

Table of Revisions from SDC 1.5 to SDC 1.6

Section	Revision
Table of Contents	New Subsections added: 7.4.5.1 and 7.8.4.1 Caption change to Subsections 3.2.1, 3.6.5.3, 3.6.7, 6.1, 7.4.4, 7.4.4.3, and 8.2.1 Repagination
1., 3.2.1, 3.6.7 3.7.3, 3.8.4, 7.8.3, 8.2, 8.2.1	References to Bridge Design Specifications changed to AASHTO LRFD Bridge Design Specifications with Interims and CA Amendments
1.	Performance goals for Ordinary bridges spelled out
2.1.3	Single span bridges supported on seat type abutments specifically excluded from vertical acceleration requirement
2.1.5	Language changed to be consistent with newly developed Design Spectrum - Appendix B Section rearranged and Equations renumbered
2.2.4	Correction to Figure 2.2 Change in terminology in Figure 2.4: ‘Pile Shaft’ changed to ‘Shaft’
3.1.1	MCE changed to Design Seismic Hazards
3.1.4.1	Minor revision
3.2.1	Caption modified Information on resistance factor, ϕ for shear and bending added
3.6.1, 3.6.6.2	Deleted: $\phi = 0.85$
3.4	Resistance factor, ϕ for bending added
3.6.2	Limits specified for the value of “ $\rho_s f_{yh}$ ” in Equation 3.20 Figure 3.8 modified Equation 3.20 was rearranged

3.6.5.3	Caption modified Correction to rebar size and spacing of bars in interlocking portion New Figure added – Figure 3.9
3.6.7	Caption modified New information added
4.1.1	Definitions of Δ_D and Δ_C streamlined
4.2	Minor correction to Figure 4.2 Minor correction to Footnote No. 5
5.3	Reference to Sections on abutment modeling guidelines added
6.1	Caption modified New information on Design Seismic Hazards added
6.2.1	MCE changed to Design Seismic Hazards
6.2.2 (A)	MCE level forces changed to ground shaking forces
7.1.1	Correction to Equations (7.1b) and (7.2b) Equations 7.1a, 7.1b, 7.2a, and 7.2b presented in tabular format
7.2.2	Minor revision
7.4.3	Expanded definition of Knee Joint added Footnote No. 8 changed
7.4.4	Caption changed to “Joint Shear Design”
7.4.4.3	Caption changed to “T Joint Shear Reinforcement” Figures 7.8 and 7.10 updated
7.4.5	Major Revision: Detailed provisions for Knee Joint Design
7.4.5.1	New subsection: “Knee Joint Shear Reinforcement” New Figures added: Figures 7.10b – 7.10g-2
7.6.5.1	Gap between column flare and bent cap soffit changed to 4 inches
7.7.1.1	Major revision to Equations 7.30 and 7.31, and Figures 7.11 and 7.12 New Equations added New information and definitions added Information deleted

7.7.1.2.1	New information added to “Lateral Design”
7.7.1.4	Minor revision
7.7.1.7	New Figures added: Figures 7.13c and 7.13d Body of Section revised and new information added
7.8.1	Major revision – Equation 7.43 New information added
7.8.2	Major revision
7.8.4	Major revision with new equations
7.8.4.1	New subsection: “Abutment Shear Key Reinforcement” New Figures: Figures 7.16(A) and 7.16(B)
8.2.1	Revision to Caption and Body of Section
8.2.4	Minor revision Slight modification to Caption
Bibliography	New References added References deleted

1. INTRODUCTION

The Caltrans Seismic Design Criteria (SDC) specify the minimum seismic design requirements that are necessary to meet the performance goals established for **Ordinary bridges**, in **Memo To Designer (MTD) 20-1**. When the **Design Seismic Hazards (DSH)** occur, Ordinary bridges designed per these specifications are expected to remain standing but may suffer significant damage requiring closure. See Sections 1.1 and 6.1, respectively, for definitions of **Ordinary bridges** and **Design Seismic Hazards**.

The SDC is a compilation of new seismic design criteria and existing seismic design criteria previously documented in various locations-publications. The goal of this document is to update all the Office of Structures Structure Design (OSD) (SD) design manuals¹ on a periodic basis to reflect the current state of practice for seismic bridge design. As information is incorporated into the design manuals, the SDC will serve as a forum to document Caltrans' latest changes to the seismic design methodology. Proposed revisions to the SDC will be reviewed by OSD SD management according to the process outlined in MTD 20-11.

The SDC applies to Ordinary Standard bridges as defined in Section 1.1. Ordinary Nonstandard bridges require project specific criteria to address their non-standard features. Designers should refer to the OSD SD design manuals for seismic design criteria not explicitly addressed by the SDC.

The following criteria identify the minimum requirements for seismic design. Each bridge presents a unique set of design challenges. The designer must determine the appropriate methods and level of refinement necessary to design and analyze each bridge on a case-by-case basis. The designer must exercise judgment in the application of these criteria. Situations may arise that warrant detailed attention beyond what is provided in the SDC. The designer should refer to other resources to establish the correct course of action. The OSD SD Senior Seismic Specialists, the OSD General Earthquake Committee, and the Earthquake Engineering Office of Structure Design Services and Earthquake Engineering (SDEE) Structure Policy and Innovation, or Caltrans Structures Design Oversight Representative should be consulted for recommendations.

Deviations to these criteria shall be reviewed and approved by the Section Design Senior Design Branch Chief or the Senior Seismic Specialist and documented in the project file. Significant departures shall be presented to the Type Selection Panel and/or the Design Branch Chief for approval as outlined in MTD 20-11.

This document is intended for use on bridges designed by and for Caltrans. It reflects the current state of practice at Caltrans. This document contains references specific and unique to Caltrans and may not be applicable to other parties either institutional or private.

¹ Caltrans Design Manuals: AASHTO LRFD Bridge Design Specifications and CA Amendments, Memo To Designers, Bridge Design Details, Bridge Design Aids, Bridge Design Practice. Throughout this document, the term "LRFD BDS" shall be used to represent AASHTO LRFD Bridge Design Specifications with Interims and CA Amendments [12,14].

2.1.3 Vertical Ground Motions

For Ordinary Standard bridges where the site peak rock acceleration is 0.6g or greater, an equivalent static vertical load shall be applied to the superstructure to estimate the effects of vertical acceleration². The superstructure shall be designed to resist the applied vertical force as specified in Section 7.2.2. **Note that this requirement does not apply to single span Ordinary Standard bridges supported on seat type abutments.** A case-by-case determination on the effect of vertical load is required for Non-standard and Important bridges.

²This is an interim method of approximating the effects of vertical acceleration on superstructure capacity. The intent is to ensure all superstructure types, especially lightly reinforced sections such as P/S box girders, have a nominal amount of mild reinforcement available to resist the combined effects of dead load, earthquake, and prestressing in the upward or downward direction. This is a subject of continued study.

2.1.5 Damping

A 5% damped elastic **ARS curve response spectrum** shall be used for determining **the accelerations for seismic demand in** Ordinary Standard concrete bridges. Damping ratios on the order of 10% can be justified for bridges that are heavily influenced by energy dissipation at the abutments and are expected to respond like single-degree-of-freedom systems. A reduction factor, R_D can be applied to the 5% damped **ARS coefficient response spectrum** used to calculate the displacement demand.

$$R_D = \frac{1.5}{[40c + 1]} + 0.5 \quad (2.1a)$$

$$ARS' = (R_D)(ARS) \quad Sd' = (R_D) \times (Sd) \quad (2.1b)$$

where: c = damping ratio ($0.05 \leq c \leq 0.1$)

ARS Sd = 5% damped **ARS curve spectral displacement**

ARS' = modified ARS curve Sd' = spectral displacement modified for higher levels of damping

The following characteristics are typically good indicators that higher damping may be anticipated [3].

- Total length less than 300 feet (90 m)
- Three spans or less
- Abutments designed for sustained soil mobilization
- Normal or slight skew (less than 20 degrees)
- Continuous superstructure without hinges or expansion joints

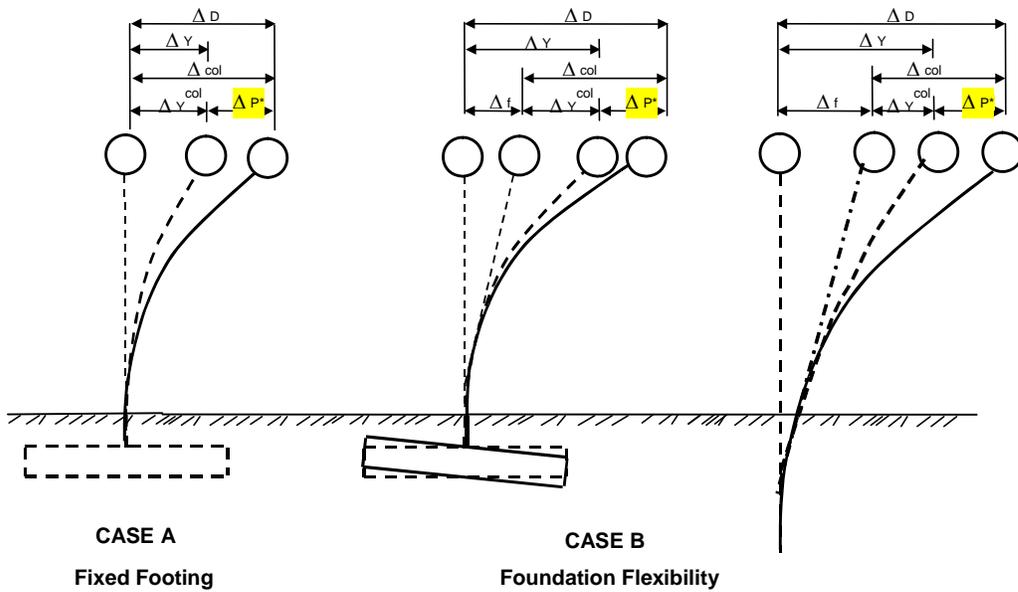
However, abutments that are designed to fuse (seat type abutment with backwalls), or respond in a flexible manner, may not develop enough sustained soil-structure interaction to rely on the higher damping ratio.

2.2.4 Target Displacement Ductility Demand

The target displacement ductility demand values for various components are identified below. These target values have been calibrated to laboratory test results of ~~fix-based~~ fixed base cantilever columns where the global displacement equals the column's displacement. The designer should recognize that as the framing system becomes more complex and boundary conditions are included in the demand model, a greater an increased percentage of the global displacement will be attributed to the flexibility of components other than the ductile members within the frame. These effects are further magnified when elastic displacements are used in the ductility definition specified in equation 2.2 and shown in Figure 2.3. For such systems, including but not limited to, Type I or Type II shafts (see Figure 2.4 for definition of shaft), the global ductility demand values listed below may not be achieved. The target values may range between 1.5 and 3.5 where specific values cannot be defined.

Single Column Bents supported on fixed foundation	$\mu_D \leq 4$
Multi-Column Bents supported on fixed or pinned footings	$\mu_D \leq 5$
Pier Walls (weak direction) supported on fixed or pinned footings	$\mu_D \leq 5$
Pier Walls (strong direction) supported on fixed or pinned footings	$\mu_D \leq 1$

Minimum ductility values are not prescribed. The intent is to utilize the advantages of flexible systems, specifically to reduce the required strength of ductile members and minimize the demand imparted to adjacent capacity protected components. Columns or piers with flexible foundations will naturally have low displacement ductility demands because of the foundation's contribution to Δ_y . The minimum lateral strength requirement in Section 3.5 or the P- Δ requirements in Section 4.2 may govern the design of frames where foundation flexibility lengthens the period of the structure into the range where the ARS demand is typically reduced.



Note: For a cantilever column with fixed base $\Delta_Y^{col} = \Delta_Y$
 ΔP^* = portion of the plastic displacement capacity Δ_p

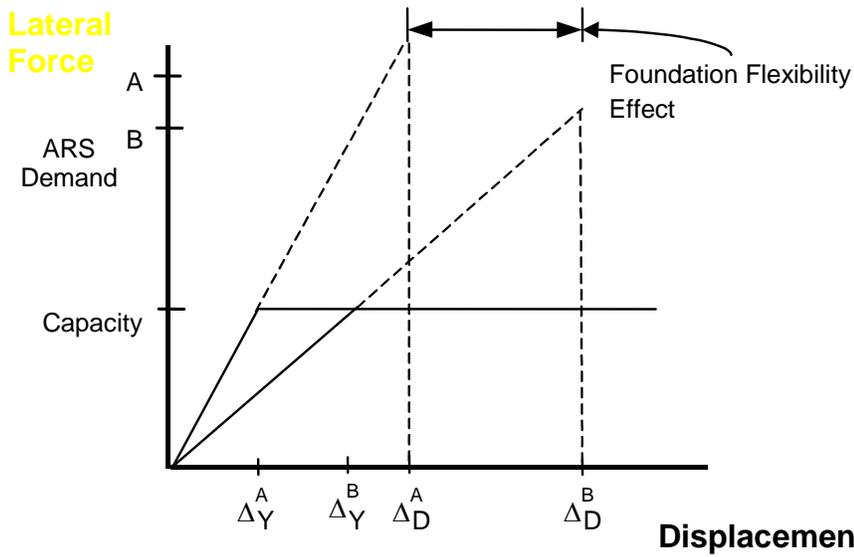


Figure 2.2 The Effects of Foundation Flexibility on the Force-Deflection Curve of a Single Column Bent

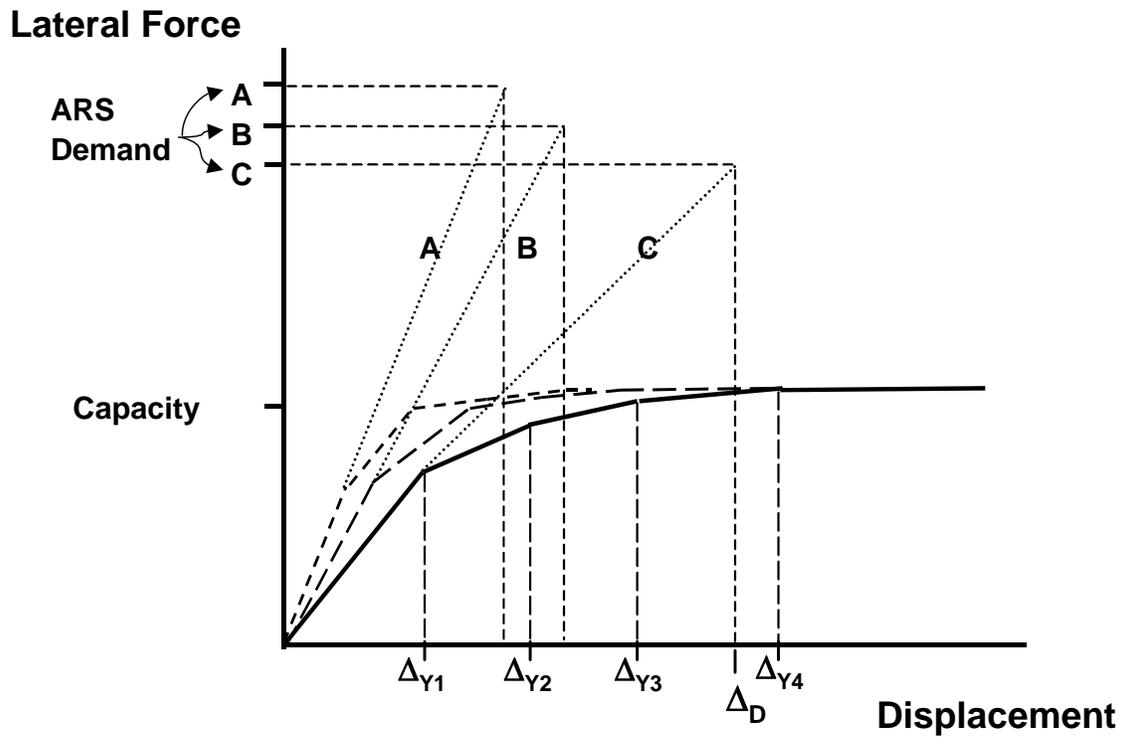
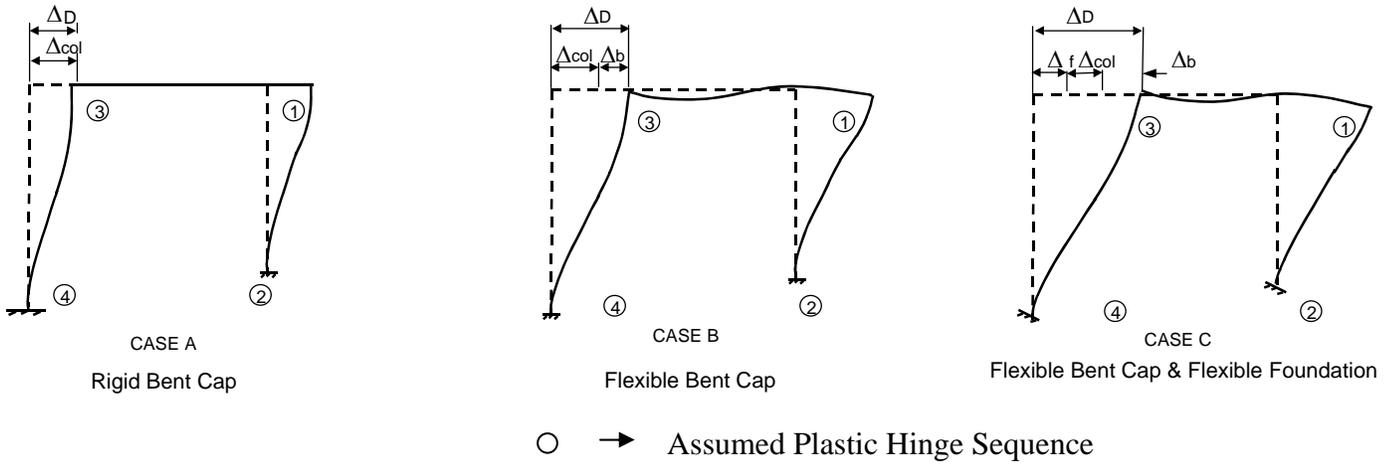
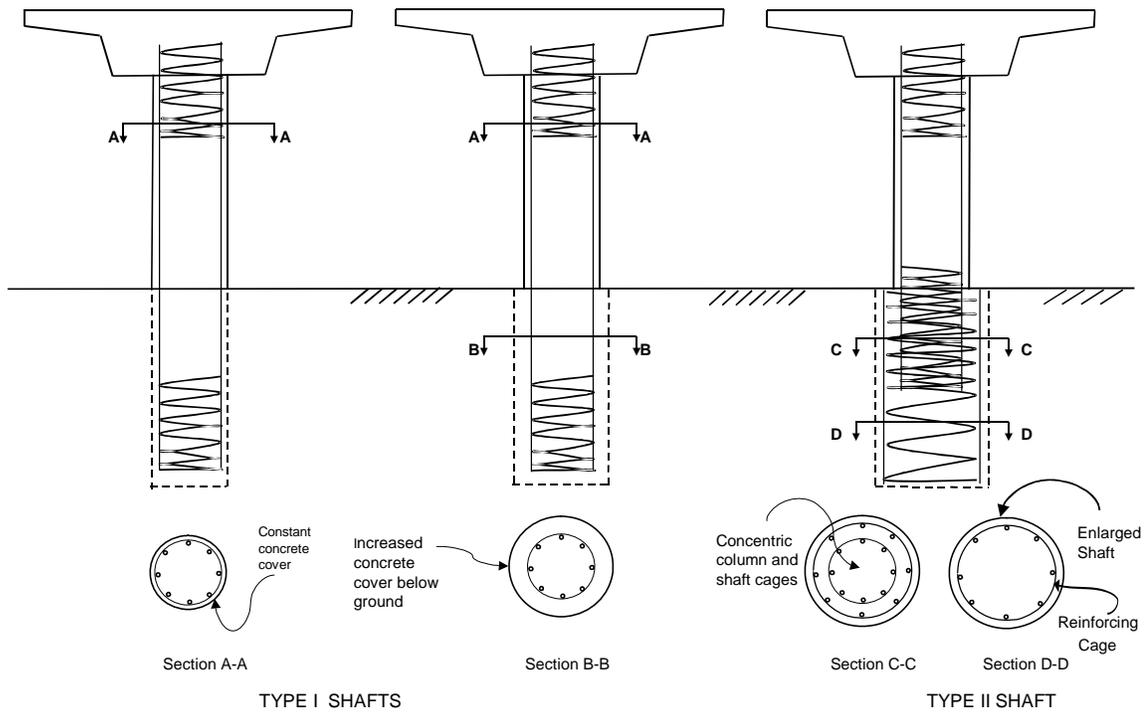


