

CALTRANS
SEISMIC DESIGN
CRITERIA
VERSION 2.0



State of California
Department of Transportation

APRIL 2019

TABLE OF REVISIONS FROM SDC 1.7 TO SDC 2.0

SDC 1.7 Section	SDC 2.0 Section	Revision
All Sections	All Sections	Rearranged design provisions in a “Code and Commentary” format Included new commentaries and references Added Units to all empirical equations Deleted Equations in Metric units
1.	1.1	Changed scope of the SDC from “Ordinary Standard bridges” to “Ordinary Standard” and “Recovery Standard” bridges Made major editorial revisions
1.1	1.2.1, 1.2.2	Modified definition of an Ordinary Standard bridge, including classification of Pier walls and “Foundations in modified soil” as Nonstandard features
1.2		Deleted “Types of Components addressed in the SDC”
	1.2	Added “Bridge Categories”
	1.2.2	Added “Nonstandard Bridge Features”
	1.2.4	Added “Recovery Bridge”
	1.2.5	Added “Important Bridge”
1.3		Deleted “Bridge Systems”
	1.3	Added “Seismic Performance Criteria”
	1.4	Added “Design Philosophy” Added “Seismic Design Procedure Flowchart” (new Appendix A)
	1.5	Added “Procedure for Modifying the SDC”
	2.1	Added “Definitions”
Appendix A	2.2 and 2.3	Separated the listing of Notations and Acronyms/Initialisms Added more “Notations” and “Acronyms/Initialisms” Clarified existing Notations

SDC 1.7 Section	SDC 2.0 Section	Revision
	3.1	Added “Earthquake Resisting Elements”
	3.2.1.1	Added “Safety Evaluation Earthquake” to the definition of Design Spectrum Deleted “deterministic criterion” from the definition of Design Spectrum for Safety Evaluation Earthquake Updated the online design tools for specification of Design Spectrum
	3.2.1.2	Added definition of Design Spectrum for Functional Evaluation Earthquake
2.1.5	3.2.1.5	Modified the use of Damping Ratios (ratios other than 5% to require a PSDC)
6.1.2	3.2.2	Clarified Liquefaction hazard
6.1.4	3.2.4	Modified provisions for “Additional Seismic Hazards”
3.2.3	3.3.3	Added provisions for Grade 80 Steel (Properties and usage limitations)
3.2.7		Deleted “Other Material Properties”
5.6.1.1	3.4.2	Deleted Figure for “Effective Stiffness of Cracked Reinforced Sections” and replaced it with equations for the relevant range of axial load ratios
5.6.1.2	3.4.3	Changed “Effective Moment of Inertia for Box Girder Superstructures” to “Effective Moment of Inertia for Superstructures and Cap Beams” Added provision for “effective moment of inertia for prestressed sections”
5.6.2	3.4.4	Clarified provision for “Torsional Moment of Inertia for Superstructures”
4.1.1	3.5.1 and 3.5.2	Added “Global Displacement Criteria” for Recovery bridges Clarified definitions of “frame/bent displacement capacity” and “frame/bent displacement demand,” and extended the definitions to cover abutments and in-span hinges Defined “Local Principal/Critical Axes” Made minor modification to Figure on “Global Force Deflection Relationship”
3.2.1	3.6	Added “Load Factor” Changed Resistance factor for shear from 0.9 to 1.0

SDC 1.7 Section	SDC 2.0 Section	Revision
4.5	4.2	Added “Nonlinear Time History Analysis” as a standard analysis method Added “Equal Displacement Approximation” language for ESA and EDA Added table: “Applicability of Methods for Displacement Demand Analysis”
4.5.1	4.2.1	Provided equation for displacement demand using ESA Added provision for determining the displacement demand of single span bridges
2.1.2.1 and 5.2.2	4.2.2	Changed “Tension” and “Compression” models to “Individual frame” and “Continuous global models” Deleted Figure on “Global Axis Definition” Deleted Method I (100/30 % rule) for application and combination of horizontal ground motion Made major editorial revisions
	4.2.3	Added provisions for “Nonlinear Time History Analysis”
	4.3.1	Added new provisions on abutment stiffness for Global models
5.5		Deleted “Simplified Analysis”
5.4	4.3.2	Clarified “Stand-Alone Models”
5.4.1 and 5.4.2	4.3.2.1 and 4.3.2.2	Modified Figures: Transverse and Longitudinal Stand-Alone Models Changed “abutment stiffness” provision for Stand-Alone longitudinal model
2.2.3 and 2.2.4	4.4.1	Clarified terms used in definition of displacement ductility demand Added limiting values for displacement ductility demand for SCMs other than columns Deleted displacement ductility demand values for pier walls
	4.4.3.1 and 4.4.3.2	Added equations for Superstructure/Bent cap “Seismic Moment and Shear Demands”
4.2	4.4.4	Changed the factor for P-Delta equation from 0.20 to 0.25
5.2.3	5.2.2	Clarified definition and application of Inelastic Static Analysis Added provision for determining the displacement capacity of single span bridges Modified the applicability of equations for Local Displacement Capacity Editorial correction to Figure titled “Local Ductility Assessment”
3.1.1	5.3.1	Streamlined definition of seismic critical members (SCMs) Removed “Pier walls” and “Type II shafts in soft/liqeuifiable soil” as SCMs

SDC 1.7 Section	SDC 2.0 Section	Revision
	5.3.3	Added provision for “Axial Load Limits”
	5.3.5	Added “Confined Core”
3.5	5.3.6.1	Changed title from “Minimum Lateral Strength” to “Minimum Flexural Capacity” Clarified provision for “Minimum Flexural Capacity”
3.3.1	5.3.6.2	Added provision for Moment Curvature analysis of multi-column bents in transverse direction to include overturning effects
3.6.3	5.3.7.3	Deleted equation for “Nominal Shear Reinforcement Capacity” of Pier walls
3.8.2	5.3.8	Section title changed from “Lateral Reinforcement...” to “Transverse Reinforcement...”
	5.3.8.1	Added design provisions for transverse reinforcement
	5.3.8.2	Added new provision for “Minimum volume of transverse reinforcement” to replace the requirement for “Minimum local displacement ductility capacity”
3.7.2	5.3.9.2	Deleted equation for “Minimum Longitudinal Reinforcement” of Pier walls
3.6.5.3	5.3.9.3	Changed “maximum spacing of longitudinal bars in interlocking portion of SCMs” from 8 to 12 inches Corrected dimension for Figure titled “Vertical Reinforcement within Interlocking Hoops”
3.4	5.4.1	Updated the list of Capacity Protected members (CPMs)
4.3.2	5.4.4	Added equations for Superstructure/Bent cap Seismic Capacity Deleted moment equilibrium equations for Superstructure and Bent Cap Seismic Capacity
3.8.6	5.4.5	Modified the provision for transverse reinforcement for Type II shafts
6.2.2	6.1.1	Adopted a new “Soil Classification” “Competent” soil now classified as “Class S1” soil All non-Competent soils (Marginal, Poor, Soft, potentially liquefiable, and soil susceptible to lateral spreading) now classified as “Class S2”
6.2.2.1	6.1.2	Modified the characteristics of Class S1 (previously “Competent”) soil
6.2.2.3		Deleted “Marginal Soil”

