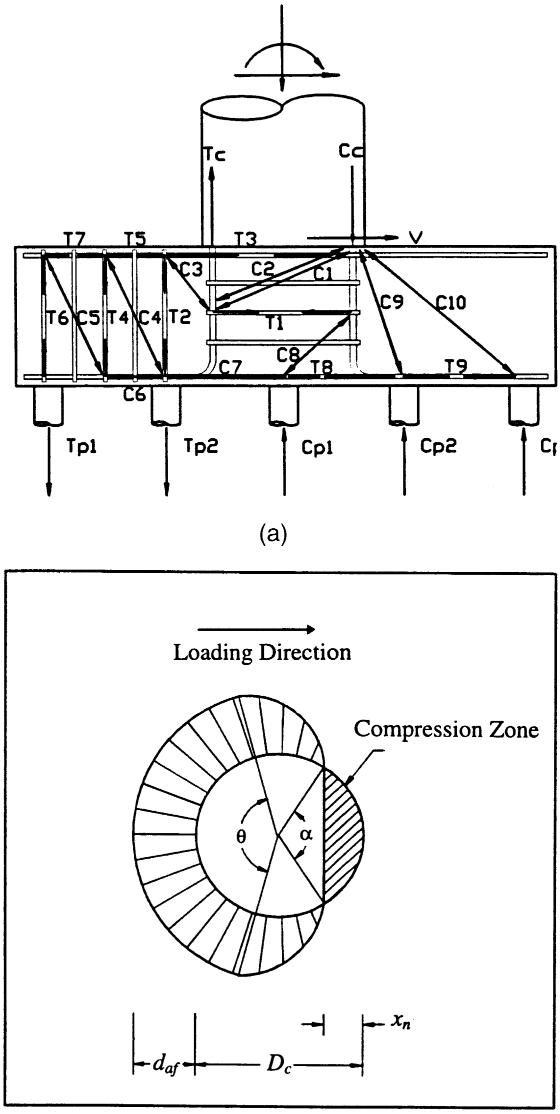


FIGURE 38.11 Column footing joint shear design: (a) force resisting mechanisms in footing; (b) effective distribution of external stirrups for joint shear resistance. (*Source*: Xiao, Y. et al. 1998. With permission.)

strain of the extreme compressive reinforcement or simply taken as the extreme concrete ultimate strain if the cover is ignored; x_n is the distance from the extreme compressive reinforcement to the neutral axis or taken as the compression zone depth, ignoring the cover.

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