Figure 2.3 The Effects of Bent Cap and Foundation Flexibility on Force-Deflection Curve of a Bent Frame



Type I pile shafts are designed so the plastic hinge will form below ground in the pile shaft. The concrete cover and area of transverse and longitudinal reinforcement may change between the column and Type I pile shaft, but the cross section of the confined core is the same for both the column and the pile shaft. The global displacement ductility demand, μ_D for a Type I pile shaft shall be less than or equal to the μ_D for the column supported by the shaft as specified in Section 2.2.4, with an upperbound value corresponding to that of the supported column.

Type II Pile Shafts

Type II pile shafts are designed so the plastic hinge will form at or above the shaft/column interface, thereby, containing the majority of inelastic action to the ductile column element. Type II shafts are usually enlarged pile shafts characterized by a reinforcing cage in the shaft that has a core diameter larger than that of the column it supports. Type II pile shafts shall be designed to remain elastic, $\mu_D \leq 1$. See Section 7.7.3.2 for design requirements for Type II pile shafts.

Figure 2.4 Pile Shaft Definitions

NOTE:

Generally, the use of Type II Pile S shafts should be discussed and approved at the Type Selection Meeting. Type II Pile Shafts will increase the foundation costs, compared to Type I Pile Shafts, however there is an advantage of improved post-earthquake inspection and repair. Typically, Type I shafts is are appropriate for short columns, while Type II shafts is are used in conjunction with taller columns. The end result shall be a structure with an appropriate fundamental period a balanced stiffness as discussed elsewhere in Section 7.

3.1.1 Ductile Member Definition

A ductile member is defined as any member that is intentionally designed to deform inelastically for several cycles without significant degradation of strength or stiffness under the demands generated by the **MCE Design Seismic Hazards**. See Section 6.1 for the definition of **Design Seismic Hazards**.

3.1.4 Local Member Displacement Ductility Capacity

Local displacement ductility capacity for a particular member is defined in Equations 3.6.

$$\begin{aligned} \mu_c &= \frac{\Delta_c}{\Delta_Y^{col}} && \text{for Cantilever columns,} \\ \mu_{c1} &= \frac{\Delta_{c1}}{\Delta_{Y1}^{col}} \quad \& \quad \mu_{c2} = \frac{\Delta_{c2}}{\Delta_{Y2}^{col}} && \text{for fixed-fixed columns} \end{aligned} \quad (3.6)$$

3.1.4.1 Minimum Local Displacement Ductility Capacity

Each ductile member shall have a minimum local displacement ductility capacity of $\mu_c = 3$ (or, $\mu_{c1} \geq 3$ and $\mu_{c2} \geq 3$, see Equations 3.6) to ensure dependable rotational capacity in the plastic hinge regions regardless of the displacement demand imparted to that member. The local displacement ductility capacity shall be calculated for an equivalent member that approximates a fixed base cantilever element as defined in Figure 3.3.

The minimum displacement ductility capacity $\mu_c = 3$ may be difficult to achieve for columns and Type I pile shafts with large diameters $D_c > 10$ ft (3m) or components with large L/D ratios. Local displacement ductility capacity less than three (3) requires approval as specified in MTD 20-11.

3.2.1 Expected Versus Nominal Material Properties

The capacity of concrete components to resist all seismic demands, except shear, shall be based on most probable (expected) material properties to provide a more realistic estimate for design strength. An expected concrete compressive strength, f'_{ce} recognizes the typically conservative nature of concrete batch design, and the expected strength gain with age. The yield stress f_y for ASTM A706 steel can range between 60 ksi to and 78 ksi. An expected reinforcement yield stress, f_{ye} is a “characteristic” strength and better represents the actual strength than the specified minimum of 60 ksi. The possibility that the yield stress may be less than f_{ye} in ductile components will result in a reduced ratio of actual plastic moment strength to design strength, thus conservatively impacting capacity protected components. The possibility that the yield stress may be less than f_{ye} in essentially elastic components is accounted for in the overstrength magnifier specified in Section 4.3.1. Expected material properties shall only be used to assess capacity for earthquake loads. The material properties for all other load cases shall comply with the Caltrans Bridge Design Specifications (BDS).

Seismic shear capacity shall be conservatively based on the nominal material strengths (i.e., f_y , f'_c) defined in Section 3.6.1, not the expected material strengths.

For all seismic-related calculations involving capacity of ductile, non-ductile and capacity protected members, the resistance factor, ϕ shall be taken as 0.90 for shear and 1.0 for bending.

3.4 Requirements for Capacity Protected Components

Capacity protected concrete components such as footings, Type II pile shafts, bent cap beams, joints and superstructure shall be designed flexurally to remain essentially elastic when the column reaches its overstrength capacity. The expected nominal moment capacity M_{ne} for capacity protected concrete components determined by either $M-\phi$ or strength design, is the minimum requirement for essentially elastic behavior. Due to cost considerations a factor of safety is not required (i.e., resistance factor $\phi = 1.0$ for flexure). Expected material properties shall only be used to assess flexural component capacity for resisting earthquake loads. The material properties used for assessing all other load cases shall comply with the Caltrans design manuals.

Expected nominal moment capacity for capacity protected concrete components shall be based on the expected concrete and steel strengths when either the concrete strain reaches 0.003 or the reinforcing steel strain reaches ϵ_{su}^R as derived from the steel stress strain model.

3.6.1 Nominal Shear Capacity

The seismic shear demand shall be based on the overstrength shear V_o associated with the overstrength moment M_o defined in Section 4.3. The shear capacity for ductile concrete members shall be conservatively based on the nominal material strengths.

$$\phi V_n \geq V_o \quad \phi = 0.85 \quad (3.14)$$

$$V_n = V_c + V_s \quad (3.15)$$

Where, ϕ = Resistance factor as defined in Section 3.2.1.

3.6.2 Concrete Shear Capacity

The concrete shear capacity of members designed for ductility shall consider the effects of flexure and axial load as specified in Equation 3.16 through 3.21.

$$V_c = v_c \times A_e \quad (3.16)$$

$$A_e = 0.8 \times A_g \quad (3.17)$$

- Inside the plastic hinge zone

$$v_c = \begin{cases} \text{Factor 1} \times \text{Factor 2} \times \sqrt{f'_c} \leq 4\sqrt{f'_c} & \text{(psi)} \\ \text{Factor 1} \times \text{Factor 2} \times \sqrt{f'_c} \leq 0.33\sqrt{f'_c} & \text{(MPa)} \end{cases} \quad (3.18)$$

- Outside the plastic hinge zone

$$v_c = \begin{cases} 3 \times \text{Factor 2} \times \sqrt{f'_c} \leq 4\sqrt{f'_c} & \text{(psi)} \\ 0.25 \times \text{Factor 2} \times \sqrt{f'_c} \leq 0.33\sqrt{f'_c} & \text{(MPa)} \end{cases} \quad (3.19)$$

where:

$$0.3 \leq \text{Factor 1} = \frac{\rho_s f_{yh}}{0.150 \text{ ksi}} + 3.67 - \mu_d \leq 3 \quad (f_{yh} \text{ in ksi units}) \quad (3.20)$$

$$0.025 \leq \text{Factor 1} = \frac{\rho_s f_{yh}}{12.5} + 0.305 - 0.083\mu_d \leq 0.25 \quad (f_{yh} \text{ in MPa units})$$

In Equation (3.20), f_{yh} is in ksi (MPa), the value of “ $\rho_s f_{yh}$ ” shall be limited to 0.35 ksi. Figure 3.8 shows how the value of Factor 1 varies over a range of ductility demand ratios, μ_d .

$$\text{Factor 2} = \begin{cases} 1 + \frac{P_c}{2000 \times A_g} < 1.5 & \text{(English Units)} \\ 1 + \frac{P_c}{13.8 \times A_g} < 1.5 & \text{(Metric Units)} \end{cases} \quad (3.21)$$

In Equation (3.21), P_c is in lb (N), and A_g is in in² (mm²).

For members whose net axial load is in tension, $v_c = 0$.



Figure 3.8 Concrete Shear Factors

The global displacement ductility demand μ_D shall may be used in the determination of Factor 1 provided a significant portion of the global displacement is attributed to the deformation of the column or pier. In all other cases a local displacement ductility demand μ_d shall be used in Factor 1 of the shear equation.

3.6.5.3 Minimum Vertical Reinforcement ~~in~~ within Interlocking ~~Portion~~ Hoops

The longitudinal rebars in the interlocking portion of the column ~~shall~~ **should** have a maximum spacing of 8 inches and need not be anchored in the footing or the bent cap unless deemed necessary for the flexural capacity of the column. The longitudinal rebar size in the interlocking portion of the column (“B” bars in Figure 3.9) shall be chosen **correspondingly to correspond** to the rebars outside the interlocking portion as follows:

Size of rebars used outside the interlocking portion (A)	Minimum size of rebars required inside the interlocking portion (B)
#10	#6
#11	#8
#14	#9
#18	#11

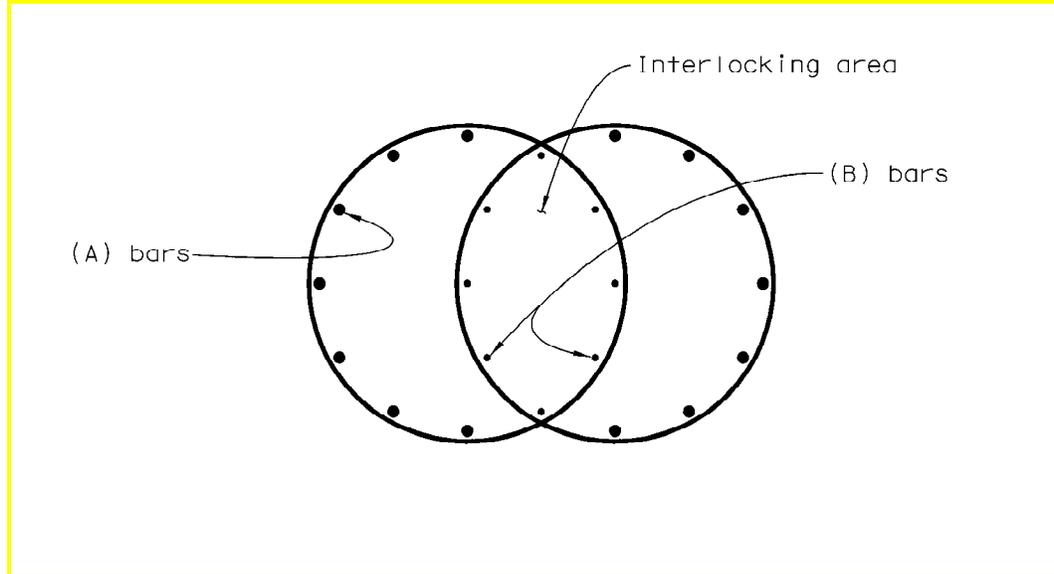


Figure 3.9 Vertical Reinforcement within Interlocking Hoops

3.6.6.2 Shear Capacity in the Strong Direction

The shear capacity of pier walls in the strong direction shall resist the maximum shear demand specified in Section 2.3.2.2.

$$\phi V_n^{pw} > V_u^{pw} \quad (3.26)$$

$$\phi = 0.85$$

Studies of squat shear walls have demonstrated that the large shear stresses associated with the moment capacity of the wall may lead to a sliding failure brought about by crushing of the concrete at the base of the wall. The thickness of pier walls shall be selected so the shear stress satisfies equation 3.27 [6].

$$\frac{V_n^{pw}}{0.8 \times A_g} < 8 \times \sqrt{f'_c} \quad (\text{psi}) \quad \frac{V_n^{pw}}{0.8 \times A_g} < 0.67 \times \sqrt{f'_c} \quad (\text{MPa}) \quad (3.27)$$

3.6.7 **Shear** Capacity of Capacity Protected Members

The shear capacity of ~~essentially elastic~~ capacity protected members shall be ~~designed~~ calculated in accordance with ~~BDS Section 8.16.6~~ LFRD BDS using nominal material properties, with the shear resistance factor ϕ taken as 0.90. The expected nominal moment capacity, M_{ne} for capacity protected members shall be determined as specified in Section 3.4 using expected values of material properties. Moment and shear demands on these structural elements are determined corresponding to the overstrength capacities of the connected ductile components.

3.7.3 Maximum Reinforcement Ratio

The designer must ensure that members sized to remain essentially elastic (i.e. superstructure, bent caps, footings, Type II enlarged-pile shafts) retain a ductile failure mode. The reinforcement ratio, ρ shall meet the requirements in BDS Section 8.16.3 for reinforced concrete members and BDS Section 9.19 for prestressed concrete members LRFD BDS.

3.8.4 Lateral Reinforcement of Pier Walls

The lateral confinement of pier walls shall be ~~provided by~~ comprised of cross ties. The total cross sectional tie area, A_{yh} required inside the plastic end regions of pier walls shall be the larger of the volume of steel required in Section 3.8.2 or BDS Sections 8.18.2.3.2 through 8.18.2.3.4. The lateral pier wall reinforcement outside the plastic hinge region shall satisfy BDS Section 8.18.2.3. in LRFD BDS.

4.1.1 Global Displacement Criteria

Each bridge or frame shall satisfy Equation 4.1. Where Δ_D is the displacement along the local principal axes of a ductile member generated by seismic deformations applied to the structural system as defined in Section 2.1.2.⁴

$$\Delta_D < \Delta_C \quad (4.1)$$

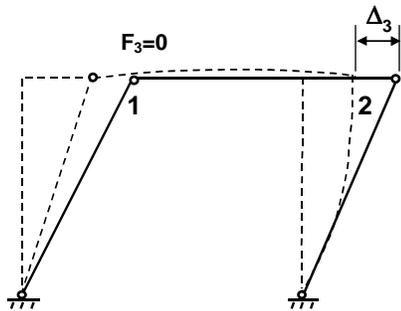
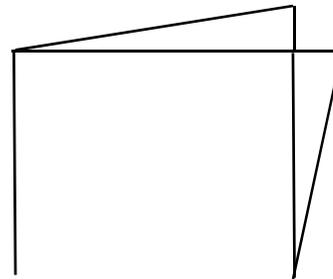
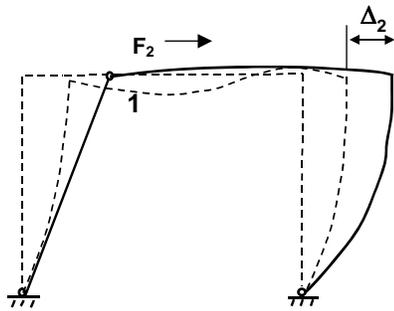
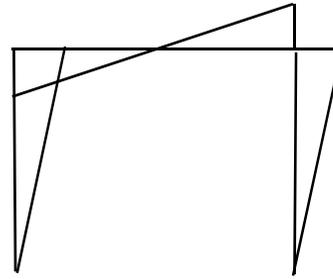
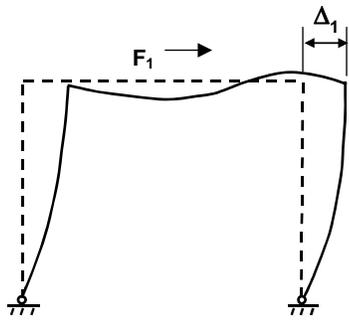
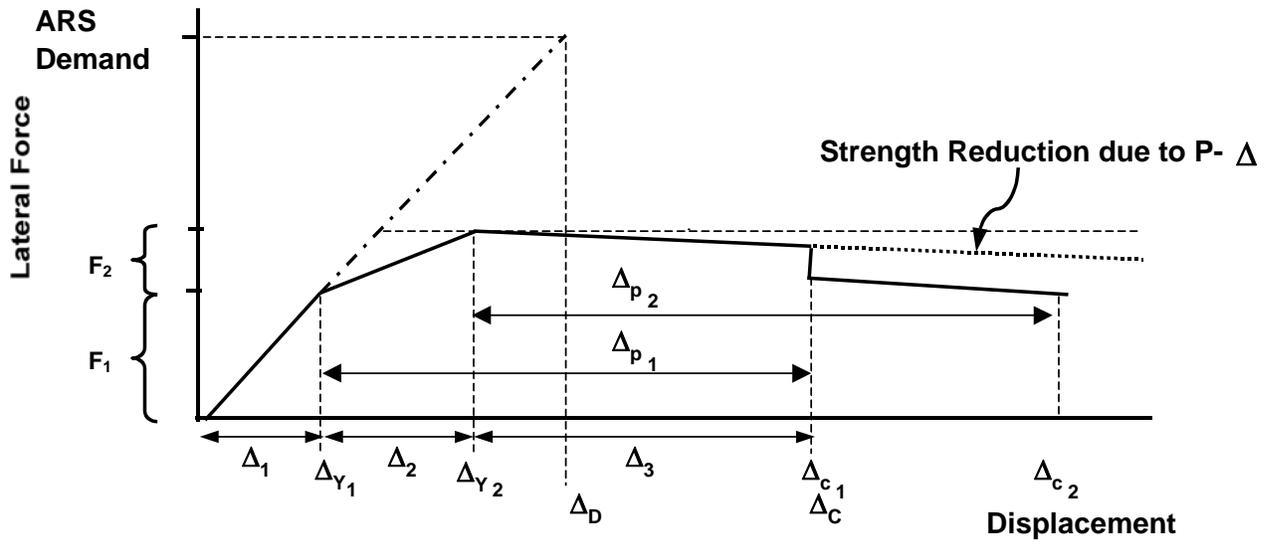
where:

Δ_C is the bridge or frame displacement capacity when any plastic hinge reaches its first ultimate capacity is reached by any plastic hinge. See Figure 4.1 [4, 7].