SDC 1.7 Section	SDC 2.0 Section	Revision
6.2.1		Deleted “Foundation Performance”
6.2.3		Deleted “Foundation Design Criteria”
	6.2.2.1	Added provisions for flexure and shear design of footings
7.7.1.6		Deleted “Effect of Large Capacity Piles on Footing Design”
7.7.1.7	6.2.2.5	Streamlined provisions for “Footing Stirrups” Modified annotations and added a Note to the Figure captioned: Footing Reinforcement – Fixed Column
	6.2.2.6	Added provisions for “Spread Footings”
7.7.1.1	6.2.3.1	Modified the assumptions for design of pile foundations in class S1 soils
	6.2.3.2	Added provision for shear resistance of piles in class S1 soil
	6.2.4.1	Added provision for design of pile foundations in class S2 soil
7.7.1.2A		Deleted the provision allowing the simplified model for Competent soil to be used for design of pile foundation in non-soft/liquefiable marginal soil
	6.2.4.2	Added “Comprehensive Foundation Design” for foundations in class S2 soils (All foundations in class S2 soil to require a comprehensive design)
7.7.1.2B		Deleted “Lateral Capacity of Fixed Head Piles”
7.7.1.2C		Deleted “Passive Earth Resistance for Pile Caps in Marginal Soil”
2.2.5	6.2.4.3	Modified the provisions for “Scour and Liquefaction Considerations”
2.2.4	6.2.5.1	Major editorial revision to Types I and II shafts, including addition of new provisions Modified figure on “Shaft Definitions” and changed title to “Shaft Types”
	6.2.6	Added “Lateral Stability of Piles and Shafts”
7.7.3.3		Deleted “Shaft Diameter”
7.7.3.4		Deleted “Minimum Shaft Length”
7.7.3.1	6.2.5.2	Modified provisions for design of Type I shafts
7.7.3.2	6.2.5.3	Modified provisions for design of Type II shafts

SDC 1.7 Section	SDC 2.0 Section	Revision
7.8.1	6.3.1.2	Made a major revision to “Bilinear Model” for abutment longitudinal stiffness, including incorporation of a skew reduction factor
7.8.1	6.3.1.3	Added an equation for “effective abutment longitudinal stiffness adjusted for a displacement coefficient between 2 and 4”
7.8.3	6.3.3	Modified the Equation and Figure for abutment support length Changed terminology: “Abutment Seat Width” to “Abutment Support Length”
7.8.4	6.3.4	Modified Equations for abutment shear key force demand
7.8.4.1	6.3.5	Added a new provision for minimum development length of Headed bars used for shear key reinforcement
	6.3.5.1	Added a new provision for Isolated Shear Key confinement reinforcement
7.1.1	7.1.2	Deleted “Balanced Stiffness” equations for Constant Width frames Modified definition of terms used for “Balanced Stiffness” Rearranged “Balanced Stiffness” equations
7.1.2	7.1.3	Rearranged “Balanced Frame Geometry” equation Modified definition of terms used for “Balanced Frame Geometry”
7.1.4		Deleted “End Span Consideration”
7.2.1.1	7.2.1.1	Modified “Effective Superstructure Width” Figures A and B
7.2.3	7.2.1.2	Clarified and added new provisions for “Precast Girders”
7.2.2	7.2.2	Clarified and revised provisions for “Vertical Acceleration”
7.2.5.4	7.2.3.1 and 7.2.3.2	Modified the Equation and Figure for In-span hinge support length Changed terminology from “Seat Width” to “Support Length” Added new design provisions and made editorial clarifications
7.2.5.3	7.2.3.3	Clarified and modified provisions for “Shear Key Requirements in the Transverse Direction”
7.2.7	7.2.3.5	Modified provisions for “Pipe Seat Extenders”
	7.2.4	Added “Superstructure Depth”
7.2.8		Deleted “Equalizing Bolts”
7.3.1	7.3.1	Modified “Effective Width of Integral Bent Caps”

SDC 1.7 Section	SDC 2.0 Section	Revision
7.4.1	7.4.1	Clarified provision for “Joint Performance”
7.4.5.1 and 7.4.5.1	7.4.5.2 and 7.4.5.3	Clarified provisions for “T” and Knee Joint reinforcement Modified figures for “Knee Joint Shear Reinforcement”
7.5	7.5.1	Added provision requiring PTFE spherical bearings to be used at all in-span hinges
	7.5.2	Added provisions for “Seismic Expansion Joints”
	7.6.1	Added new provisions for Columns (core geometry, framing, etc.)
	7.6.2	Added provision for Column Dimensions for superstructures with drop cap
7.6.5	7.6.3	Clarified provisions for Column Flares, including addition of Figures for Horizontal and Vertical Flare Isolation
7.6.6		Deleted provisions for “Pier Wall”
7.6.7	7.6.4	Clarified equations for “Column Key Design” Added provision and commentary to account for moment generated by shear key steel
	8.2.1	Added provisions on types and uses of “Reinforcing Bar Splices”
8.1.1	8.2.2.1	Clarified provisions for “No-Splice Zones” in SCMs
8.1.4	8.2.2.3	Changed caption to “Hoop Splices and Spiral Terminations” Clarified provisions for “Hoop and Spiral Reinforcement” in SCMs Added provisions for spiral reinforcement terminations
	8.2.3.2	Added provisions for transverse reinforcement in Capacity Protected Members
8.2.1	8.3.1.1	Modified the provision for “Minimum Development Length of Longitudinal Reinforcement”
8.2.1	8.3.1.2	Clarified provision for development length of “Epoxy-coated Longitudinal Bars” in SCMs
8.2.2	8.3.1.3	Clarified provision for development length of “Bundled Bars” in SCMs
8.2.3.1		Deleted “Maximum Bar Diameter”

SDC 1.7 Section	SDC 2.0 Section	Revision
8.2.4	8.3.2	Modified the caption and provisions for “Embedment Length for Column Reinforcement Extended into Type II Shafts”
	8.3.3	Added “Reinforcing Bar Hooks”
8.2.5	8.4.1.1	Modified the provision for maximum spacing of transverse reinforcement inside the plastic hinge region
	8.4.1.2	Added provisions for maximum spacing of transverse reinforcement “Outside the Plastic Hinge Region”
	8.4.2	Added provisions for “Maximum and Minimum Lateral Spacing of Longitudinal Reinforcement”
7.2.4	9 (9.1 through 9.8)	Added a new Section dealing with specific design provisions for “Slab Bridges”
	Appendix A	Added a new appendix: “Seismic Design Procedure Flowchart”
Appendix B	Appendix B	Updated the web links and References for “Design Spectrum Development” Deleted information on “Deterministic criteria” Deleted graphs for preliminary Design Spectrum for Soil Profile Types B, C, D
	Index	Added an Index