Δ_D is the displacement generated from the global analysis, the stand-alone analysis, or the larger of the two if both types of analyses are necessary is the displacement demand along the local principal axes of a ductile member generated by seismic deformations applied to the structural system as defined in Section 2.1.2.⁴ Δ_D is obtained by performing analyses as defined in Section 5.2.

In applying Equation 4.1, care must be taken to ensure that Δ_D is compared to Δ_C corresponding to the same local principal axis as Δ_D .

⁴ The SDC Development Team elected not to include an interaction relationship for the displacement demand/capacity ratios along the principal axes of ductile members. This decision was based on the inherent factor of safety provided elsewhere in our practice. This factor of safety is provided primarily by the limits placed on permissible column displacement ductility and ultimate material strains, as well as the reserve capacity observed in many of the Caltrans sponsored column tests. Currently test data is not available to conclusively assess the impact of bi-axial displacement demands and their effects on member capacity especially for columns with large cross-sectional aspect ratios.



Idealized Frame

$$\text{Force Capacity} = \sum F_{(i)} = F_1 + F_2$$

$$\text{Displacement Capacity} = \sum \Delta_{(i)} = \Delta_1 + \Delta_2 + \Delta_3$$

Figure 4.1 Global Force Deflection Relationship [4],[7]

4.2 P-Δ Effects

The dynamic effects of gravity loads acting through lateral displacements shall be included in the design. The magnitude of displacements associated with $P-\Delta$ effects can only be accurately captured with non-linear time history analysis. In lieu of such analysis, equation 4.3 can be used to establish a conservative limit for lateral displacements induced by axial load for columns meeting the ductility demand limits specified in Section 2.2.4. If equation 4.3 is satisfied, $P-\Delta$ effects can typically be ignored.⁵ See Figure 4.2. [4]

$$P_{dl} \times \Delta_r \leq 0.20 \times M_p^{col} \quad (4.3)$$

Where: Δ_r = The relative lateral offset between the point of contra-flexure and the base of the plastic hinge. For Type I pile shafts $\Delta_r = \Delta_D - \Delta_s$
 Δ_s = The pile shaft displacement at the point of maximum moment

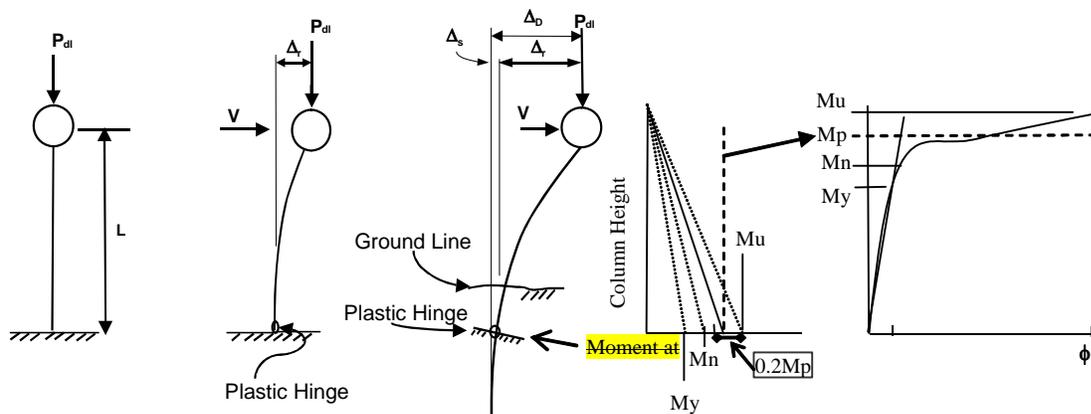


Figure 4.2 P-Δ Effects on Bridge Columns [4]

⁵ The moment demand at the point of maximum moment in the shaft is shown in Figure 4.2. As the displacement of the top of column is increased, moment demand values at the base pass through M_y , M_n , M_p , and M_u (key values defining the moment-curvature curve, see Figure 4.2). The idealized plastic moment M_p is always less than M_u in a well-confined column. Therefore, the $0.2M_p$ allowance for the $P-\Delta$ effects is justifiable, given the reserve moment capacities shown above.

5.3 Structural System “Global” Analysis

Structural system or global analysis is required when it is necessary to capture the response of the entire bridge system. Bridge systems with irregular geometry, in particular such as curved bridges and skew bridges, bridges with multiple transverse expansion joints, massive substructures components, and foundations supported by soft soil can exhibit dynamic response characteristics that are not necessarily obvious and may not be captured in a separate subsystem analysis [7].

Two global dynamic analyses are normally required to capture the assumed nonlinear response of a bridge because it possesses different characteristics in tension versus compression [3].

In the tension model, the superstructure joints including the abutments are released longitudinally with truss elements connecting the joints to capture the effects of the restrainers. In the compression model, all of the truss (restrainer) elements are inactivated and the superstructure elements are locked longitudinally to capture structural response modes where the joints close up, mobilizing the abutments when applicable and the abutments are mobilized. Abutment modeling guidance is given in Sections 7.8.1 and 7.8.2.

The structure’s geometry will dictate if both a tension model and a compression model are required. Structures with appreciable superstructure curvature may require additional models, which combine the characteristics identified for the tension and compression models.

Long multi-frame bridges shall be analyzed with multiple elastic models. A single multi-frame model may not be realistic since it cannot account for out-of-phase movement among the frames and may not have enough nodes to capture all of the significant dynamic modes.

Each multi-frame model should be limited to five frames plus a boundary frame or abutment on each end of the model. Adjacent models shall overlap each other by at least one useable frame, see Figure 5.1.

The boundary frames provide some continuity between adjacent models but are considered redundant and their analytical results are ignored. A massless spring should be attached to the dead unconnected end of the boundary frames to represent the stiffness of the remaining structure. Engineering judgement judgment should be exercised when interpreting the deformation results among various sets of frames since the boundary frame method does not fully account for the continuity of the structure [3].

6.1 Site ~~Assessment~~ Seismicity

The **Design Seismic Hazards (DSH)** include ground shaking (defined as ground motion time histories or response spectrum), liquefaction, lateral spreading, surface fault rupture, and tsunami. The response spectrum used in the design is called **Design Spectrum** as defined in Section 2.1 and Appendix B.

6.2.1. Foundation Performance

- Bridge foundations shall be designed to respond to seismic loading in accordance with the seismic performance objectives outlined in MTD 20-1
- The capacity of the foundations and their individual components to resist **MCE seismic demands** **the Design Seismic Hazards** shall be based on ultimate structural and soil capacities

6.2.2(A) Competent Soil

Foundations surrounded by competent soil are capable of resisting MCE-level ground shaking forces while experiencing small deformations. This type of performance characterizes a stiff foundation subsystem that usually has an insignificant impact on the overall dynamic response of the bridge and is typically ignored in the demand and capacity assessment. Foundations in competent soil can be analyzed and designed using a simple model that is based on assumptions consistent with observed response of similar foundations during past earthquakes. Good indicators that a soil is capable of producing competent foundation performance include the following:

- Standard penetration, upper layer (0-10 ft, 0-3 m) $N = 20$ (Granular soils)
- Standard penetration, lower layer (10-30 ft, 3-9 m) $N = 30$ (Granular soils)
- Undrained shear strength, $s_u > 1500$ psf (72 KPa) (Cohesive soils)
- Shear wave velocity, $v_s > 600$ ft/sec (180 m/sec)
- Low potential for liquefaction, lateral spreading, or scour

N = The uncorrected blow count from the Standard Test Method for Penetration Test and Split-Barrel Sampling of Soil.

7.1.1 Balanced Stiffness

It is strongly recommended that the ratio of effective stiffness between any two bents within a frame or between any two columns within a bent satisfy Equation 7.1. It is strongly recommended that the ratio of effective stiffness between adjacent bents within a frame or between adjacent columns within a bent satisfy Equation 7.2. An increase in superstructure mass along the length of the frame should be accompanied by a reasonable increase in column stiffness. For variable width frames the tributary mass supported by each bent or column shall be included in the stiffness comparisons as specified by Equations 7.1(b) and 7.2(b). The simplified analytical technique for calculating frame capacity described in Section 5.5 is only permitted if either Equations 7.1(a) & 7.2(a) or Equations 7.1(b) & 7.2(b) are satisfied.

Constant Width Frames	Variable Width Frames
$\frac{k_i^e}{k_j^e} \geq 0.5 \quad (7.1a)$	$2 \geq \frac{\frac{k_i^e}{m_i}}{\frac{k_j^e}{m_j}} \geq 0.5 \quad (7.1b)$
$\frac{k_i^e}{k_j^e} \geq 0.75 \quad (7.2a)$	$1.33 \geq \frac{\frac{k_i^e}{m_i}}{\frac{k_j^e}{m_j}} \geq 0.75 \quad (7.2b)$

	Constant Width Frames	Variable Width Frames
For any 2 Bents in a frame or any 2 Columns in a Bent	$\frac{k_i^e}{k_j^e} \geq 0.5 \quad (7.1a)$	$2 \geq \frac{\frac{k_i^e}{m_i}}{\frac{k_j^e}{m_j}} \geq 0.5 \quad (7.1b)$
For adjacent bents in a frame or adjacent Columns in a Bent	$\frac{k_i^e}{k_j^e} \geq 0.75 \quad (7.2a)$	$1.33 \geq \frac{\frac{k_i^e}{m_i}}{\frac{k_j^e}{m_j}} \geq 0.75 \quad (7.2b)$

k_i^e = The smaller effective bent or column stiffness

m_i = Tributary mass of column or bent i

k_j^e = The larger effective bent or column stiffness

m_j = Tributary mass of column or bent j

The following considerations shall be taken into account when calculating effective stiffness: framing effects, end conditions, column height, percentage of longitudinal and transverse column steel, column diameter, and foundation flexibility. Some of the consequences of not meeting the relative stiffness recommendations defined by Equations 7.1 and 7.2 include:

- Increased damage in the stiffer elements
- An unbalanced distribution of inelastic response throughout the structure
- Increased column torsion generated by rigid body rotation of the superstructure

7.2.2 Vertical Acceleration

If vertical acceleration is considered, per Section ~~2.1~~ 2.1.3, a separate analysis of the superstructure's nominal capacity shall be performed based on a uniformly applied vertical force equal to 25% of the dead load applied upward and downward, see Figure 7.3. The superstructure at seat type abutments is assumed to be pinned in the vertical direction, up or down. The superstructure flexural capacity shall be calculated, based only on continuous mild reinforcement distributed evenly between across the top and bottom slabs. The effects of dead load, primary prestressing and secondary prestressing shall be ignored. The continuous steel mild reinforcement shall be spliced with "service level" couplers as defined in Section 8.1.3, and is considered effective in offsetting the mild reinforcement required for other load cases. Lap splices equal to two times the standard lap may be substituted for the "service splices," provided the laps are placed away from the critical zones (mid-spans and near supports).

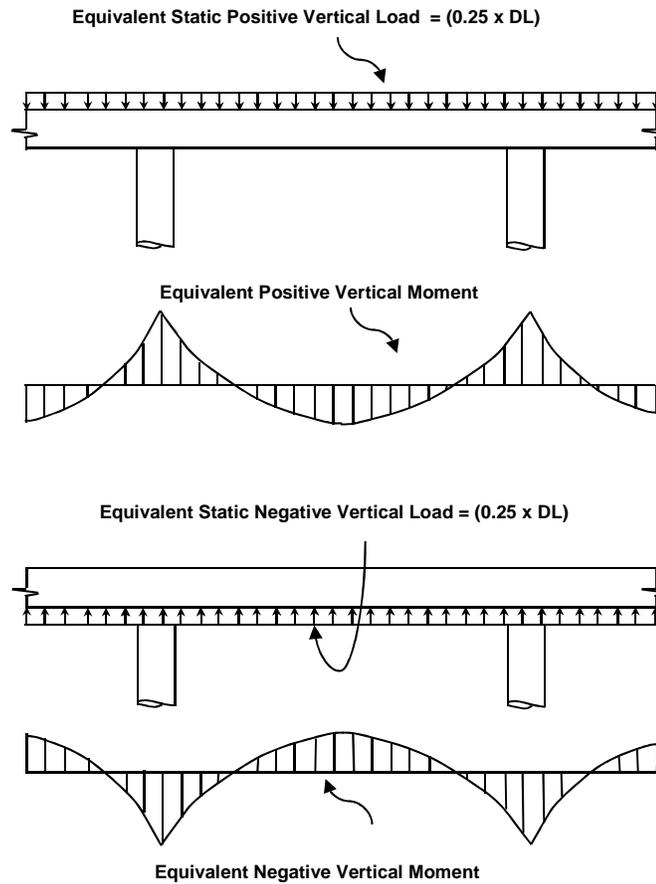


Figure 7.3 Equivalent Static Vertical Loads & Moments

The longitudinal side reinforcement in the girders, if vertical acceleration is considered per Section 2.1, shall be capable of resisting 125% of the dead load shear at the bent face by means of shear friction. ~~The This~~ enhanced longitudinal side reinforcement shall extend continuously for a minimum of $2.5D_s$ beyond the face of the bent cap.

7.4.3 Joint Description

The following types of joints are considered T joints for joint shear analysis:

- Integral interior joints of multi-column bents in the transverse direction
- All integral column/superstructure joints in the longitudinal direction
- Exterior column joints for box girder superstructures if the cap beam extends beyond the joint far enough to develop the longitudinal cap reinforcement.⁸

Any exterior column joint that satisfies Equation 7.10b shall be designed as a Knee joint.⁸

$$S < \max(D_c, l_d) \quad (7.10b)$$

where:

S = Cap beam short stub length, defined as the minimum distance from the exterior girder edge at soffit to the intersection of the bent centerline and face of the column (see Figure 7.10c),

D_c = Column dimension measured along the centerline of bent, and

l_d = Development length of the main bent cap reinforcement

⁸ It may be desirable to pin the top of the column to avoid knee joint requirements. This eliminates the joint shear transfer through the joint and limits the torsion demand transferred to the cap beam. However, the benefits of a pinned exterior joint should be weighed against increased foundation demands and the effect on the frame's overall performance.

7.4.4 **T** Joint Shear Design

(Body of this section is unchanged from Version 1.5)

7.4.4.3 **T** Joint Shear Reinforcement

A) **Vertical Stirrups:**

$$A_s^{jv} = 0.2 \times A_{st} \quad (7.19)$$

A_{st} = Total area of column reinforcement anchored in the joint

Vertical stirrups or ties shall be placed transversely within a distance D_c extending from either side of the column centerline. The vertical stirrup area, A_s^{jv} is required on each side of the column or pier wall, see Figures 7.7, 7.8, and 7.10. The stirrups provided in the overlapping areas shown in Figure 7.7 shall count towards meeting the requirements of both areas creating the overlap. These stirrups can be used to meet other requirements documented elsewhere including the shear in the bent cap.

B) **Horizontal Stirrups:**

Horizontal stirrups or ties shall be placed transversely around the vertical stirrups or ties in two or more intermediate layers spaced vertically at not more than 18 inches (450mm). This horizontal reinforcement A_s^{jh} shall be placed within a distance D_c extending from either side of the column centerline, see Figure 7.9.

$$A_s^{jh} = 0.1 \times A_{st} \quad (7.20)$$

C) **Horizontal Side Reinforcement:**

The total longitudinal side face reinforcement in the bent cap shall be at least equal to the greater of the areas specified in equation 7.21 and shall be placed near the side faces of the bent cap with a maximum spacing of 12 inches (300mm), see Figures 7.8 and 7.10. Any side reinforcement placed to meet other requirements shall count towards meeting the requirement in this section.

$$A_s^{sf} \geq \begin{cases} 0.1 \times A_{cap}^{top} \\ \text{or} \\ 0.1 \times A_{cap}^{bot} \end{cases} \quad A_{cap} = \text{Area of bent cap top or bottom flexural steel} \quad (7.21)$$

D) J-Dowels

For bents skewed greater than 20°, J-dowels hooked around the longitudinal top deck steel extending alternatively 24 inches (600 mm) and 30 inches (750 mm) into the bent cap are required. The J-dowel reinforcement shall be equal or greater than the area specified in Equation 7.22.

$$A_s^{\text{j-bar}} = 0.08 \times A_{st} \quad (7.22)$$

The J-dowels shall be placed within a rectangular region defined by the width of the bent cap and the distance D_c on either side of the centerline of the column, see Figure 7.10.

E) Transverse Reinforcement

Transverse reinforcement in the joint region shall consist of hoops with a minimum reinforcement ratio specified by Equation 7.23. The column confinement reinforcement extended into the bent cap may be used to meet this requirement.

$$\rho_s = 0.4 \times \frac{A_{st}}{l_{ac}^2} \quad (\text{in, mm}) \quad (7.23)$$

For interlocking cores ρ_s shall be based on area of reinforcement (A_{st}) of each core.

All vertical column bars shall be extended as close as possible to the top bent cap reinforcement.

F) Main Column Reinforcement

The main column reinforcement shall extend into the cap as deep as possible to fully develop the compression strut mechanism in the joint.

Bent Cap Details, Section at Column for Bridges with 0 to 20-Degree Skew.

(Detail Applies to Sections Within $2 \times$ Diameter of Column, Centered About CL of Column).

(Detail Applies to T-Beam and Box Girder Bridges Where Deck Reinforcement is Placed Parallel to Cap).

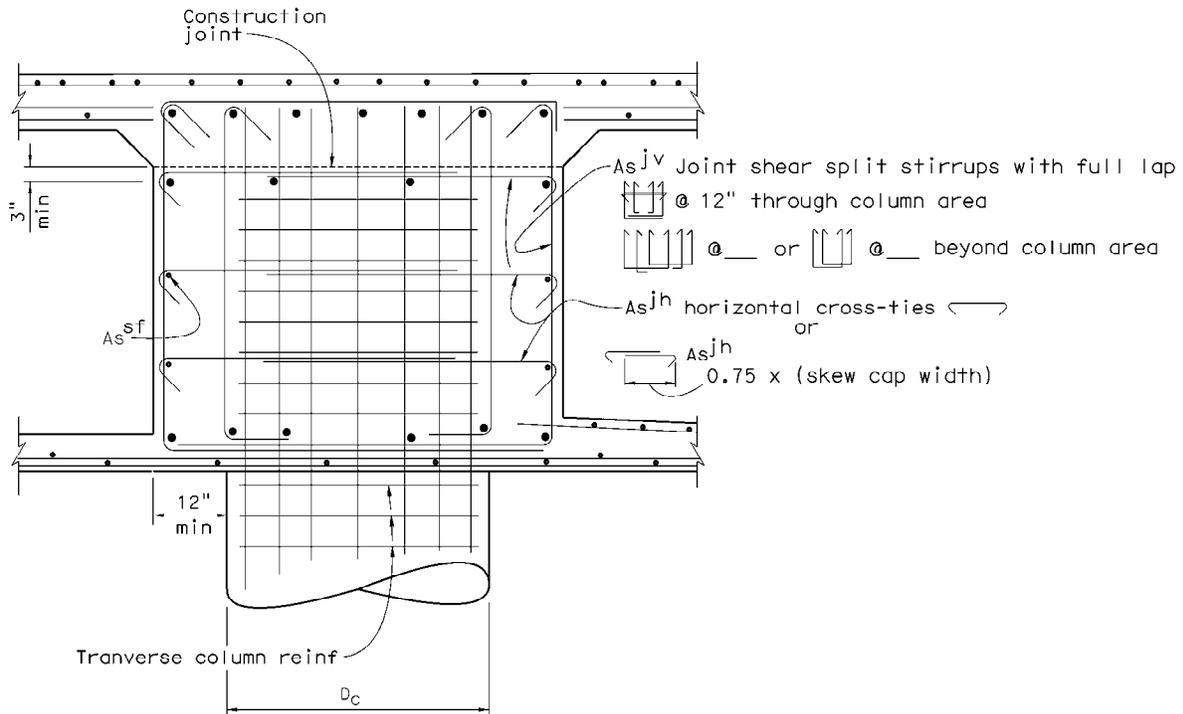


Figure 7.8 Joint Shear Reinforcement Details¹¹

¹¹ Figures 7.8, 7.9 and 7.10 illustrate the general location for joint shear reinforcement in the bent cap.

Bent Cap Elevation.
Horizontal Cross Tie and J-bar Placing Pattern.

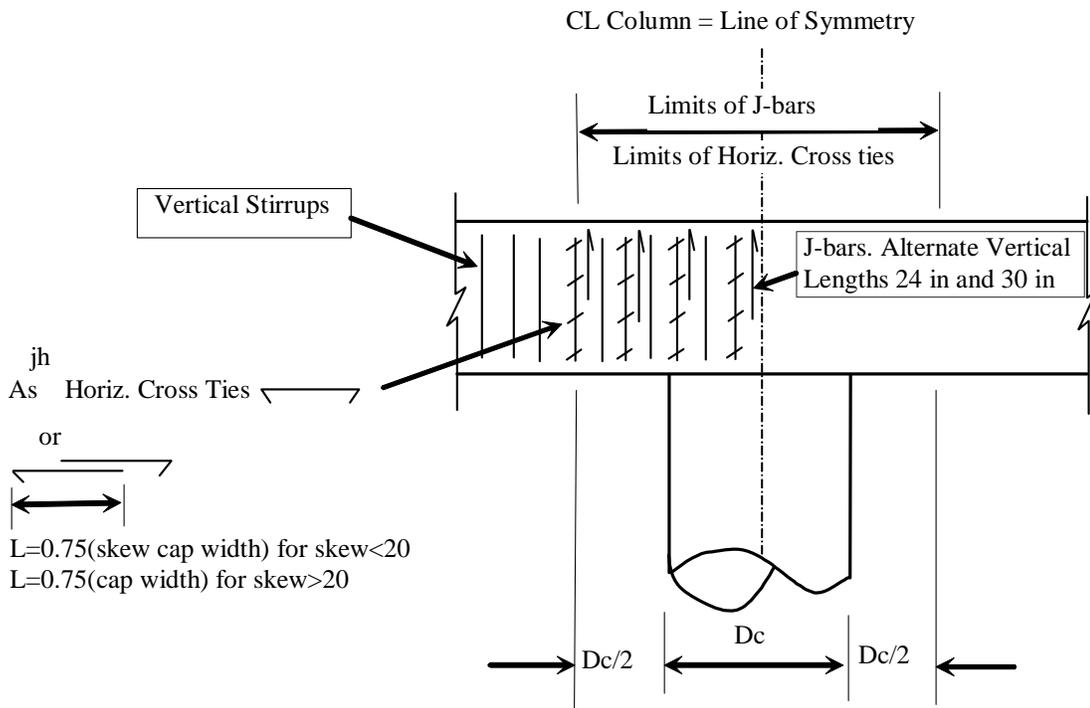


Figure 7.9 Location Of Horizontal Joint Shear Steel¹²

¹² Figures 7.8, 7.9 and 7.10 illustrate the general location for joint shear reinforcement in the bent cap.

Bent Cap Details, Section at Column for Bridges with Skew Larger than 20 Degrees.

(Detail Applies to Sections Within 2 x Diameter of Column, Centered About CL of Column).

(Detail Applies to T-Beam and Box Girder Bridges Where Deck Reinforcement is Placed Normal or Radial to CL Bridge).

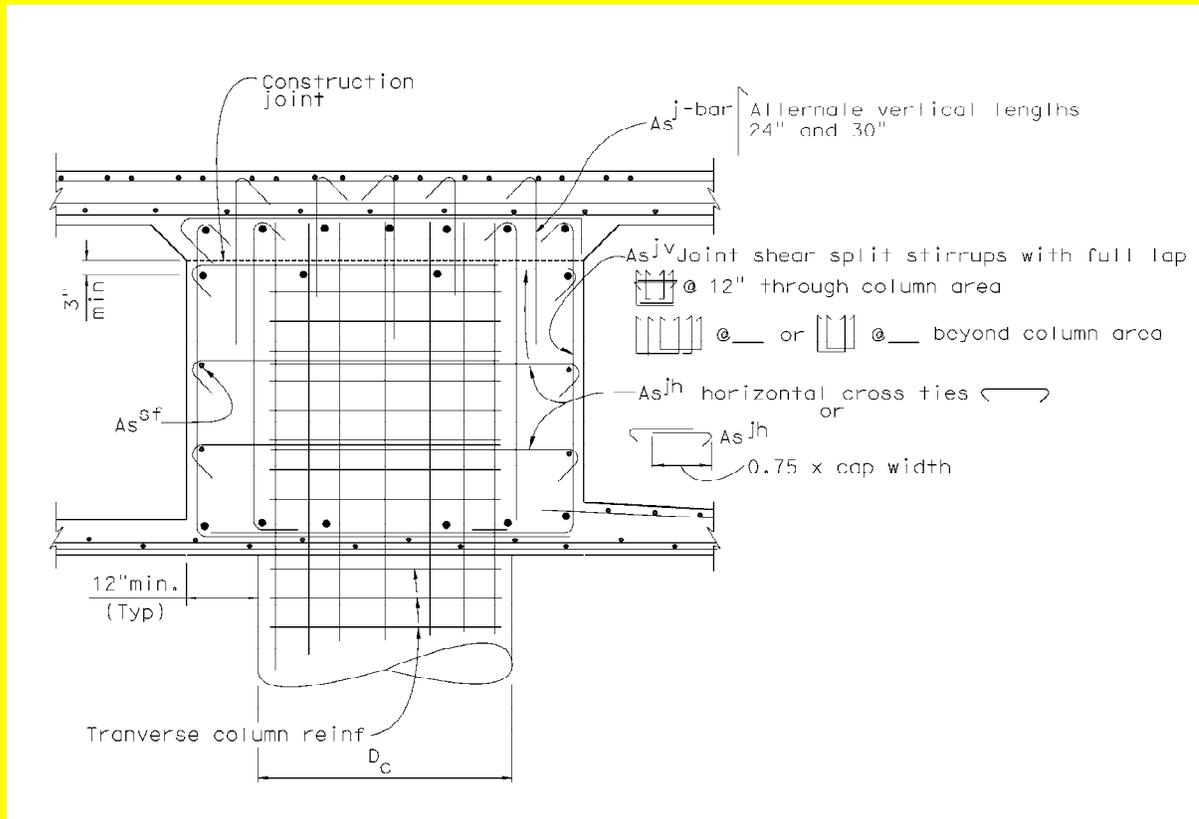


Figure 7.10 Additional Joint Shear Steel For Skewed Bridges¹³

¹³ Figures 7.8, 7.9 and 7.10 illustrate the general location for joint shear reinforcement in the bent cap.

7.4.5 Knee Joints

Knee joints differ from T joints because the joint response varies with the direction of the moment (opening or closing) applied to the joint (see Figures 7.10b). Therefore, knee joints must be evaluated for both opening and closing failure modes.

~~Knee joints require special reinforcing details that are considered non-standard and shall be included in the project specific seismic design criteria.~~

~~— It may be desirable to pin the top of the column to avoid knee joint requirements. This eliminates the joint shear transfer through the joint and limits the torsion demand transferred to the cap beam. However, the benefits of a pinned exterior joint should be weighed against increased foundation demands and the effect on the frame's overall performance.~~

In the opening moment case (Figure 7.10b-1), a series of arch-shaped cracks tends to form between the compression zones at the outside of the column and top of the beam. The intersection of the arch strut and the flexural compression zones at the top of the beam and the back of the column create outward-acting resultant forces. If the beam bottom reinforcement is anchored only by straight bar extension, there will virtually be no resistance to the horizontal resultant tensile force. It will cause vertical splitting, reducing competence of the anchorage of the outer column rebars and beam top rebars.

In the closing moment case (Figure 7.10b-2), a fan-shaped pattern of cracks develops, radiating from the outer surfaces of beam and column toward the inside corner. If there is no vertical reinforcement clamping the beam top reinforcement into the joint, the entire beam tension, T_b is transferred to the back of the joint as there isn't an effective mechanism to resist the moment at the base of the wedge-shaped concrete elements caused by bond-induced tension transfer to the concrete.

Knee joint shear reinforcement details for straight ($0 - 20^\circ$ skew) and skew ($> 20^\circ$ skew) bridge configurations are similar to those shown in Figures 7.8 and 7.10, respectively.

A) Bent Cap Top and Bottom Flexural Reinforcement – Use for both Cases 1 and 2

The top and bottom reinforcement within the bent cap width used to meet this provision shall be in the form of continuous U-bars with minimum area as specified in Equation 7.23d (see illustration in Figures 7.10e - 7.10g-1).

$$A_s^{u-bar} = 0.33 \times A_{st} \quad (7.23d)$$

where, A_{st} = Area of longitudinal column reinforcement anchored in the bent cap

The U-bars may be combined with bentcap main top and bottom reinforcement using mechanical couplers. Splices in the U-bars shall not be located within a distance, l_d from the interior face of the column.

B) Vertical Stirrups – Use for both Cases 1 and 2

Vertical stirrups or ties, A_s^{jv} as specified in Equation 7.23e, shall be placed transversely within each of regions 1, 2, and 3 of Figure 7.10d (see also Figures 7.8, 7.10, and 7.10g-2 for rebar placement).

$$A_s^{jv} = 0.2 \times A_{st} \quad (7.23e)$$

The stirrups provided in the overlapping areas shown in Figure 7.10d shall count towards meeting the requirements of both areas creating the overlap. These stirrups can be used to meet other requirements documented elsewhere including shear in the bent cap.

C) Horizontal Stirrups - Use for both Cases 1 and 2

Horizontal stirrups or ties, A_s^{jh} , as specified in Equation 7.23f, shall be placed transversely around the vertical stirrups or ties in two or more intermediate layers spaced vertically at not more than 18 inches (see Figures 7.8, 7.10, and 7.10g-2 for illustration).

$$A_s^{jh} = 0.1 \times A_{st} \quad (7.23f)$$

This horizontal reinforcement shall be placed within the limits shown in Figures 7.10e and 7.10f).

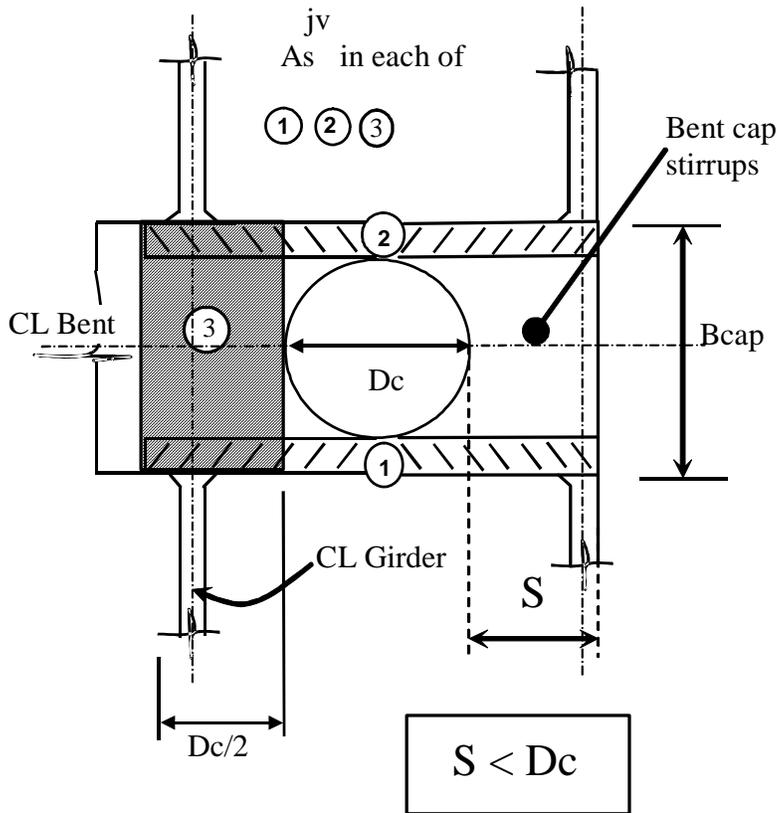


Figure 7.10d: Location of Knee Joint Vertical Shear Reinforcement (Plan view)

(D) Horizontal Side Reinforcement - Use for both Cases 1 and 2

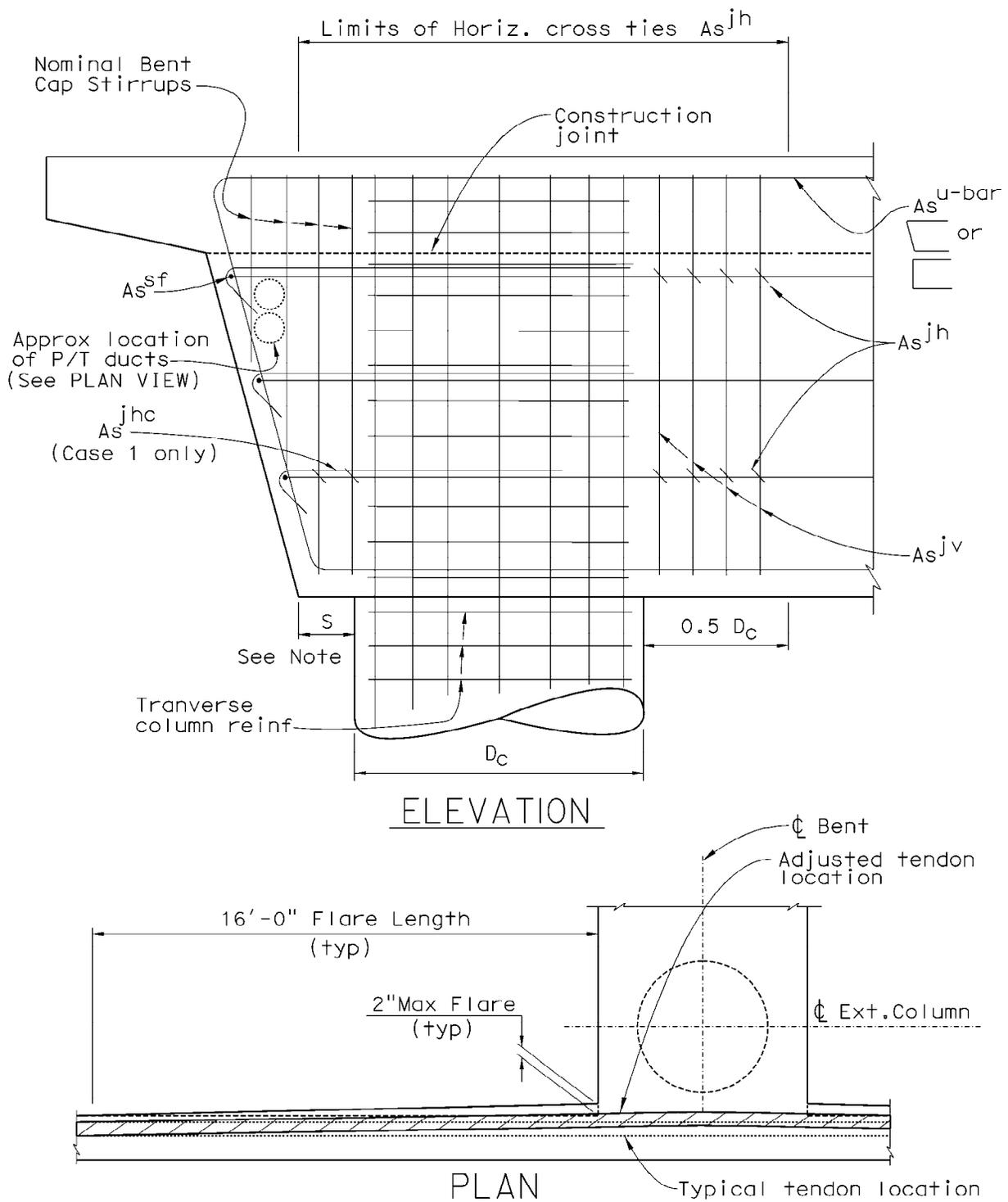
The total longitudinal side face reinforcement in the bent cap, A_s^{sf} shall be at least equal to the greater of the areas specified in Equation 7.23g and shall be placed near the side faces of the bent cap with a maximum spacing of 12 inches.

$$A_s^{sf} \geq \begin{cases} 0.1 \times A_{cap}^{top} \\ or \\ 0.1 \times A_{cap}^{bot} \end{cases} \quad (7.23g)$$

Where:

$A_{cap}^{top}, A_{cap}^{bot}$ = Area of bent cap top and bottom flexural steel, respectively.