CHANGES IN SECTION NUMBERING FROM SDC 1.7 TO SDC 2.0

Version 1.7	Version 2.0	Version 1.7	Version 2.0
1.	1.1	3.3.1	5.3.6.2
1.1	1.2.1	3.4	5.4.1
1.2	Deleted	3.5	5.3.6.1
1.3	Deleted	3.6.1	5.3.7.1
1.4	2.1, 3.5.1, 4.3.1, C5.2.2	3.6.2	5.3.7.2
2.1	3.2.1.1	3.6.3	5.3.7.3
2.1.1	C3.2.1.1	3.6.5.1	5.3.7.4
2.1.2	3.2.1.3	3.6.5.2	5.3.7.5
2.1.2.1	4.2.2	3.6.5.3	5.3.9.3
2.1.2.2	4.2.1	3.6.6.1	Deleted
2.1.3	3.2.1.4	3.6.6.2	Deleted
2.1.4	Deleted	3.6.7	5.4.2, 5.4.3
2.1.5	3.2.1.5	3.7.1	5.3.9.1
2.2.1	4.2	3.7.2	5.3.9.2
2.2.2	4.3.1, 3.5, C5.2.2	3.7.3	Deleted
2.2.3	4.4.1	3.8.1	5.3.8.2
2.2.4	4.4.1, 6.2.5.1	3.8.2	5.3.8.1
2.2.5	6.2.4.3	3.8.3	5.3.8.3
2.3	2.1, C1.3.1	3.8.4	Deleted
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7.7.1.7	6.2.2.5
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SECTION 1

INTRODUCTION

1.1 SCOPE

The California Department of Transportation (Caltrans) *Seismic Design Criteria*, hereinafter referred to as the *SDC*, specifies the minimum seismic design requirements for newly designed “Standard” concrete bridges. Nonstandard bridges shall require Project Specific Seismic Design criteria (PSDC) in addition to the *SDC*. A Seismic Safety Peer Review team shall be established for prominent or unusually complex bridges requiring a PSDC.

For seismic design of steel bridges, the *Caltrans Seismic Design Specifications for Steel Bridges* (Caltrans, 2016) shall apply.

C1.1

The *SDC* is a compilation of seismic design criteria documented in various Division of Engineering Services (DES) publications. The goal is to update all the DES seismic design publications on a periodic basis to reflect the current state of practice for seismic bridge design.

The *SDC* is intended for use on new bridges designed for the California State Highway System. New bridges may be constructed by Cast-in-Place (CIP), or Precast (PC) with or without Accelerated Bridge Construction (ABC). Designers should refer to DES manuals for seismic design guidance and standards not explicitly addressed by the *SDC*.

Standard bridges may be either “Ordinary” or “Recovery” as specified in Sections 1.2 and 1.2.1.

For the procedure to establish Project Specific Seismic Design Criteria (PSDC), refer to MTD 20-11. For information on Seismic Safety Peer Review, refer to MTD 20-16.

The term “shall” denotes a requirement for compliance with these Criteria.

The term “should” indicates a strong preference for a given criterion.

The term “may” indicates a criterion that is usable, but other local and suitably documented, verified, and approved criterion may also be used in a manner consistent with *SDC* design philosophy.

The term “recommended” is used to give guidance based on past experiences.

Throughout this document, the term “AASHTO-CA BDS” represents the current California approved AASHTO LRFD Bridge Design Specifications, and the corresponding California Amendments to AASHTO LRFD Bridge Design Specifications.

1.2 BRIDGE CATEGORIES

C1.2

All bridges shall be categorized as:

- Ordinary,
- Recovery, or
- Important.

Depending on their bridge features, Ordinary and Recovery bridges shall be classified as either “Standard” or “Nonstandard,” as follows:

- Ordinary Standard bridge,
- Ordinary Nonstandard bridge,
- Recovery Standard bridge, or
- Recovery Nonstandard bridge

The main difference between an Ordinary Standard bridge and a Recovery Standard bridge is the performance criteria (see Section 1.3).

1.2.1 Standard Bridge Features

C1.2.1

Standard bridges shall have the following features:

- Each span length less than 300 feet,
- Either box girder, slab on girder, or slab superstructures,
- Either solid column-type or pile-bent substructures,
- Fundamental period of the bridge system is greater than or equal to 0.7 seconds in the transverse and longitudinal directions of the bridge.
- Bridge site is more than 300 feet away from a fault

The bulleted listing are the basic features of Standard (i.e., Ordinary Standard and Recovery Standard) bridges. Standard bridges possess other features which are common to all bridge types.

Bridges located within 300 feet of active faults (per the Alquist-Priolo Earthquake Fault Zoning Act) are susceptible to unusually large deformation demands due to near fault effects and are analyzed and designed for fault-crossing hazard as established through a PSDC (See also MTD 20-8).

1.2.2 Nonstandard Bridge Features

C1.2.2

Bridges that do not satisfy any one of the Standard bridge features specified in Section 1.2.1 shall be classified as Nonstandard.

Nonstandard bridge features may lead to bridges with complex or non-ductile response during earthquakes. Nonstandard bridges may require a more detailed analysis than that described in the *SDC* in order to capture their complex response.

Examples of Nonstandard bridge features are:

Irregular Geometry

- Multiple superstructure levels
- Bifurcating superstructures
- Skew angles (greater than 20°) that vary from support to support, excluding curved bridges with circular or square columns
- Hollow columns

Unusual Framing

- Outrigger and C bent supports
- Unsymmetric layout of columns within a bent
- Unbalanced mass and/or stiffness distribution
- Multiple superstructure types
- Pier walls
- Isolation bearings and dampers
- Single column bents supported on spread footings

Unusually complex bridges

- Cable-stayed
- Suspension
- Arch
- Truss

Uncommon bridge Substructure (Foundation)

- Micropiles
- Hollow concrete piles
- Other nonstandard piles such as timber, tapered, and composite piles

Unusual Soil/Hazard

- Class S2 soil that has been modified (See Section 6.1.3 for definition of class S2 soil)
- Tsunami
- Lateral spreading induced by liquefaction

Pier walls are nonstandard because adequate ductility cannot be achieved in the wall's strong direction. When the Hydraulics Engineer recommends the use of pier walls such as in waterway crossings per MTD 16-1, sacrificial guide walls are recommended in combination with ductile columns.

1.2.3 Important Bridges

Important bridges shall satisfy one or more of the following:

- (a) Expected to provide immediate access to emergency and similar life-safety facilities after an earthquake
- (b) Time to restore functionality after closure would create a major economic impact
- (c) Formally designated as critical by a local emergency management plan

Important bridges shall be designated by the sponsoring district or local agency, in consultation with Caltrans Division of Engineering Services.