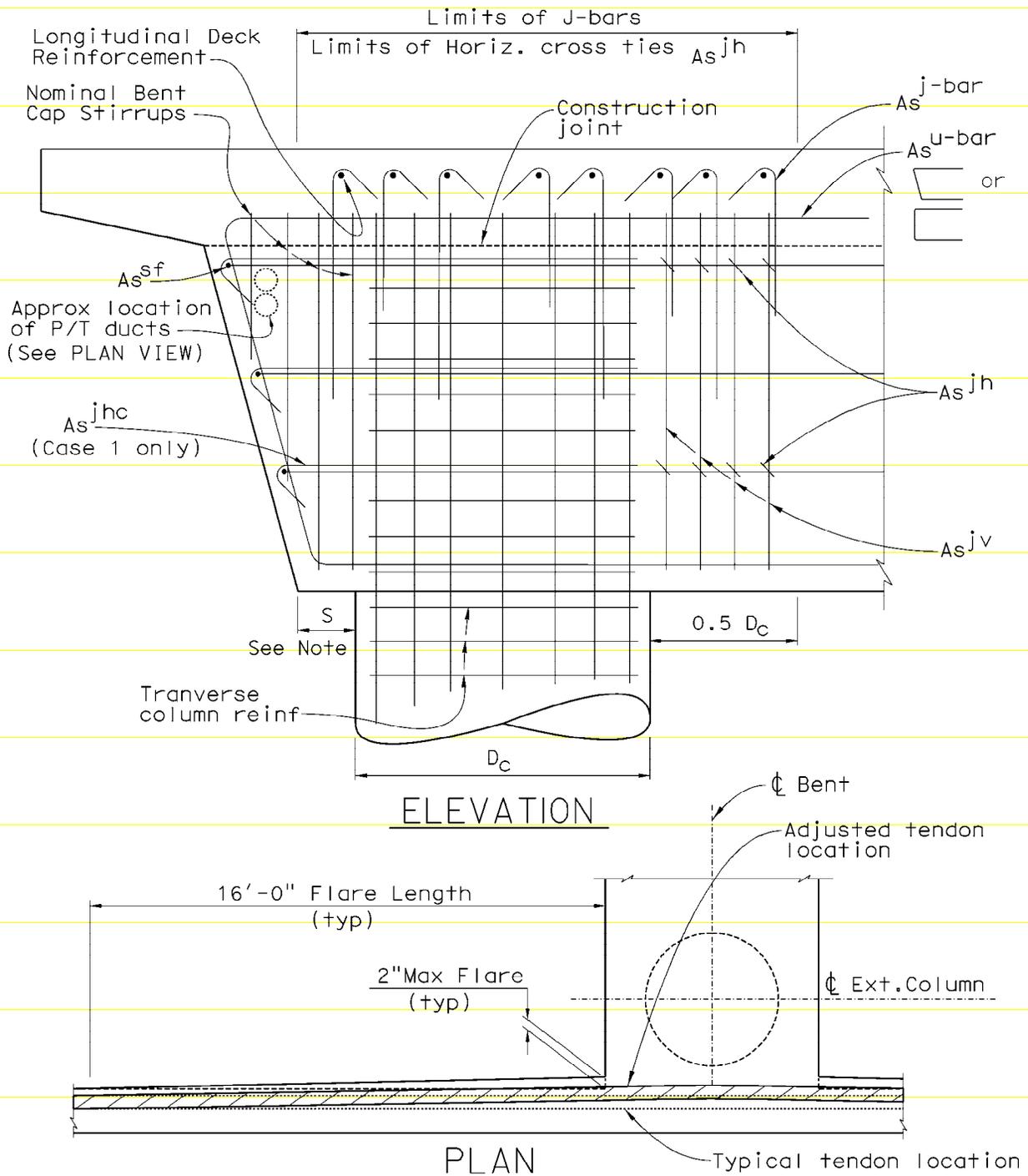
This side reinforcement shall be in the form of U-bars and shall be continuous over the exterior face of the Knee Joint. Splices in the U-bars shall be located at least a distance l_d from the interior face of the column. Any side reinforcement placed to meet other requirements shall count towards meeting this requirement.



NOTES:

1. CASE 1 Knee Joint: $S < D_c/2$
2. CASE 2 Knee Joint: $D_c/2 \leq S < \max(D_c, l_d)$
3. Flaring the exterior girders may be required for cast-in-place post-tensioned box girder construction in order to meet clearance requirements for ducts and mild reinforcement. For this situation, the inside face of exterior girders may be flared up to 2.5 inches at the bent cap. The flare length shall be 16 ft. To accommodate all girder and bent cap reinforcement in other situations, it may be necessary to adjust rebar positions to meet required concrete covers.

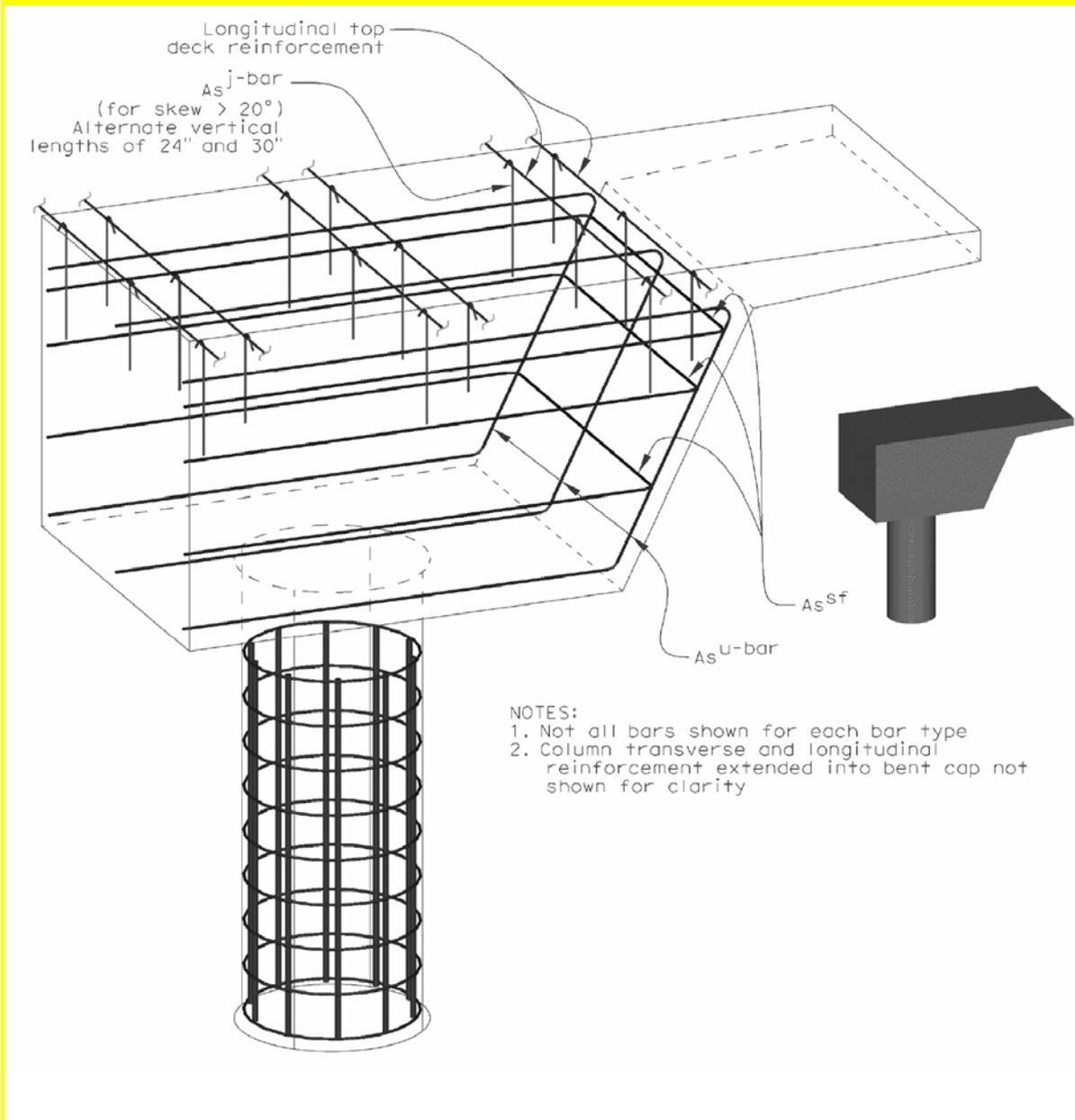
Figure 7.10e: Knee Joint Shear Reinforcement - Skew $\leq 20^\circ$



NOTES:

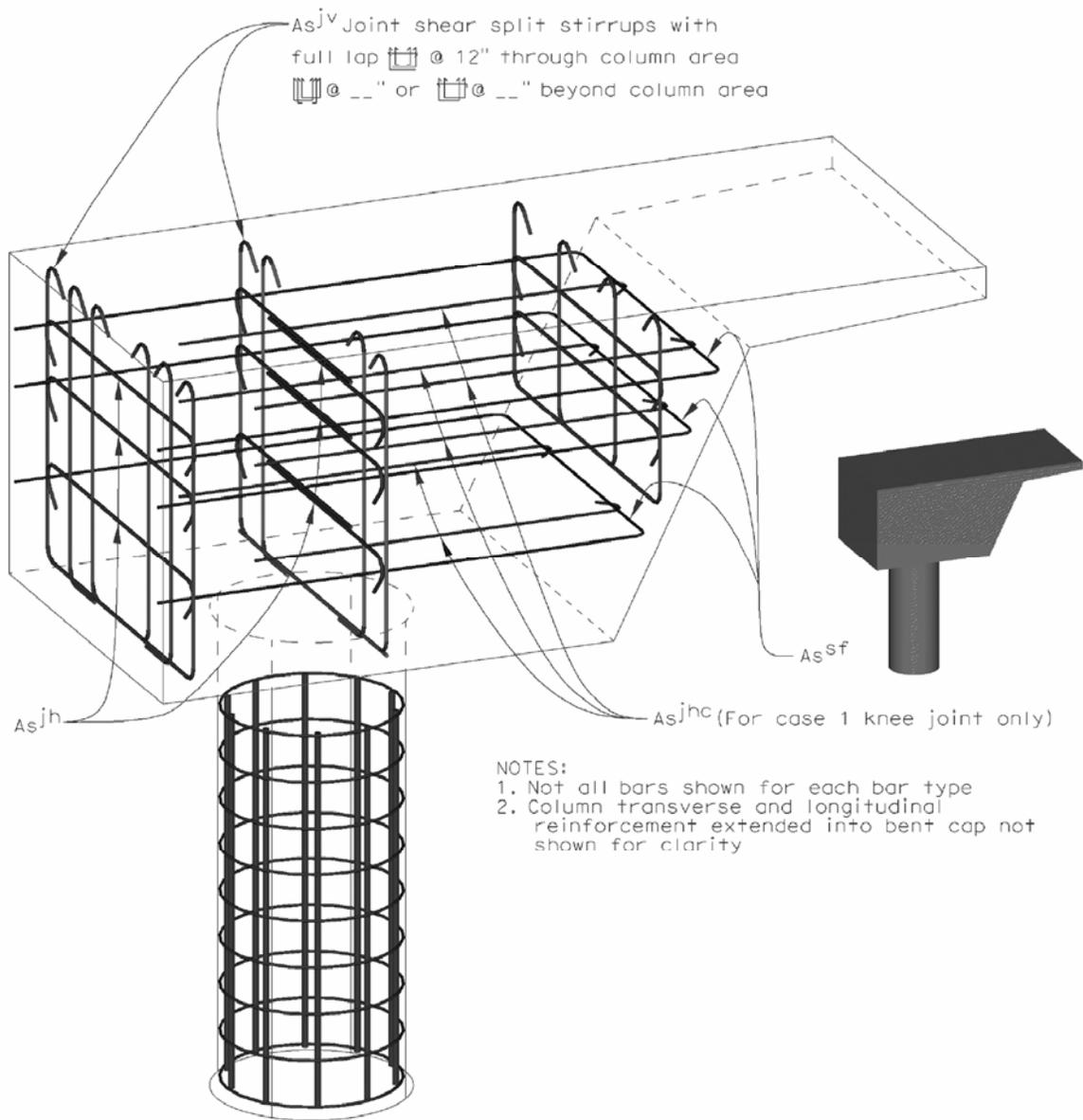
1. CASE 1 Knee Joint: $S < D_c/2$
2. CASE 2 Knee Joint: $D_c/2 \leq S < \max(D_c, l_d)$
3. Flaring the exterior girders may be required for cast-in-place post-tensioned box girder construction in order to meet clearance requirements for ducts and mild reinforcement. For this situation, the inside face of exterior girders may be flared up to 2.5 inches at the bent cap. The flare length shall be 16 ft. To accommodate all girder and bent cap reinforcement in other situations, it may be necessary to adjust rebar positions to meet required concrete covers.

Figure 7.10f: Knee Joint Shear Reinforcement - skew > 20



See Figure 7.10g-2 for 3-D representation of other knee joint shear bars not shown

Figure 7.10g-1: 3-D Representation of Knee Joint Shear Reinforcement



See Figure 7.10g-1 for 3-D representation of other knee joint shear bars not shown

Figure 7.10g-2: 3-D Representation of Knee Joint Shear Reinforcement

(E) Horizontal Cap End Ties (For Case 1 Only)

The total area of horizontal ties placed at the end of the bent cap, A_s^{jhc} (see Figures 7.10e, 7.10f, and 7.10g-2) shall be as specified in Equation 7.23h.

$$A_s^{jhc} = 0.33 \times A_s^{u-bar} \quad (7.23h)$$

This reinforcement shall be placed around the intersection of the bent cap horizontal side reinforcement and the continuous bent cap U-bar reinforcement, and spaced at not more than 12 inches vertically and horizontally. The horizontal reinforcement shall extend through the column cage to the interior face of the column.

F) J-Dowels - Use for both Cases 1 and 2

For bents skewed more than 20° , J-dowels hooked around the longitudinal top deck steel extending alternately 24 inches and 30 inches into the bent cap are required (see Figures 7.10, 7.10f, and 7.10g-1). The J-dowel reinforcement, A_s^{j-bar} shall be equal to or greater than the area specified in Equation 7.23i.

$$A_s^{j-bar} = 0.08 \times A_{st} \quad (7.23i)$$

The J-Dowels shall be placed within a rectangular region defined by the bent cap width and the limits shown in Figure 7.10f.

G) Transverse Reinforcement

Transverse reinforcement in the joint region shall consist of hoops with a minimum reinforcement ratio as specified in Equations 7.23j - 7.23l.

$$\rho_s = 0.6 \times \frac{\rho_l D_c}{l_{ac,provided}} \quad (\text{For Case 1 Knee joint}) \quad (7.23j)$$

$$\rho_s = 0.4 \times \frac{A_{st}}{l_{ac,provided}^2} \quad (\text{For Case 2 Knee joint, Integral bent cap}) \quad (7.23k)$$

$$\rho_s = 0.6 \times \frac{A_{st}}{l_{ac,provided}^2} \quad (\text{For Case 2 Knee joint, Non-integral bent cap}) \quad (7.23l)$$

where:

$l_{ac,provided}$ = Actual length of column longitudinal reinforcement embedded into the bent cap

ρ_l = Area ratio of longitudinal column reinforcement

The column transverse reinforcement extended into the bent cap may be used to satisfy this requirement. For interlocking cores, ρ_s shall be based on ρ_l of each core (for Case 1 knee joints) and on area of reinforcement A_{st} of each core (for Case 2 knee joints). All vertical column bars shall be extended as close as possible to the top bent cap reinforcement.

7.6.5.1 Horizontally Isolated Column Flares

The preferred method for detailing flares is to horizontally isolate the top of flared sections from the soffit of the cap beam. Isolating the flare allows the flexural hinge to form at the top of the column, minimizing the seismic shear demand on the column. The added mass and stiffness of the isolated flare typically can be ignored in the dynamic analysis.

A horizontal gap isolating the flare from the cap beam shall extend over the entire cross section of the flare excluding a core region equivalent to the prismatic column cross section. The gap shall be large enough so that it will not close during a seismic event. The gap thickness, G shall be based on the estimated ductility demand and corresponding plastic hinge rotation capacity. The minimum gap thickness shall be ~~2 inches (50 mm)~~ 4 inches (100 mm). See Section 7.6.2 for the appropriate plastic hinge length of horizontally isolated flares.