1.2.4 Recovery Bridges

Recovery bridges shall serve as vital links for rebuilding damaged areas and provide access to the public shortly after an earthquake.

Recovery bridges shall be designated by the sponsoring district or local agency, in consultation with Caltrans Division of Engineering Services.

1.2.5 Ordinary Bridges

All bridges not designated as either *Important* or *Recovery* shall be designated as *Ordinary*.

1.3 SEISMIC PERFORMANCE CRITERIA

The seismic hazard evaluation level for designing Ordinary bridges shall be the Safety Evaluation Earthquake (SEE) as specified in Table 1.3-1 and Section 3.2.1.1.

The seismic hazard evaluation level for designing Recovery bridges shall be both the Safety Evaluation Earthquake as specified in Table 1.3-1 and Section 3.2.1.1 and the Functional Evaluation Earthquake (FEE) as specified in Table 1.3-1 and Section 3.2.1.2.

Pier walls may be needed in bridge retrofits and/or widenings to match existing bridge configuration and ensure predictable seismic performance.

C1.2.3

Designating a bridge as *Important* can have a significant implication with respect to cost. By 2017, Caltrans had designated only two new bridges as Important, namely: the New Benicia-Martinez Bridge and the New East Span of San Francisco-Oakland Bay Bridge.

C1.2.4

Recovery bridges serve a critical role in the economic recovery of an area impacted by a damaging earthquake. They are designed for enhanced performance above Ordinary bridges but less than the performance of Important bridges.

C1.3

The expected post-earthquake damage states and service levels corresponding to the seismic hazard evaluation levels are also shown in Table 1.3-1.

The expected post-earthquake damage states and service levels of Important bridges are included in Table 1.3-1 to provide an indication of their expected performance relative to other bridge categories. It should be noted that the design spectra for Safety and Functional Evaluation

Table 1.3-1: Seismic Hazard Evaluation Levels and Expected Performance

BRIDGE CATEGORY	SEISMIC HAZARD EVALUATION LEVEL	EXPECTED POST EARTHQUAKE DAMAGE STATE	EXPECTED POST EARTHQUAKE SERVICE LEVEL
Ordinary	SEE	Major	No Service
Recovery	SEE	Moderate	Limited Service-weeks
	FEE	Minimal	Full Service
Important	SEE	Minimal to Moderate	Limited Service-days
	FEE	None to Minimal	Full Service

Ordinary and Recovery bridges subjected to the seismic hazard levels specified in Table 1.3-1 shall satisfy the displacement criteria specified in Section 3.5.1 and the displacement ductility demand, μ_D specified in Table 4.4.1-1.

Earthquakes for an Important bridge are defined by a PSDC and are typically greater than those for Ordinary and Recovery bridges.

Expected Post-earthquake Damage States

- Major – Bridge replacement is likely. All plastic hinges within the structure have formed with ductility demand values approaching the limits specified in Table 4.4.1-1. Bridges near the epicenter of the 1000-year event are expected to experience major damage but bridges farther away are expected to experience moderate or minimal damage.
- Moderate – Bridge repair is likely but bridge replacement is unlikely
- Minimal – Essentially elastic performance
- None – No damage

Major, Moderate, and Minimal damage states may be correlated to the following damage states proposed by Vosooghi and Saiidi (2010):

- Major damage: Damage State 5 (DS-5) - “imminent failure,” i.e., onset of compressive failure of core concrete
- Moderate damage: DS-3 - “extensive cracks and spalling” and DS-4 - “visible lateral and/or longitudinal reinforcing bars”
- Minimal damage: DS-1 - “flexural cracks” and DS-2 - “minor spalling and possible shear cracks”

Expected Post-earthquake Service Levels

- No Service – Bridge is closed for repair or replacement.
- Limited Service – weeks:
Bridge is open for emergency vehicle traffic; A reduced number of lanes for normal traffic is available within ten weeks of the earthquake; Vehicle weight restriction may be imposed until repairs are completed. It is expected that within ten weeks of the earthquake, repair works on a damaged bridge would have reached the stage that would permit normal traffic on at least some portion of the bridge.
- Limited Service – days:
Bridge is open for emergency vehicle traffic; A reduced number of lanes for normal traffic is available within a couple of days of the earthquake; Vehicle weight restriction may be imposed until repairs are completed. It is expected that a bridge inspection would have been completed within a couple of days of the earthquake.
- Full Service – Full access to normal traffic is available almost immediately after the earthquake.

1.4 DESIGN PHILOSOPHY

Ordinary and Recovery bridges shall be designed based on “strong beam - weak column” proportioning principle. Plastic hinging shall be directed to Seismic Critical Members (SCMs), allowing a mechanism to form and facilitates transverse and longitudinal movement of bridge bents and frames. The plastic hinge ductility or other means of energy dissipation/bridge damping shall be adequate to satisfy the deformation demands imposed by the Design Seismic Hazards (DSH).

When the design seismic hazards for life safety (i.e., the safety evaluation earthquake) occur, ordinary bridges may experience major damage requiring bridge replacement but shall have a minimal probability of collapse.

When the design seismic hazards for life safety occur, recovery bridges may experience moderate damage requiring bridge repair but shall have a minimal probability of bridge replacement.

When subjected to the design seismic hazard that has a significant chance of occurring during the life of the bridge (i.e., the functional evaluation earthquake), recovery bridges shall respond in an essentially elastic manner with minimal bridge damage.

Every bridge shall be designed with an Earthquake Resisting System that ensures a load path for gravity loads and provides sufficient strength and ductility to achieve the performance criteria specified in Section 1.3.

Except as specified below, Earthquake Resisting Systems shall consist of the following:

- **Seismic critical members (SCMs)** – ductile structural members that are intentionally designed to deform inelastically through several cycles without significant loss of strength, thereby limiting the forces transmitted to adjoining capacity protected members,

C1.4

The “strong beam - weak column” proportioning principle, otherwise known as capacity design, entails that certain bridge members (generically represented by seismic critical members) are intentionally designed for energy dissipation/damping while other members (the capacity protected members) are intentionally designed to remain essentially elastic when the design seismic hazards (DSH) occur. See Sections 2.1 and 3.2 for definition and specification of DSH.

In general, energy dissipation may be achieved by the formation of flexural plastic hinges in seismic critical members, failure of sacrificial elements such as shear keys, abutment backwalls, conventional expansion joints, link beams, and elastomeric bearings, the mobilization of soil behind abutment backwalls and around foundation elements, and the use of isolation devices and dampers. Although sacrificial elements are prone to being damaged during design level earthquakes, they provide longitudinal and transverse resistance for service loads and small to moderate earthquakes, and their failure is associated with energy dissipation and a consequent limitation of the forces transmitted to bridge substructures. Additional provisions and commentary on sacrificial elements are given in Sections 6.3.4, C6.3.4, 7.5.1 and C7.5.1.