If the plastic hinge rotation based on the plastic hinge length specified in Section 7.6.2 (b) provides insufficient column displacement capacity, the designer may elect to add vertical flare isolation. When vertical flare isolation is used, the analytical plastic hinge length shall be taken as the lesser of L_p calculated using Equations 7.25 and 7.26 where G is the length from the bent cap soffit to the bottom of the vertical flare isolation region¹⁴.

¹⁴ The horizontal flare isolation detail is easier to construct than a combined horizontal and vertical isolation detail and is preferred wherever possible. Laboratory testing is scheduled to validate the plastic hinge length specified in equation 7.26.

7.7.1.1 Pile Foundations in Competent Soil

The lateral, vertical, and rotational capacity of the foundation shall exceed the respective demands. The size and number of piles and the pile group layout shall be designed to resist service level moments, shears, and axial loads and the moment demand induced by the column plastic hinging mechanism. Equations 7.28 and 7.29 define lateral shear and moment equilibrium in the foundation when the column reaches its overstrength capacity, see Figure 7.11.

$$V_o^{col} - \sum V_{(i)}^{pile} - R_s = 0 \quad (7.28)$$

$$M_o^{col} + V_o^{col} \times D_{ftg} + \sum M_{(i)}^{pile} - R_s \times (D_{ftg} - D_{R_s}) - \sum (C_{(i)}^{pile} \times c_{(i)}) - \sum (T_{(i)}^{pile} \times c_{(i)}) = 0 \quad (7.29)$$

$c_{(i)}$ = Distance from pile (i) to the center of gravity of the pile group in the X or Y direction

$C_{(i)}^{pile}$ = Axial compression demand on pile (i)

D_{ftg} = Depth of footing

D_{R_s} = Depth of resultant soil resistance measured from the top of footing

$M_{(i)}^{pile}$ = The moment demand generated in pile (i), $M_{(i)}^{pile} = 0$ if the piles are pinned to the footing

R_s = Estimated resultant soil resistance on the end of the footing

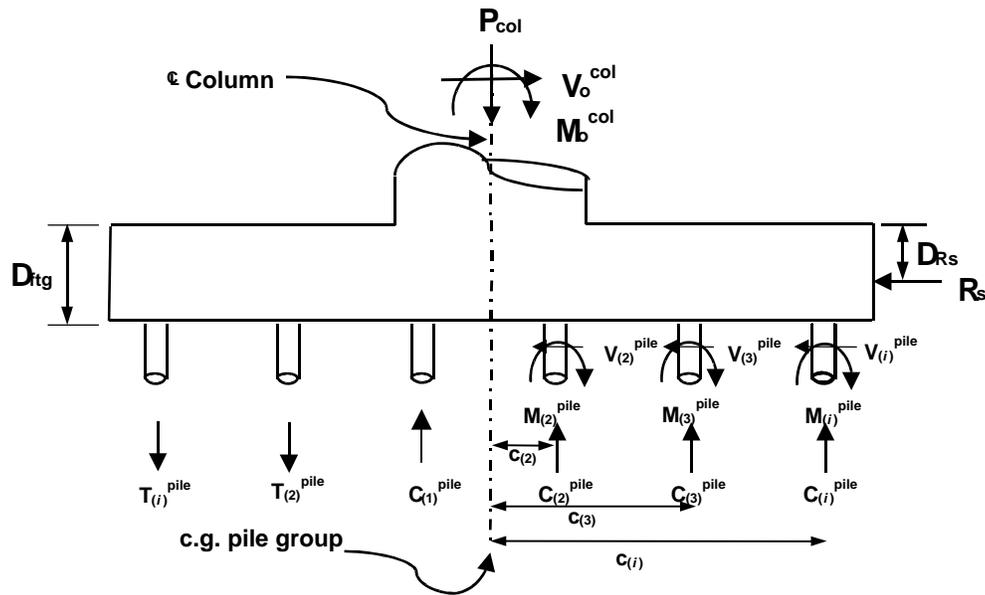
$T_{(i)}^{pile}$ = Axial tension demand on pile (i)

$V_{(i)}^{pile}$ = Lateral shear resistance provided by pile (i)

The design of pile foundations in competent soil can be greatly simplified if we rely on inherent capacity that is not directly incorporated in the foundation assessment. For example, typically pile axial resistance exceeds the designed nominal resistance and axial load redistributes to adjacent piles when an individual pile's geotechnical capacity is exceeded.

The simplified foundation model illustrated in Figure 7.12 is based on the following assumptions. A more sophisticated analysis may be warranted if project specific parameters invalidate any of these assumptions:

- The passive resistance of the soil along the leading edge of the footing and upper 4 to 8 pile diameters combined with the friction along the sides and bottom of the pile cap is sufficient to resist the column overstrength shear V_o^{col} .



Pile shears and moments shown on right side only, left side similar
Effects of footing weight and soil overburden not shown

Figure 7.11 Footing Force Equilibrium

- The pile cap is infinitely rigid, its width is entirely effective, and the pile loads can be calculated from the static equations of equilibrium.
- The pile group's nominal moment resistance is limited to the capacity available when any individual pile reaches its nominal axial resistance.
- Group effects for pile footings surrounded by competent soil and a minimum of three diameters center-to-center pile spacing are relatively small and can be ignored.
- Piles designed with a pinned connection to the pile cap will not transfer significant moment to the pile cap.
- In a competent soil, the moment at the top of the pile is relatively small and may be ignored.
- However, in a marginal or liquefiable soil, the effects of the plastic moment at the top of the pile, M_p^{pile} should be considered (see equations 7.31b and 7.31c).
- Pile groups designed with the simplified foundation model can be sized to resist the plastic moment of the column M_p in lieu of M_o .

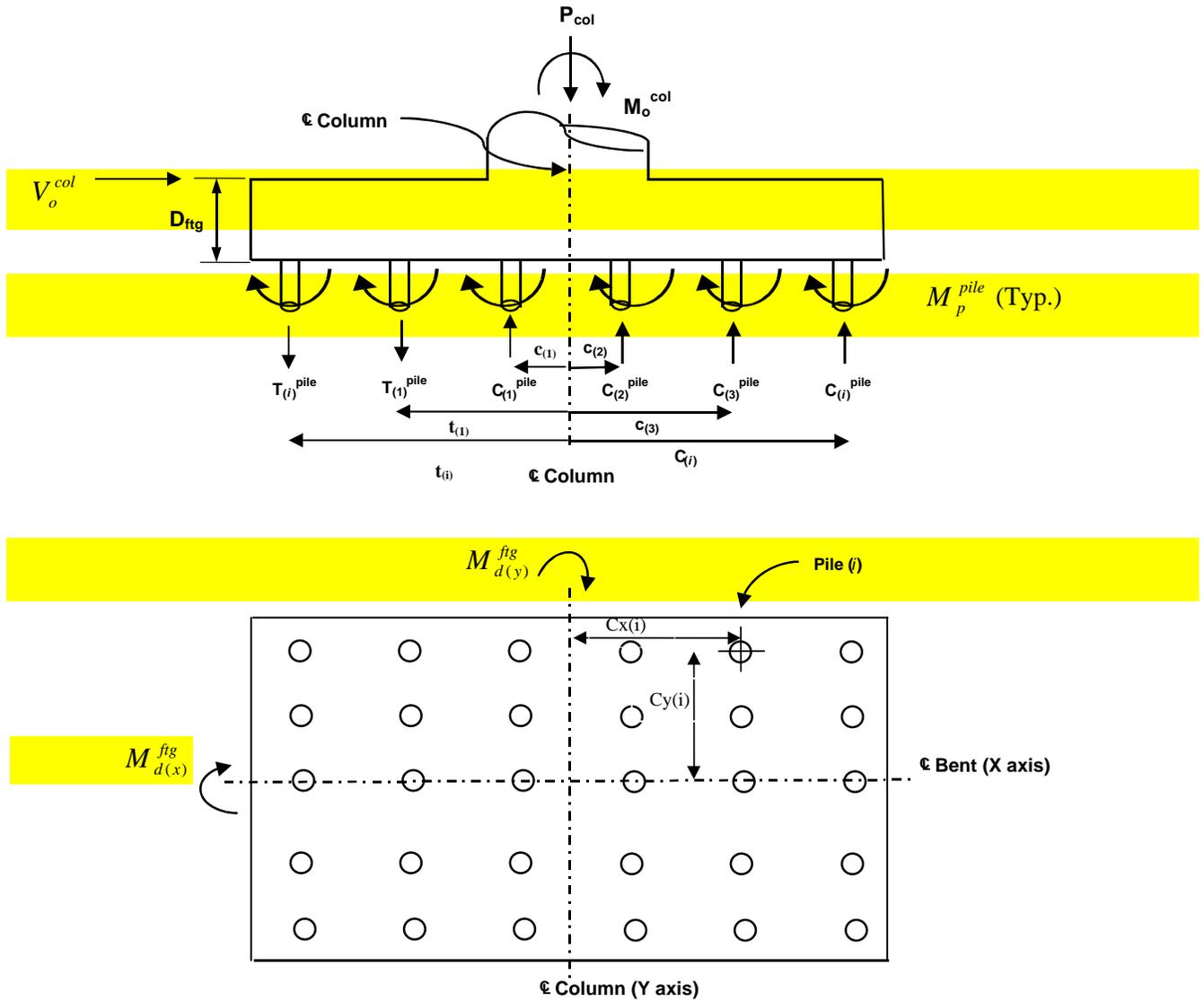


Figure 7.12 Simplified Pile Model For Foundations In Competent Soil

Equation 7.30 defines the axial demand on an individual pile when the column reaches its plastic hinging capacity based on force equilibrium in conjunction with the previously stated assumptions. A similar model can be used to analyze and design spread footing foundations that are surrounded by competent soil.

$$\left. \begin{array}{l} C_{(i)}^{pile} \\ T_{(i)}^{pile} \end{array} \right\} = \frac{P_p}{N_p} \pm \frac{M_{d(y)}^{fig} \times c_{x(i)}}{I_{p \cdot g \cdot (y)}} \pm \frac{M_{d(x)}^{fig} \times c_{y(i)}}{I_{p \cdot g \cdot (x)}} \quad (7.30)$$

Where:

$$I_{p.g.(x)} = \sum n \times c_{y(i)}^2 \quad I_{p.g.(y)} = \sum n \times c_{x(i)}^2 \quad (7.31a)$$

$$M_{d(y)}^{ftg} = M_{o(y)}^{col} + V_{o(x)}^{col} \times D_{ftg} + N_p \times M_{p(y)}^{pile} \quad (7.31b)$$

$$M_{d(x)}^{ftg} = M_{o(x)}^{col} + V_{o(y)}^{col} \times D_{ftg} + N_p \times M_{p(x)}^{pile} \quad (7.31c)$$

$I_{p.g.(x),(y)}$ = Moment of inertia of the pile group about the X or Y axis as defined in Equation 7.31

$M_{d(x),(y)}^{ftg}$ = The component of the moment demand on the footing about the X or Y axis

$M_{o(x),(y)}^{col}$ = The component of the column overstrength moment capacity about the X or Y axis

N_p = Total number of piles in the pile group

n = The total number of piles at distance $c_{(i)}$ from the centroid of the pile group

P_p = The total axial load on the pile group including column axial load (dead load+EQ load), footing weight, and overburden soil weight

$M_{p(x),(y)}^{pile}$ = The component of the pile plastic moment capacity at the pile cap connection due to total average axial load about the X or Y axis

$V_{o(x),(y)}^{col}$ = The component of column overstrength shear demand along the X or Y axis

Note that Equations 7.30, 7.31a, 7.31b, and 7.31c are used by the Caltrans WinFOOT Computer Program.

7.7.1.2.1 *Lateral Design*

In marginal soils the pile cap may not dominate the lateral stiffness of the foundation, as is expected in competent soil, possibly leading to significant lateral displacements. The designer shall verify that the lateral capacity of the foundation exceeds the lateral demand transmitted by the column, including the pile's capability of maintaining axial load capacity at the expected lateral displacement. **A lateral analysis of pile footings may be performed using a more sophisticated computer program such as LPILE, GROUP, SAP2000, or WFRAME.**

The designer should select the most cost effective strategy for increasing the lateral resistance of the foundation when required. The following methods are commonly used to increase lateral foundation capacity.

- Deepen the footing/pile cap to increase passive resistance
- Increase the amount of fixity at the pile/footing connection and strengthen the upper portion of the pile
- Use a more ductile pile type that can develop soil resistance at larger pile deflections
- Add additional piles

7.7.1.4 Footing Joint Shear

All footing/column moment resisting joints shall be proportioned so the principal stresses meet the following criteria:

$$\text{Principal compression: } p_c \leq 0.25 \times f'_c \quad (7.33)$$

$$\text{Principal tension: } p_t \leq \begin{cases} 12 \times \sqrt{f'_c} & (\text{psi}) \\ 1.0 \times \sqrt{f'_c} & (\text{MPa}) \end{cases} \quad (7.34)$$

Where:

$$p_t = \frac{f_v}{2} - \sqrt{\left(\frac{f_v}{2}\right)^2 + v_{jv}^2} \quad (7.35)$$

$$p_c = \frac{f_v}{2} + \sqrt{\left(\frac{f_v}{2}\right)^2 + v_{jv}^2} \quad (7.36)$$

$$v_{jv} = \frac{T_{jv}}{B_{eff}^{ftg} \times D_{ftg}} \quad (7.37)$$

(See Figure 7.13)

$$T_{jv} = T_c - \sum T_{(i)}^{pile} \quad (7.38)$$

T_c = Column tensile force associated with M_o^{col}

$\sum T_{(i)}^{pile}$ = Summation of the hold down force in the tension piles.

$$B_{eff}^{ftg} = \begin{cases} \sqrt{2} \times D_c & \text{Circular Column} \\ B_c + D_c & \text{Rectangular Column} \end{cases} \quad (7.39)$$

$$f_v = \frac{P_{col}}{A_{jh}^{ftg}} \quad (7.40)$$

(see Figure 7.13a)

P_{col} = Column axial force including the effects of overturning

$$A_{jh}^{ftg} = \left((D_c + D_{ftg}) \times (B_c + D_{ftg}) \right) \quad (7.41)$$

where:

A_{jh}^{ftg} = the effective horizontal area at mid-depth of the footing, assuming a 45° spread away from the boundary of the column in all directions, see Figure 7.13.

D_c = Column cross-sectional dimension in the **dimension** direction of interest.

For circular or square columns, $B_c = D_c$

For rectangular columns, B_c = the other column cross-section dimension

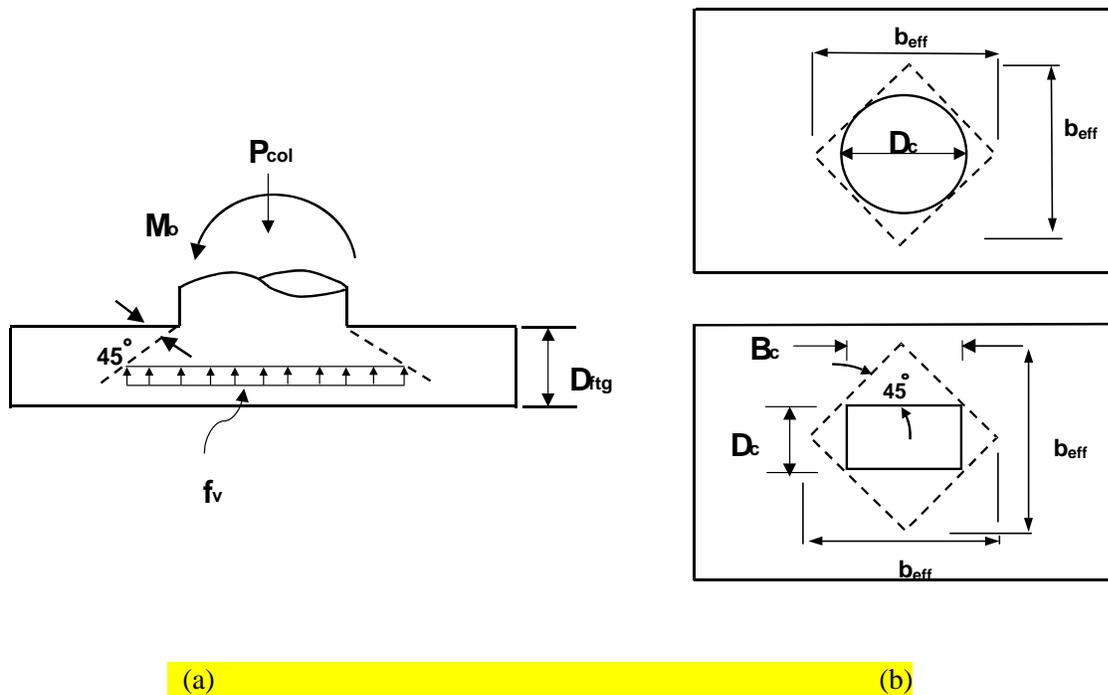


Figure 7.13 Assumed Effective Dimensions for Footing Joint Stress Calculation

7.7.1.7 Use of “T” Headed Stirrups and Bars in Footings

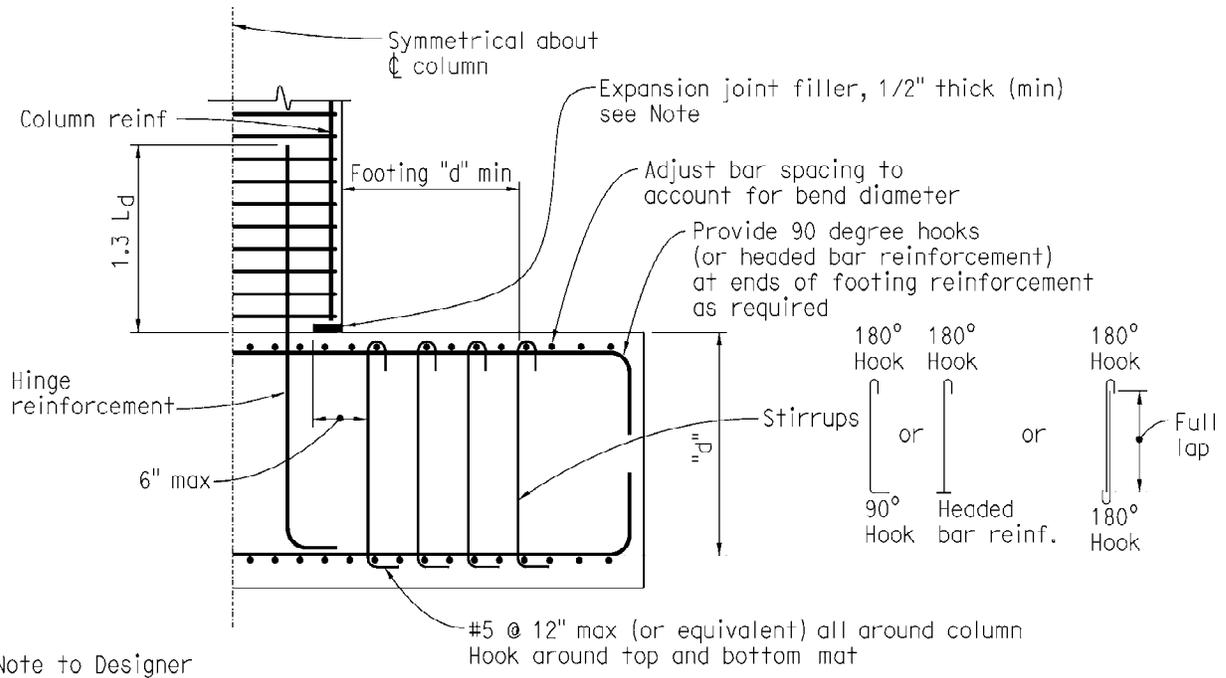
—Use of T headed stirrups in footings The types of hooks used for stirrups in footings depends on the column fixity condition and the level of principal tensile stress. The following guidelines are recommended to assist engineers in regards to the choice of T head versus 90 degree hook stirrup in footings. To assist engineers with the proper choice of hooks for footing stirrups, the following stirrup configurations are defined: (a) Stirrups with 180-degree hooks at the top and 90-degree hooks at the bottom, (b) Stirrups with 180-degree hooks at the top and T-heads at the bottom, and (c) Fully lapped stirrups with 180-degree hooks at opposite ends.