In order to ensure failure by inelastic flexural yielding while minimizing the chance of non-ductile failure modes, SCMs are designed for high ductility by providing the potential plastic hinge locations with enhanced lateral confinement which acts to maintain the integrity of the confined core of the SCM enabling it to sustain high strain levels as well as prevent premature buckling of the longitudinal bars.

Caltrans takes advantage of ductility and post elastic strength to meet the performance criteria specified in Section 1.3 with a minimum of capital investment. This philosophy is based on achieving a balance between the acceptable risk level and the

- **Capacity protected members (CPMs)** - structural members that remain essentially elastic after the adjoining members fuse or form plastic hinges,
- **Earthquake resisting elements (EREs)** – bridge elements or portions of bridge members that undergo inelastic deformation, dissipate energy, or increase bridge damping, and
- **Sacrificial elements** – Bridge elements that are typically designed to disengage in order to limit forces transmitted to adjoining capacity-protected members.

The earthquake resisting system for single span bridges may be achieved without seismic critical members.

Live load shall not be applied simultaneously with seismic loading.

initial cost of a bridge.

Force-based design of SCMs using Design Spectrum (DS) forces is considered an exception to *SDC* design philosophy.

Significant inelastic response in concrete superstructures is not desirable because of the potential to jeopardize public safety and the difficulty in repairing such damage, especially for continuous bridges.

Seismic critical members and capacity protected members are listed in Sections 5.3.1 and 5.4.1, respectively. Additional provisions for earthquake resisting elements are given in Section 3.1.

Bridge accessories such as soundwalls, lighting poles, and sign structures are not considered a part of the earthquake resisting system.

Experimental and analytical investigations (Wibowo et al., 2013) have shown that seismic response is minimally affected by live load.

A seismic design flowchart intended to serve as a procedural guide for the engineer using the *SDC* seismic design philosophy to design a Standard Bridge is shown in Appendix C. The list of processes in the flowchart is only typical and is not intended to be an exhaustive listing of the processes required by the *SDC* for the seismic design of every Standard bridge.

1.5 PROCEDURE FOR MODIFYING THE *SDC*

Modification to the *SDC* shall include addition of new *SDC* provisions, removal of existing *SDC* provisions or revision of existing *SDC* provisions. The steps to modify the *SDC* shall be as summarized in Figure 1.5-1.

C1.5

In Figure 1.5-1, the abbreviations represent the following Caltrans entities:

EQC = Earthquake Committee,

TSC = Technical Steering Committee,

OEEAR = Office of Earthquake Engineering,
Analysis and Research,

SQM = Structure Quality Management,

SC = Structure Construction, and

SBE = State Bridge Engineer.

Additional information on the procedure for modifying the *SDC* can be found in MTD 20-11.

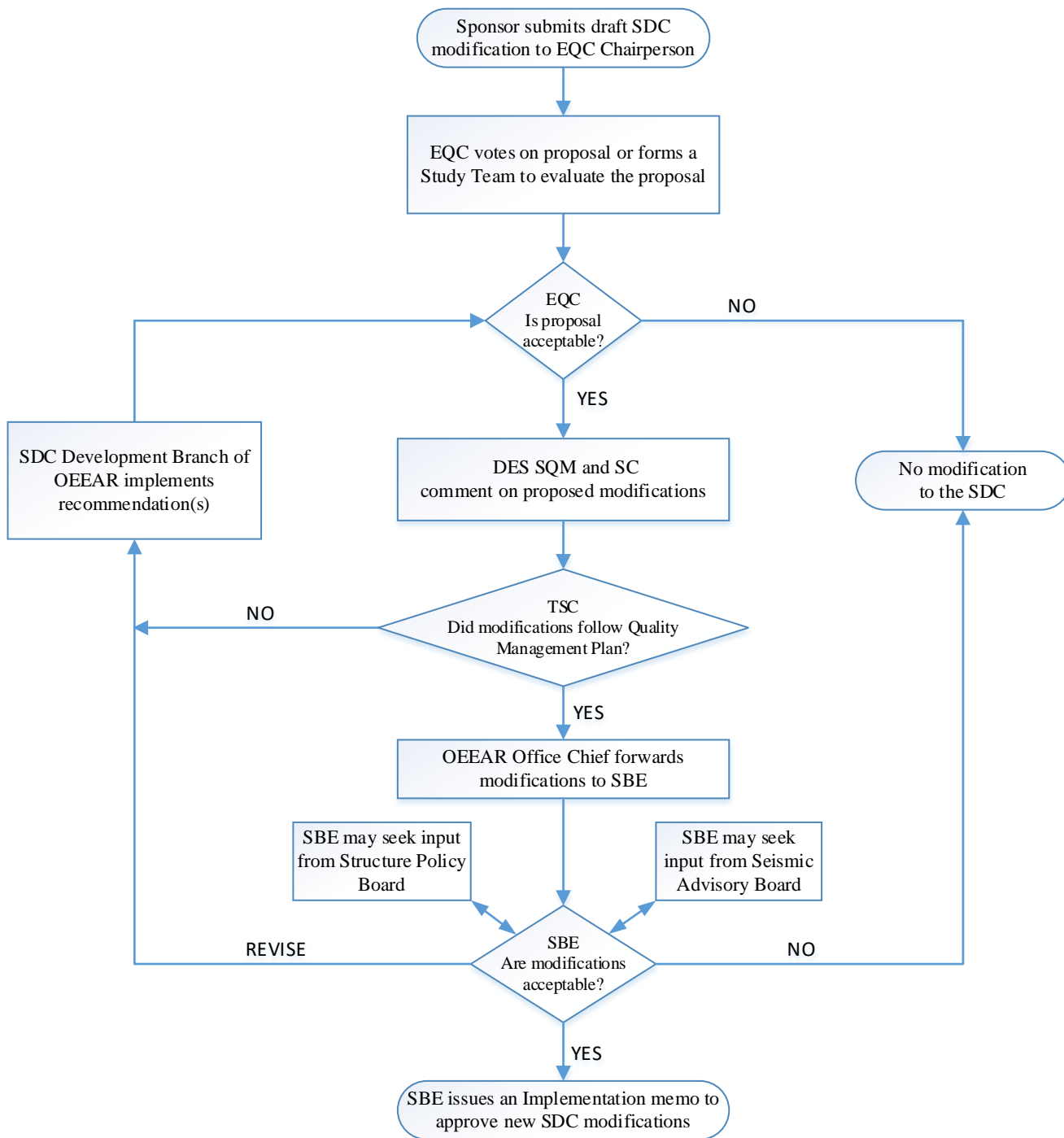


Figure 1.5-1 Flowchart for Modifying the SDC



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SECTION 2



DEFINITIONS, NOTATIONS, AND ACRONYMS/INITIALISMS

2.1 DEFINITIONS

Bent – A bridge component consisting of cap beam, columns/pile extensions, shafts, footings and/or pile group. Some bridges may be analyzed in the transverse direction using the bent model with its tributary bridge mass.