- For pinned-column footings, use stirrups with 180-degree hooks at the top and either 90-degree hooks or T-heads at the bottom may be used (See Figure in BDD 7-7.1.3e) stirrup type (a) or (b) or (c) may be used (See Figure 7.13c).
- For fixed-column footings, a “T” head must be used if calculate the principal tensile stress demand in the footing (see Section 7.7.1.4) exceeds $3.5\sqrt{f'_c}$ (psi) [$0.29\sqrt{f'_c}$ (MPa)] and compare it to the threshold value of $0.29\sqrt{f'_c}$ (MPa) [$3.5\sqrt{f'_c}$ (psi)] stirrup type (b) or (c) shall be used if the principal tensile stress demand (see Section 7.7.1.4) in the footing exceeds $3.5\sqrt{f'_c}$ (psi) [$0.29\sqrt{f'_c}$ (MPa)]. If the principal tensile stress demand is less than $3.5\sqrt{f'_c}$ (psi) [$0.29\sqrt{f'_c}$ (MPa)] [$3.5\sqrt{f'_c}$ (psi)], use the same detail as in the pinned column footing case. otherwise use a “T” head at the bottom of the stirrup in place of the 90 degree hook, to account for joint shear effects. The region around the column bounded by a distance of $D_c/2$ from the face of the column is recommended for the stirrup placement (See Figure in BDD 7-6 7.13d). If the principal tensile stress demand is less than $3.5\sqrt{f'_c}$ (psi) [$0.29\sqrt{f'_c}$ (MPa)], stirrup type (a) or (b) or (c) may be used.

The designer may avoid the use of “T” heads by increasing the depth of the footing and reducing the principal stress demand below $3.5\sqrt{f'_c}$ (psi) [$0.29\sqrt{f'_c}$ (MPa)] [$3.5\sqrt{f'_c}$ (psi)].

The designer needs to check for shall ensure development of the main footing bars beyond the centerline of piles near the footing edges and provide a 90-degree hook or “T” head, if development of the bar is needed.

The bar size in the footing mats along with the principal tensile stress level and the spacing of the mat are all critical factors in the choice of the stirrup bar size. Use of #18 (Metric #57) bars in footings needs a careful review as it affects the choice of the stirrup bar and hook detailing to fit the mat.



The thickness of the expansion joint filler should allow for maximum column deflection and prevent crushing the edge of the column concrete against the footing

Figure 7.13c Footing Reinforcement – Pinned Column

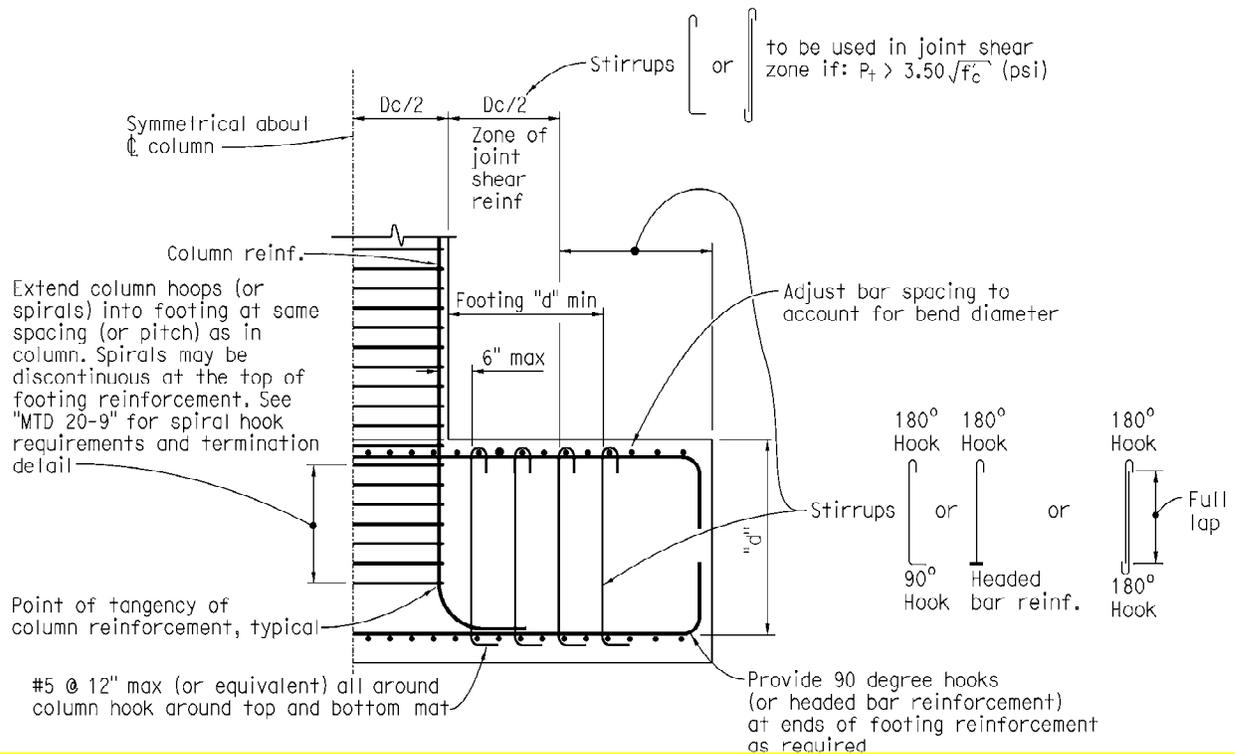


Figure 7.13d Footing Reinforcement – Fixed Column

7.8.1 Longitudinal Abutment Response

The backfill passive pressure force resisting movement at the abutment varies nonlinearly with longitudinal abutment displacement and is dependent upon the material properties of the backfill. Abutment longitudinal response analysis may be accomplished by using a bilinear approximation of the force-deformation relationship as detailed herein or by using the nonlinear force-deformation relationship documented in Reference [15].

The ~~linear-elastic~~ bilinear demand model shall include an effective abutment stiffness, K_{eff} that accounts for expansion gaps, and incorporates a realistic value for the embankment fill response. ~~The abutment embankment fill stiffness is nonlinear and is dependent upon on the material properties of the abutment backfill.~~ Based on passive earth pressure tests and the force deflection results from large-scale abutment testing at UC Davis [13] and UCLA [16] and idealized by Reference [17], the initial ~~embankment fill~~ stiffness is $K_i \approx 20 \frac{\text{kip/in}}{\text{ft}}$ ($11.5 \frac{\text{kN/mm}}{\text{m}}$). K_i for embankment fill material meeting the requirements of Caltrans Standard Specifications is estimated as shown in Equation 7.43a.

$$K_i \approx \frac{50 \text{ kip/in}}{\text{ft}} \quad \left(\frac{28.70 \text{ KN/mm}}{\text{m}} \right) \quad (7.43a)$$

For embankment fill material not meeting the requirements of the Standard Specifications, the initial embankment fill stiffness may be taken as $K_i \approx 25 \frac{\text{kip/in}}{\text{ft}}$ ($14.35 \frac{\text{kN/mm}}{\text{m}}$).

The initial stiffness¹⁵ shall be adjusted proportional to the backwall/diaphragm height, as documented in Equation 7.43b.

$$K_{abut} = \begin{cases} K_i \times w \times \left(\frac{h}{5.5 \text{ ft}} \right) & \text{U.S. units} \\ K_i \times w \times \left(\frac{h}{1.7 \text{ m}} \right) & \text{S.I. units} \end{cases} \quad (7.43b)$$

where, w is the **projected** width of the backwall or ~~the~~ diaphragm for seat and diaphragm abutments, respectively (see Figures 7.14B and 7.14C for effective abutment dimensions).

For seat-type abutments, the effective abutment wall stiffness K_{eff} shall account for the expansion hinge gaps as shown in Figure 7.14A.

Based on a bilinear idealization of the force-deformation relationship (see Figure 7.14A), the passive pressure force resisting the movement at the abutment (P_{bw} or P_{dia}) is calculated according to Equation 7.44.

¹⁵ This proportionality may be revised in future as more data becomes available.

The passive pressure resisting the movement at the abutment increases linearly with the displacement, as shown in Figure 7.14A.

$$P_{bw \text{ or } P_{dia}} = \begin{cases} A_e \times 5.0 \text{ ksf} \times \left(\frac{h_{bw \text{ or } h_{dia}}}{5.5}\right) & (\text{ft, kip}) \\ A_e \times 239 \text{ kPa} \times \left(\frac{h_{bw \text{ or } h_{dia}}}{1.7}\right) & (\text{m, kN}) \end{cases} \quad (7.44)$$

The maximum passive pressure of 5.0 ksf (239 kPa), presented in Equation 7.44 is based on the ultimate static force developed in the full scale abutment testing conducted at UC Davis [13, 16]. The height proportionality factor, $\frac{h}{5.5 \text{ ft}} \left(\frac{h}{1.7 \text{ m}}\right)$ is based on the height of the tested UC Davis abutment walls, specimen 5.5 ft (1.7 m).

The effective abutment wall area, A_e for calculating the ultimate longitudinal force capacity of an abutment is presented in Equation 7.45a.

$$A_e = \begin{cases} h_{bw} \times w_{bw} & \text{for Seat Abutments} \\ h_{dia} \times w_{dia} & \text{for Diaphragm Abutments} \end{cases} \quad (7.45a)$$

where: $h_{dia} = h_{dia}^*$ = Effective height if the diaphragm is not designed for full soil pressure (see Figure 7.14B)

$h_{dia} = h_{dia}^{**}$ = Effective height if the diaphragm is designed for full soil pressure (see Figure 7.14B)

w_{bw} , w_{dia} , w_{abut} = Effective abutment widths corrected for skew (see Figures 7.14B and 7.14C)

For seat abutments the backwall is typically designed to break off in order to protect the foundation from inelastic action. The area considered effective for mobilizing the backfill longitudinally is equal to the area of the backwall.

For diaphragm abutments the entire diaphragm, above and below the soffit, is typically designed to engage the backfill immediately when the bridge is displaced longitudinally. Therefore, the effective abutment area is equal to the entire area of the diaphragm. If the diaphragm has not been designed to resist the passive earth pressure exerted by the abutment backfill, the effective abutment area is limited to the portion of the diaphragm above the soffit of the girders.

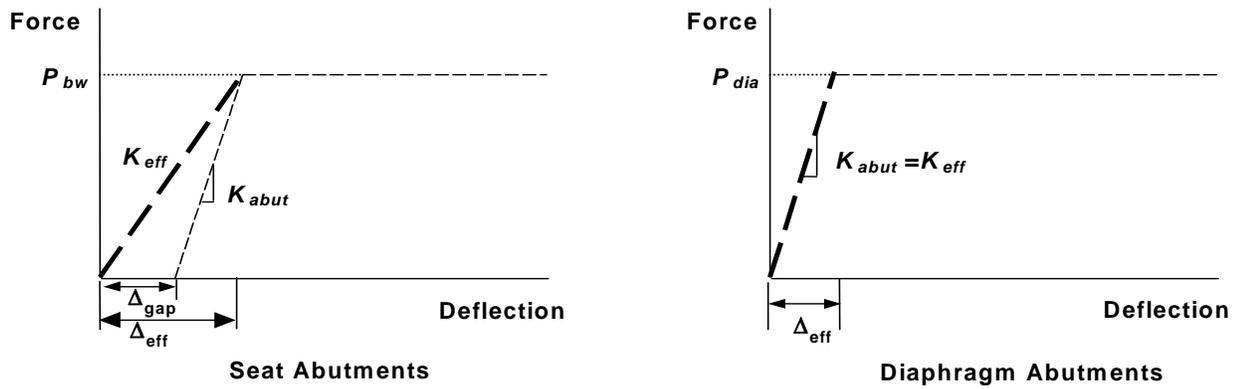


Figure 7.14A Effective Abutment Stiffness

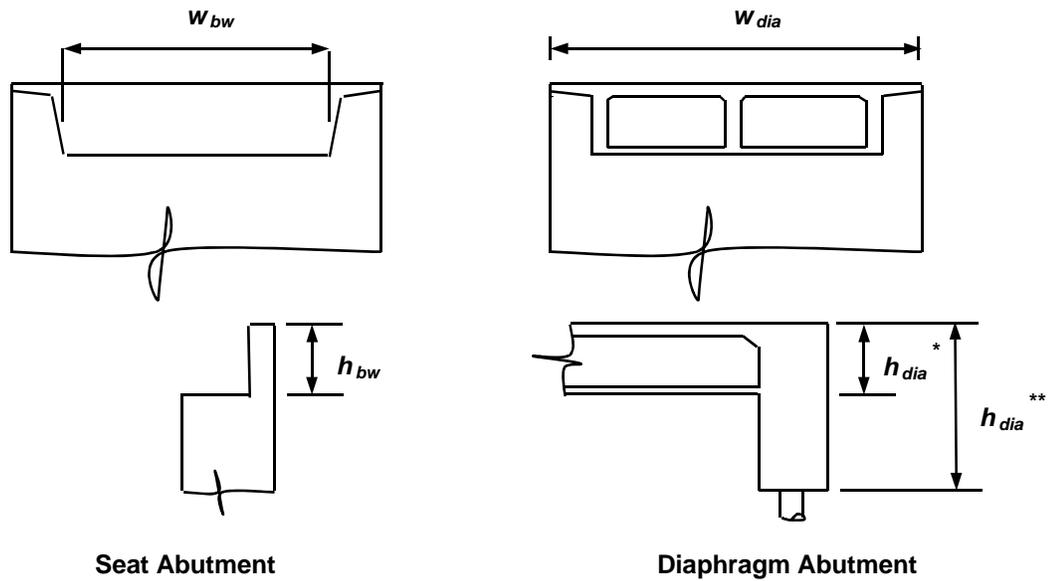


Figure 7.14B Effective Abutment Area

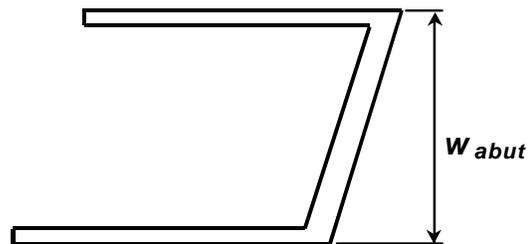


Figure 7.14C Effective Abutment Width For Skewed Bridges

The abutment displacement coefficient R_A , shall be used in the assessment of the effectiveness of the abutment (see Equation 7.45b).

$$R_A = \Delta_D / \Delta_{eff} \quad (7.45b)$$

where: Δ_D = The longitudinal displacement demand at the abutment from elastic analysis.

Δ_{eff} = The effective longitudinal abutment displacement at idealized yield.

If $R_A \leq 2$: The elastic response is dominated by the abutments. The abutment stiffness is large relative to the stiffness of the bents or piers. The column displacement demands generated by the linear elastic model can be used directly to determine the displacement demand and capacity assessment of the bents or piers.

If $R_A \geq 4$: The elastic model is insensitive to the abutment stiffness. The abutment contribution to the overall bridge response is small and the abutments are insignificant to the longitudinal seismic performance. The bents and piers will sustain significant deformation. The effective abutment stiffness K_{eff} in the elastic model shall be reduced to a minimum residual stiffness K_{res} , (see Equation 7.45c) and the elastic analysis shall be repeated for revised column displacements. The residual spring has no relevance to the actual stiffness provided by the failed backwall or diaphragm but should suppress unrealistic response modes associated with a completely released end condition.

$$K_{res} \approx 0.1 \times K_{eff} \quad (7.45c)$$

If $2 < R_A < 4$: The abutment stiffness in the elastic model shall be adjusted by interpolating effective abutment stiffness between K_{eff} and the residual stiffness K_{res} based on the R_A value. The elastic analysis shall be repeated to obtain revised column displacements.

7.8.2 Transverse Abutment Response

Seat type abutments are designed to resist transverse service load and moderate earthquake demands levels of ground motion elastically. Typically seat abutments cannot be elastically designed to elastically resist MCE the design earthquake demands because linear elastic analysis cannot capture the inelastic response of the shear keys, wingwalls, or piles. The lateral transverse capacity of seat abutments should not be considered effective for the MCE design seismic hazards unless the designer can demonstrate the force-deflection characteristics and stiffness for each element that contributes to the transverse resistance.