Bridge Accessories - Appurtenant equipment and components attached to the bridge structure such as sign structures, crash cushions, hand railings, soundwalls, and lighting poles.

Bridge System – A structural system consisting of superstructure, substructure, foundations and abutments framed into a single or multiple frame structural system. An example of a bridge system is a slab bridge composed of a slab superstructure supported by multi-column bents on pile/shaft extension foundations and abutments.

Capacity Design – A design philosophy in which certain bridge components are intentionally designed for energy dissipation/damping while adjoining components are intentionally designed to remain essentially elastic. Energy absorption may occur by yielding and/or disengaging.

Capacity Protected Member – A structural member within a bridge system that remains essentially elastic after the adjoining members fuse or form plastic hinges.

Confined Concrete - Concrete in a structural member which is confined by a combination of two or more of the following types of reinforcement: longitudinal reinforcement, closely spaced butt-spliced hoops, cross-ties with seismic hooks, and closely spaced continuous spirals.

Confinement – The use of reinforcing, steel or composite fiber wrapping, or similar devices to produce lateral and/or circumferential pressures that prevent disintegration of concrete in a structural member subjected to deformation.

Critical Pile Length - The embedded length of pile for which greater lengths do not result in a reduction of 5% or more in the deflection at the pile/shaft cut-off elevation in a lateral stability analysis.

Curvature – A measure of flexural deformation defined as the reciprocal of the radius of curvature of the neutral axis of a member in bending. Curvature is mathematically calculated as the ratio of the extreme fiber strain to the distance from the neutral axis to the extreme fiber.

Design Seismic Hazards - The collection of seismic hazards at a bridge site used in the design of bridges. Such hazards consist of ground shaking (i.e. Design Spectrum), surface fault rupture, liquefaction, lateral spreading, and tsunamis.

Design Spectrum – The acceleration response spectrum used in design to characterize the ground shaking hazard at a bridge site. Also, see definition of Response Spectrum.

Ductile Member – A structural member that can deform inelastically for several cycles without significant degradation of strength under the demands generated by the design seismic hazards.

Earthquake-Resisting Elements – bridge elements that undergo inelastic deformation, dissipate energy, or increase bridge damping under the deformation demands imposed by the design seismic hazards.

Earthquake-Resisting System – A system that provides sufficient strength and ductility for the bridge, ensures a load path for gravity loads, and controls seismic displacements.

Energy Dissipation Devices – Energy-dissipating elements introduced between bridge components for the purpose of increasing the bridge damping, resulting in a decrease in superstructure displacement and hence a decrease in the ductility demand on the supporting seismic critical members.

Equivalent Plastic Hinge Length - The equivalent length of a plastic-hinging member over which the plastic curvature is assumed constant for estimating plastic rotation.

Essentially Elastic – State of deformation of a bridge member limited to minor cracking or spalling of cover concrete.

Expected Material Properties – The most probable material properties (as opposed to specified minimum material properties) used to provide a more realistic estimate of member design strength and deformation capacity.

Functional Evaluation Earthquake (FEE) – A seismic hazard design Spectrum that has a relatively small magnitude but may occur several times during the life of the bridge. FEE is taken as a spectrum based on a 20% probability of exceedance in 50 years (or 225-year return period).

Global Behavior – The response of a bridge system including the effects of adjacent components, subsystems, and/or boundary conditions. For example, the behavior of an entire bridge system.

Ground Shaking – Seismic hazard measured by ground motion time histories or response spectrum.

Joint Shear – Vertical and horizontal shear forces developed in moment-resisting beam-to-column or footing-to-column joints subjected to seismic deformation.

Liquefaction – Loss of shear strength in loose, cohesionless soil resulting from build-up of pore water pressure as the soil tries to consolidate when exposed to seismic vibrations.

Local Behavior – The response of an individual component or subsystem independent of the effects of adjacent components, subsystems, or boundary conditions. For example, the behavior of an individual bent or column.

No-Splice Zone – Region(s) of a seismic-critical member identified on bridge plans, where splicing of the main flexural reinforcement is prohibited.

Overstrength Moment – The expected maximum moment that a seismic critical member can develop, taken as 120 % of the idealized plastic moment capacity of the seismic critical member. Adjacent capacity protected members are designed to remain essentially elastic when the adjoining seismic critical member reaches its overstrength moment.

Overstrength Shear – Shear force associated with the overstrength moment, taken as 120 % of the plastic shear demand of the seismic critical member. Adjacent capacity protected members are designed to remain essentially elastic when the adjoining seismic critical member reaches its overstrength shear.

Plastic Hinge – The region of a structural member that undergoes flexural yielding and plastic rotation while retaining flexural strength.

Plastic Hinge Region – Region of a structural member expected to form a plastic hinge and requiring special detailing in the form of enhanced confinement.

Prestress Secondary Moments and Shears - Moments and shears induced by support restraints in a statically indeterminate prestressed frame.

Project Specific Seismic Design Criteria (PSDC) – Seismic design criteria for bridges with nonstandard features as defined by the SDC. A PSDC consists of the SDC and the specific criteria developed by the PSDC team for the nonstandard feature(s).

Pushover Analysis – An inelastic static analysis procedure used to evaluate the displacement capacity of a structure or frame.

Response Spectrum - A plot showing peak responses (acceleration, velocity, or displacement) of bridges idealized as single degree of freedom oscillators versus the periods of the bridges.

Sacrificial Elements – Bridge elements that are typically designed to disengage in order to limit forces in adjoining capacity-protected members. Sacrificial elements include shear keys, backwalls, joints, and bearings.

Safety Evaluation Earthquake (SEE) - A seismic hazard design spectrum that has a small probability of occurring during the life of the bridge. SEE is taken as a spectrum based on a 5% probability of exceedance in 50 years (or 975-year return period). This design spectrum is practically equivalent to that based on a 7% probability of exceedance in 75 years life of a bridge.

Seismic Critical Member – A ductile structural member intentionally designed to deform inelastically through several cycles without significant degradation of strength, thereby limiting the forces transmitted to adjoining capacity protected members.

Seismic Isolation – A design approach in which a bearing device with an energy dissipation characteristic is used to reduce seismic design forces by increasing the period of a relatively stiff bridge.

Specified Material Properties – Nominal or specified minimum material properties as provided by design codes or ASTM. Corresponding values of expected material properties are obtained by multiplying the nominal values by a factor greater than 1.0.

Standard Bridge – An Ordinary or Recovery bridge possessing all the standard bridge features listed in Section 1.2.1 but not any of the nonstandard bridge features.

Type I Shaft – A drilled shaft foundation having the same confined core diameter as that of the supported column but may have the same or different concrete cover and area of transverse and longitudinal reinforcement as the supported column.

Type II Shaft - A drilled shaft foundation that is at least 24 inches larger than the maximum dimension of the supported column and has a reinforcing cage diameter larger than that of the supported column.

Ultimate Curvature Capacity - The curvature at which the concrete reaches its ultimate compression strain or the longitudinal reinforcing steel reaches its reduced ultimate tensile strain.

Unconfined Concrete – Concrete without special confinement detailing as defined for Confined concrete.

Volumetric ratio of transverse reinforcement – the ratio of volume of transverse reinforcement to the core volume confined by the transverse reinforcement (measured out-to-out).

2.2 NOTATIONS

A = cross sectional area of hollow steel section (Section C7.2.3.3)

A_b = cross sectional area of an individual reinforcing steel bar (Sections 5.3.7.3, C5.3.8.2, C6.3.5, 9.5)

A_{cap}^{top} = area of bent cap top flexural steel (Sections 7.4.5.2, 7.4.5.3); area of top flexural reinforcement in each direction of slab superstructure within the effective width (Section 9.5)

A_{cap}^{bot} = area of bent cap bottom flexural steel (Sections 7.4.5.2, 7.4.5.3); area of bottom flexural reinforcement in each direction of slab superstructure within the effective width (Section 9.5)

A_{cv} = area of concrete engaged in interface shear transfer (Sections 6.3.5.2, 7.6.4)

A_e = effective shear area (Sections 5.3.7.2, 5.3.7.4)

A_g = gross cross sectional area (Sections C3.4.2, 5.3.3, 5.3.7.2, 5.3.9.1, 5.3.9.2)

A_{jh} = effective horizontal area of a moment resisting joint (Sections 7.4.2, 9.5)

A_{jh}^{fig} = effective horizontal area at mid-depth of the footing, assuming a 45° spread away from the boundary of the column in all directions (Section 6.2.2.2)

A_{jv} = effective vertical joint area for a moment resisting joint (Sections 7.4.2, 9.5)

A_s^{jh} = area of horizontal stirrups or ties required at moment resisting joints (Sections 7.4.5.2, 7.4.5.3); total area of horizontal ties in the effective width of slab superstructure along the longitudinal direction (Section 9.5)

A_s^{jhc} = total area of horizontal ties placed at the end of the bent cap in Case 1 Knee joints (Section 7.4.5.3); total area of horizontal end ties along the transverse direction for slab superstructure knee joints (Section 9.5)

A_s^{jv} = area of vertical stirrups or ties required at moment resisting joints (Sections 7.4.5.2, 7.4.5.3); total area of vertical stirrups in each of the Joint Region and Joint Perimeter of a slab superstructure (Section 9.5)

A_s^{j-bar} = area of vertical “J” bar reinforcement required at moment resisting joints with a skew angle greater than 20° (Sections 7.4.5.2, 7.4.5.3); area of “J” shaped bars inside the Core Region of a slab superstructure above the SCM (Section 9.5)

A_s^{jf} = total area of bent cap side face reinforcement required at moment resisting joints (Sections 7.4.5.2, 7.4.5.3, 9.5)

A_{sk} = area of interface shear reinforcement crossing the shear plane (Sections 6.3.5.1, 7.6.4)

A_{st} = total area of column longitudinal reinforcement anchored in the joint (Sections 7.4.5.2, 7.4.5.3, 8.3.1.4, 9.5); total cross sectional area of column longitudinal reinforcement (Sections C3.4.2, C5.3.8.2)

$A_{st,max}$ = maximum area of longitudinal reinforcement allowed in a seismic critical member (Section 5.3.9.1)

$A_{st,min}$ = minimum area of longitudinal reinforcement allowed in a seismic critical member (Section 5.3.9.2)

- A_s^{u-bar} = area of bent cap top and bottom reinforcement bent in the form of “U” bars in Knee joints (Section 7.4.5.3); area of additional bent cap reinforcement (u-shaped) for knee joints in the transverse direction of slab superstructures (Section 9.5)
- A_{sh} = area of horizontal shear key reinforcement in the stem wall (Section 6.3.5.1)
- A_{sk} = area of interface shear key reinforcement (Section 7.6.4)
- A_{sk}^{Iso} = area of interface shear reinforcement required for isolated shear key (Section 6.3.5.1)
- $A_{sk}^{Iso(provided)}$ = area of interface shear reinforcement provided for isolated shear key (Section 6.3.5.1)
- A_{sk}^{mono} = area of interface shear reinforcement required for monolithic shear key (Section 6.3.5.2)
- $A_{sk}^{mono(provided)}$ = area of interface shear reinforcement provided for monolithic shear key (Section 6.3.5.2)
- A_v = area of shear reinforcement perpendicular to the flexural tension reinforcement (Sections 5.3.7.3, 5.3.7.5, C5.3.8.2, 9.5)
- $A_{v, joint}$ = seismic critical member transverse reinforcement in the joint region of slab superstructures (Section 9.5)
- a = vertical distance from the location of the applied force on the shear key to the top surface of the stem wall (Section C6.3.5.1); thickness of joint filler (Sections 6.3.3, 7.2.3.2)
- B_c = column cross-sectional dimension perpendicular to the direction of bending (Section 6.2.2.2)
- B_{cap} = bent cap width (Sections 7.4.2, 7.4.3)
- B_{eff} = effective width of the superstructure for resisting longitudinal seismic moments (Section 7.2.1.1)
- B_{eff}^{ftg} = effective width of footing for calculating average normal stress in the vertical direction within a footing moment resisting joint (Section 6.2.2.2)
- B_{eff}^{slab} = effective width of slab superstructure (Section 9.3)
- b = vertical distance from the top surface of the stem wall to the centroid of the lowest layer of shear key horizontal reinforcement (Section C6.3.5.1)
- $C_{(i)}^{pile}$ = axial compression force demand on a pile (Section 6.2.3.2)
- c = damping ratio (Section 3.2.1.5)
- D_c = column cross-sectional dimension parallel to the direction of bending (Sections 6.2.2.2, 7.2.1.1, 7.4.2, 7.6.2, 8.3.1.4, 9.3, C9.5); diameter or cross sectional dimension of column in the direction of the cap width or length (Sections 7.4.3, 7.4.4.2, 7.4.5.2, 7.4.5.3)
- $D_{c.g}$ = distance from the top of column to the center of gravity of the superstructure (Section 4.4.3.1)
- $D_{c,max}$ = largest cross sectional dimension of a seismic critical member (Sections 5.3.8.5, 8.3.2)
- D_{ftg} = depth of footing (Sections 6.2.2.2, 6.2.2.4, 6.2.3.2, C6.2.4.2, 7.6.2)
- D_s = depth of superstructure at the bent cap (Sections 6.3.3, 7.2.1.1, 7.2.2, 7.2.3.2, 7.4.2, 7.6.2); depth of slab at the seismic critical member support (Sections 9.3, 9.5)