The magnitude of the transverse abutment stiffness and the resulting displacement is most critical in the design of the adjacent bent, not the abutment itself. Reasonable transverse displacement of superstructure relative to the abutment seat can easily be accommodated without catastrophic consequences. A nominal transverse spring stiffness, K_{nom} equal to 50% of the elastic transverse stiffness of the adjacent bent shall be used at the abutment in the elastic demand assessment models. The nominal spring stiffness, K_{nom} has no direct correlation or relevance to the actual residual stiffness (if any) provided by the failed shear key but should suppress unrealistic response modes associated with a completely released end condition. This approach is consistent with the stand-alone pushover analysis based design of the adjacent bents and it is conservative since larger additional amounts of lateral resistance at the abutments that are not generally captured by the nominal spring will only reduce the transverse displacement demands at the bents. Any additional element, such as pile shafts (used for transverse ductility), shall be included in the transverse analysis with a characteristic force-deflection curve. The initial slope of the force-deflection curve shall be included in the elastic demand assessment model.

Transverse stiffness of Diaphragm type abutments supported on standard piles surrounded by dense or hard material can conservatively be estimated, ignoring the wingwalls, as 40 kips/in (7.0 kN/mm) per pile.

7.8.3 Abutment Seat Width

Sufficient abutment seat width shall be available to accommodate the anticipated thermal movement, prestress shortening, creep, shrinkage, and the relative longitudinal earthquake displacement. The seat width normal to the centerline of bearing shall be calculated by Equation 7.46 but shall not be less than 30 inches (760 mm).

$$N_A \geq \begin{cases} (\Delta_{p/s} + \Delta_{cr+sh} + \Delta_{temp} + \Delta_{eq} + 4) & \text{(in)} \\ (\Delta_{p/s} + \Delta_{cr+sh} + \Delta_{temp} + \Delta_{eq} + 100) & \text{(mm)} \end{cases} \quad (7.46)$$

N_A = Abutment seat width normal to the centerline of bearing

$\Delta_{p/s}$ = Displacement attributed to pre-stress shortening

Δ_{cr+sh} = Displacement attributed to creep and shrinkage

Δ_{temp} = Displacement attributed to thermal expansion and contraction

Δ_{eq} = The largest relative earthquake displacement between the superstructure and the abutment calculated by the global or stand-alone analysis

The “Seat Width” requirements due to the service load considerations (Caltrans AASHTO LRFD Bridge Design Specifications and AASHTO requirements) shall also be met.

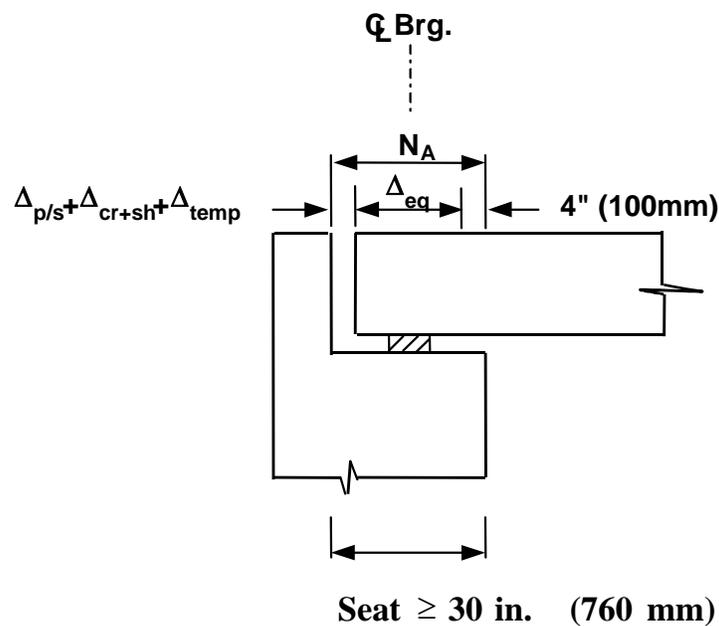


Figure 7.15 Abutment Seat Width Requirements

7.8.4 Abutment Shear Key Design

Typically abutment shear keys are expected to transmit the lateral shear forces generated by small to moderate earthquakes and service loads. Determining the earthquake force demand on shear keys is difficult. The forces generated with elastic demand assessment models should not be used to size the abutment shear keys. Shear key capacity for seat abutments supported on piles and spread footings shall be limited to the smaller of the following, determined according to Equations 7.47 (a-d).

$$F_{sk} \leq \begin{cases} 0.75 \times \sum V_{pile} & \sum V_{pile} = \text{Sum of the lateral pile capacity} \\ 0.3 \times P_{dl}^{sup} & P_{dl}^{sup} = \text{Axial dead load reaction at the abutment} \end{cases} \quad (7.47)$$

$$F_{sk} = \alpha \times (0.75 \times V_{piles} + V_{ww}) \quad \text{For Abutment on piles} \quad (7.47a)$$

$$F_{sk} = \alpha \times P_{dl} \quad \text{For Abutment on Spread footing} \quad (7.47b)$$

in which,

$$0.5 \leq \alpha \leq 1 \quad (7.47c)$$

where:

F_{sk} = Abutment shear key force capacity (kips)

V_{piles} = Sum of lateral capacity of the piles (kips)

V_{ww} = Shear capacity of one wingwall (kips)

P_{dl} = Superstructure dead load reaction at the abutment plus the weight of the abutment and its footing (kips)

α = factor that defines the range over which F_{sk} is allowed to vary

Note that the shear keys for abutments supported on spread footings are only designed to $0.3 \times P_{dl}^{sup}$.

It is recognized that the shear key design limits in Equation 7.47a may not be feasible for high abutments where unusually large number of piles support the abutment structure. In such cases it is recommended that the shear key be designed for the lateral strength specified in Equation 7.47d, provided the value of F_{sk} is less than that furnished by Equation 7.47a.

$$F_{sk} = \alpha \times P_{dl}^{\text{sup}} \quad (7.47d)$$

where:

P_{dl}^{sup} = Superstructure dead load reaction at the abutment.

The limits of α are as defined in Equation 7.47c.

7.8.4.1 Abutment Shear Key Reinforcement

Abutment shear key reinforcement may be designed using Equations 7.48 and 7.52 (referred to herein as the Isolated shear key method) or Equations 7.49, 7.50, 7.51, and 7.53 (referred to herein as the Non-isolated shear key or Shear friction design method). Shear key construction using normal weight concrete placed monolithically is assumed.

Equations 7.48 and 7.52 and the reinforcement details shown in Figure 7.16(A) are based on experimental tests on exterior shear keys conducted at UCSD [18]. This reinforcing detail (Figure 7.16A) was developed to ensure that exterior shear keys fail through a well-defined horizontal plane that is easily repaired after an earthquake, and is recommended for exterior shear key design for bridge abutments with skews $\leq 20^\circ$. Figure 7.16 shows typical reinforcing details for abutment shear keys designed using both methods.

A) Vertical Shear Key Reinforcement

For the Isolated key design method, the required area of interface shear reinforcement crossing the shear plane, A_{sk} is given by Equation 7.48.

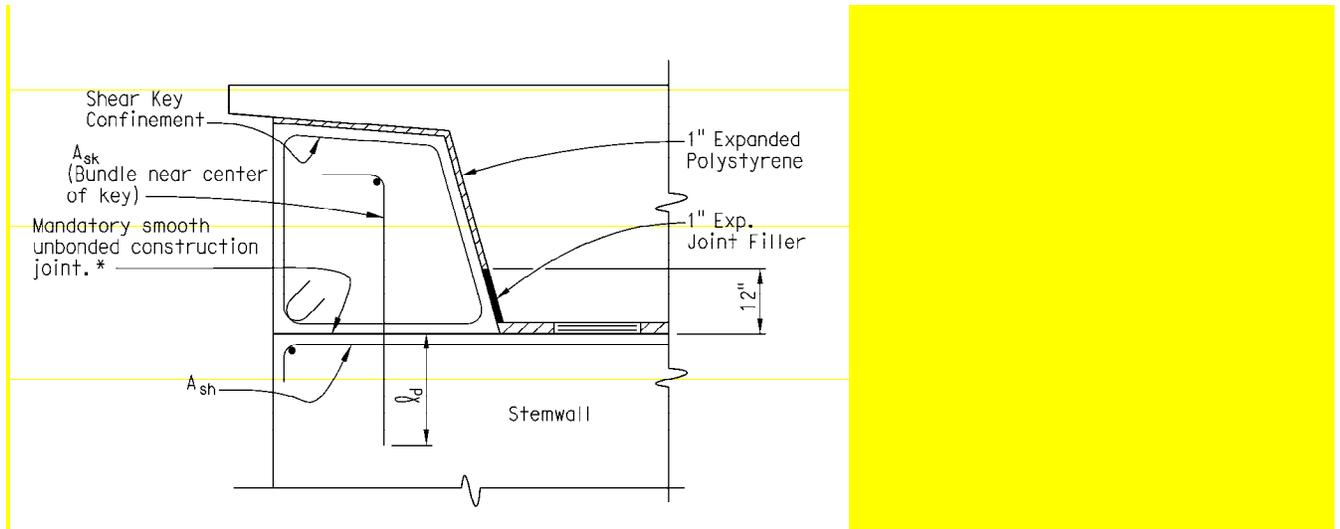
$$A_{sk} = \frac{F_{sk}}{1.8 \times f_{ye}} \quad \text{Isolated shear key} \quad (7.48)$$

The shear key vertical reinforcement provided above should be placed in a single line parallel to the bridge, and as close as possible to the center of the key, transversely (see Figure 7.16A).

If the Non-isolated key or Shear-friction design method is used, A_{sk} , is given by (see Figure 7.16B):

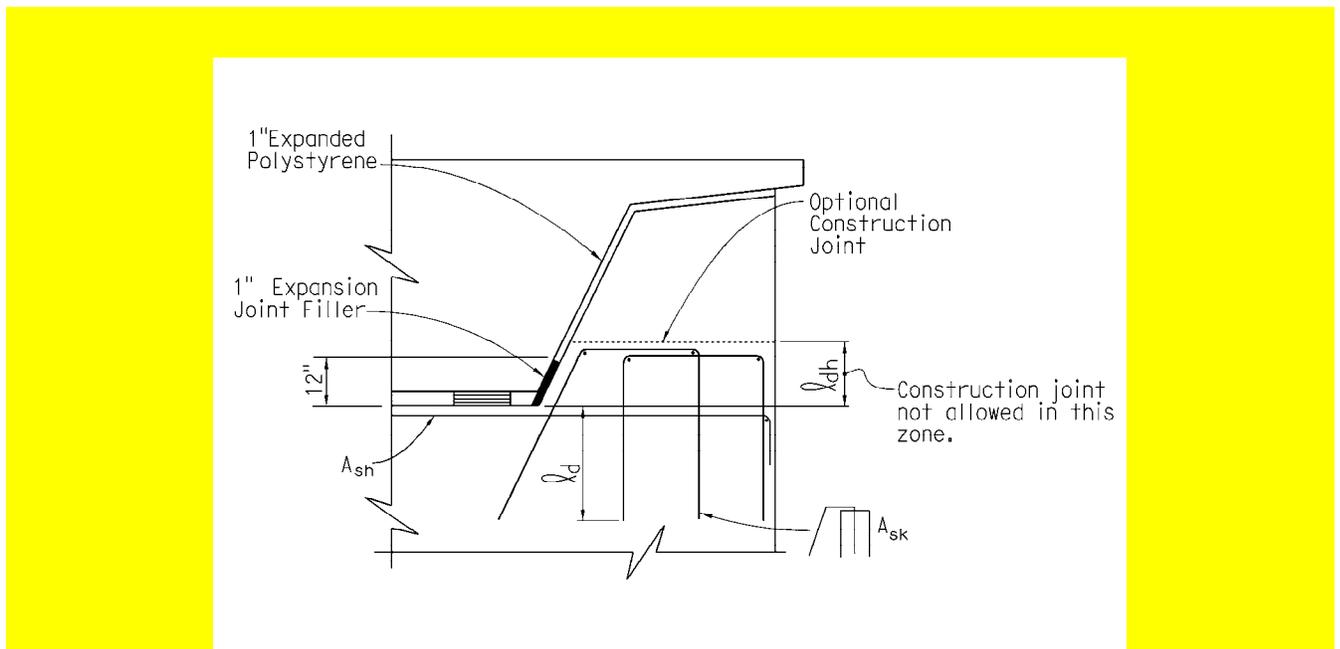
$$A_{sk} = \frac{1}{1.4 \times f_{ye}} (F_{sk} - 0.4 \times A_{cv}) \quad \text{Non-isolated shear key} \quad (7.49)$$

in which:



* Smooth construction joint is required at the shear key interfaces with the stemwall and backwall to effectively isolate the key except for specifically designed reinforcement. These interfaces should be trowel-finished smooth before application of a bond breaker such as construction paper. It is not recommended to use form oil as a bond breaker for this purpose.

(A) Isolated shear key



(B) Non-isolated shear key

NOTES:

- (a) Not all shear key bars shown
- (b) On high skews, use 2" expanded polystyrene with 1" expanded polystyrene over the 1" expansion joint filler to prevent binding on post-tensioned bridges.

Figure 7.16 Abutment Shear key Reinforcement Details

$$A_{cv} \geq \max \left\{ \begin{array}{l} \frac{4.0 \times F_{sk}}{f_{ce}'} \\ 0.67 \times F_{sk} \end{array} \right. \quad (7.50)$$

$$A_{sk,min} = \frac{0.05 \times A_{cv}}{f_{ye}} \quad (7.51)$$

where:

A_{cv} = Area of concrete considered to be engaged in interface shear transfer (in²)

$A_{sk,min}$ = Minimum area of interface shear reinforcement (in²)

In Equations 7.48 – 7.51, f_{ye} and f_{ce}' have units of ksi, F_{sk} is in kips, and A_{sk} is in in²

Due to development length requirements, it is recommended that vertical shear key reinforcement be no larger than #11 bars. If the height of the shear key is not adequate to develop straight bars, hooks or T-heads may be used.

The concrete shear key block should be well confined to ensure shear failure of the vertical key reinforcement instead of deterioration of the key block itself.

B) Horizontal Reinforcement in the Stem wall (Hanger bars)

The horizontal reinforcement in the stem wall below the shear key shall be designed to carry the shear key force elastically. The required area of horizontal reinforcement in the stem wall, A_{sh} is given by Equations 7.52 and 7.53 for Isolated and Non-isolated shear keys, respectively.

$$A_{sh} = 2.0 \times A_{sk(provided)}^{Iso} \quad \text{Isolated shear key} \quad (7.52)$$

$$A_{sh} = \max \left\{ \begin{array}{l} 2.0 \times A_{sk(provided)}^{Non-iso} \\ \frac{F_{sk}}{f_{ye}} \end{array} \right. \quad \text{Non-isolated shear key} \quad (7.53)$$

where:

$A_{sk(provided)}^{Iso}$ = Area of interface shear reinforcement provided in Equation 7.48 for Isolated shear key

$A_{sk(provided)}^{Non-iso}$ = Area of interface shear reinforcement provided in Equation 7.49 for non-isolated shear key

Horizontal stem wall tension reinforcement can be provided using headed bars or standard hooked hanger bars. “T” heads should be considered in place of large radius hooks.

In situations where limited space prevents placement of the required shear key reinforcement, the design engineer must use judgment. Such situations may occur due to non-standard overhangs, high skews, and retrofit conditions at widenings.

Wide bridges may require internal shear keys to ensure adequate lateral resistance is available for service load and moderate earthquakes. Internal shear keys should be avoided whenever possible because of maintenance problems associated with premature failure caused by binding due to **the** superstructure rotation or shortening.

8.2 Development of Longitudinal Column Reinforcement

Refer to ~~Chapter 8 in the Bridge Design Specifications~~ LRFD BDS for the development requirements for all reinforcement not addressed in this Section.

8.2.1 Minimum Development Length of Column Longitudinal Bars into Cap Beams Reinforcing Steel for Seismic Loads Considerations

Column longitudinal reinforcement shall be extended into footings and cap beams as close as practically possible to the opposite face of the footing or cap beam.

If the joint shear reinforcement prescribed in Section 7.4.4.2, and the minimum bar spacing requirements in BDS 8.21 Section 8.2.5 and AASHTO LRFD Articles 5.10.3.1 and 5.10.3.2 are met, the anchorage for longitudinal column bars developed into the cap beam for seismic loads shall not be less than the length specified in Equation 8.1[1]:

$$l_{ac} = 24d_{bt} \quad (\text{in, or mm}) \quad (8.1)$$

The anchorage length calculated specified in Equation 8.1 was based on test data on straight column longitudinal bars extended into the cap beam and therefore cannot be reduced by adding hooks or mechanical anchorage devices.

The reinforcing development requirements in other Caltrans documents must be met for all load cases other than seismic. Note that the minimum development length of column longitudinal bars into footings is governed by the reinforcing development provisions in other Caltrans documents.

The column reinforcement shall be confined along the development length l_{ac} by transverse hoops or spirals with the same volumetric ratio as required at the top of the column. If the joint region is not confined by solid adjacent solid members or prestressing, the volumetric ratio of the confinement along l_{ac} shall not be less than the value specified by Equation 8.2.

$$\rho_s = \frac{0.6 \times \rho_l \times D_c}{l_{ac}} \quad (8.2)$$

8.2.4 Development Length For Column Reinforcement Extended Into Enlarged Type II Shafts

Column longitudinal reinforcement shall be extended into Type II (enlarged) shafts in a staggered manner with the minimum recommended embedment lengths of $(D_{c,max} + l_d)$ and $(D_{c,max} + 2 \times l_d)$, where $D_{c,max}$ is the largest cross section dimension of the column, and l_d is the development length in tension of the column longitudinal bars. The development length l_d shall be determined by multiplying the basic tension development length l_{db} as specified in AASHTO LRFD Section 5.11.2.1 by the compounded modification factors of 0.9 and 0.6 for epoxy-coated and non epoxy-coated reinforcement, respectively. Nominal Expected values of $f_y = 68$ ksi and $f'_c = 5$ ksi for f_y and f'_c , respectively, shall be used in calculating l_{db} .

In addition to ensuring adequate anchorage beyond the plastic hinge penetration into the shaft, this provision will ensure that the embedment lengths for a majority of bridge columns supported on Type II shafts are less than 20 ft. Construction cost increases significantly when embedment lengths exceed 20 ft as the shaft excavations are governed by the more stringent Cal-OSHA requirements for tunneling and mining.

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