- D' = cross-sectional dimension of confined concrete core measured between the centerlines of the peripheral hoop or spiral reinforcement (Sections 5.3.7.3, 5.3.7.5, C5.3.8.2)
- D'_c = confined column cross-section dimension, measured out to out of ties, in the direction parallel to the axis of bending (Section C5.3.8.2)
- D^* = diameter for circular shafts (Section 5.3.4)
- D_n = thickness of layer n (Section Appendix B)
- d_b = nominal reinforcing bar diameter (Sections 6.3.5, 8.3.3)
- d_{bl} = nominal bar diameter of longitudinal reinforcement (Sections 5.3.4, 8.3.1.1, 8.3.2, 9.2)
- $d_{(i)x}$ = distance from pile (i) to the centerline of the column along the X axis (Section 6.2.3.2)
- $d_{(i)y}$ = distance from pile (i) to the centerline of the column along the Y axis (Section 6.2.3.2)
- E_c = modulus of elasticity of concrete (Sections 3.3.6, 3.4.2, 3.4.1)
- E_s = Modulus of elasticity of steel (Sections 3.3.3, 3.3.4)
- F_{abut} = idealized ultimate passive capacity of the backfill behind abutment backwall or diaphragm (Section 6.3.1.2)
- F_{bw} = idealized ultimate passive capacity of the backfill behind seat abutment backwall (Section 6.3.1.2)
- F_{dia} = idealized ultimate passive capacity of the backfill behind diaphragm abutment (Section 6.3.1.2)
- F_{sk} = abutment shear key force demand (Sections 6.3.4, 6.3.5.1)
- F_u = specified tensile strength of steel (Section C7.2.3.3)
- f'_c = specified compressive strength of unconfined concrete (Sections 3.3.1, 3.3.6, C3.4.2, 5.3.3, 5.3.7.2, 5.3.7.4, 6.2.2.2, 7.4.2, 7.4.5.1, 7.6.4)
- f'_{cc} = compressive strength of confined concrete (Section 3.3.6)
- f'_{ce} = expected compressive strength of unconfined concrete (Sections 3.3.1, 3.3.6)
- f_h = average normal stress in the horizontal direction within a moment resisting joint (Section 7.4.2)
- f_{ps} = tensile stress for 7-wire low relaxation prestress strand (Section 3.3.4)
- f_u = specified minimum tensile strength of reinforcement (Sections 3.3.3, 3.3.4)
- f_{ue} = expected tensile strength of reinforcement (Section 3.3.3)
- f_v = average normal stress in the vertical direction within a moment resisting joint (Sections 6.2.2.2, 7.4.2)
- f_y = specified minimum yield strength of reinforcement (Sections 3.3.1, 7.6.4)
- f_{ye} = expected yield strength of reinforcement (Sections 3.3.1, 5.3.4, 6.3.5.1)
- f_{yh} = specified minimum yield strength of transverse column reinforcement (Sections 5.3.7.2, 5.3.7.3, 5.3.7.5, 7.4.5.1)
- G = gap between the isolated flare and soffit of the bent cap; gap between the steel jacket and soffit of the bent cap or top of the footing (Sections 5.3.4, 7.6.3.1)

- G_c = shear modulus (modulus of rigidity) for concrete (Sections 3.3.6, 3.4.1)
- g = acceleration due to gravity (Sections C4.2.1, 7.2.2)
- H_{o-max} = length of shaft/column from point of maximum moment to point of contraflexure above ground considering the base of plastic hinge at the point of maximum moment (Section 5.3.4)
- h_{abut} = height of abutment backwall or diaphragm (Section 6.3.1.2)
- h_{bw} = abutment backwall height for seat abutment (Section 6.3.1.2)
- h_{dia}^* = height of diaphragm abutment if not designed for full soil pressure (Section 6.3.1.2)
- h_{dia}^{**} = height of diaphragm abutment if designed for full soil pressure (Section 6.3.1.2)
- I = moment of inertia (Section 3.4.1)
- I_{eff} = effective moment of inertia for computing member stiffness (Sections 3.4.2, C3.4.3)
- I_g = moment of inertia about centroidal axis of the gross section of a member (Sections C3.4.2, 3.4.3)
- $I_{p.g.(x)}$ = moment of inertia of the pile group about the X axis (Sections 6.2.3.2, C6.2.4.2)
- $I_{p.g.(y)}$ = moment of inertia of the pile group about the Y axis (Sections 6.2.3.2, C6.2.4.2)
- J = torsional moment of inertia for computing member stiffness (Section 3.4.1)
- J_{eff} = effective torsional moment of inertia for computing member stiffness (Section 3.4.4)
- J_g = gross torsional moment of inertia about centroidal axis of a member (Section 3.4.4)
- K = effective stiffness of a bent or frame (Section 4.2.1)
- K_{abut} = abutment longitudinal stiffness (Section 6.3.1.2)
- K_{eff} = effective abutment longitudinal stiffness (Sections 4.2.1, C4.2.1, 6.3.1.2, 6.3.1.3)
- K_{eff}^{adj} = effective abutment longitudinal stiffness adjusted for displacement coefficient (Section 6.3.1.3)
- K_{nom} = nominal abutment transverse stiffness (Section 6.3.2)
- K_{res} = residual stiffness for assessing abutment effectiveness (Section 6.3.1.3)
- k_i^e = effective stiffness of bent or column i (Section 7.1.2)
- k_j^e = effective stiffness of bent or column j (Section 7.1.2)
- K_θ = abutment longitudinal stiffness for theta degree skewed abutment (Section 6.3.1.2)
- L = member length from the point of maximum moment to the point of contra-flexure (Sections C5.2.2, 5.3.4); portion of the shaft length measured from the center of the plastic hinge to the center of gravity of the superstructure (Section C5.2.2); pad dimension along the bridge for elastomeric bearings, or length of masonry plate for PTFE bearings (Section 6.3.3)
- L_{col} = length of column (Section C6.2.6)
- L_{fg} = cantilever length of the footing or pile cap measured from face of column to edge of footing along the principal axis of the footing (Section 6.2.2.4)
- L_{min} = minimum horizontal distance from the vertical shear key bars to the hooked or headed end of the lowest layer of hanger bars for isolated shear key (Section C6.3.5.1)

- L_{mp} = length of masonry plate for the bearing (Section 7.2.3.2)
- L_p = equivalent plastic hinge length (Sections C5.2.2, 5.3.4)
- L_{pile} = length of pile/shaft (Section 6.2.6)
- L_{pr} = region(s) of a structural member member expected to form plastic hinge(s) (Section 5.3.2)
- l_{ac} = minimum length of longitudinal reinforcement in seismic critical members extended into cap beams or footings (Sections 8.3.1.1, 8.3.1.4)
- $l_{ac,provided}$ = actual length of column longitudinal reinforcement embedded into the bent cap (Sections 7.4.2, 7.4.5.2, 7.4.5.3, 9.5)
- l_d = development length in tension of straight bars (Section 7.4.5.3)
- l_{db} = basic tension development length (Section 7.2.2)
- l_{dh} = development length in tension of standard hooked bars (Section C6.3.5.1)
- M = moment (Section 5.3.6.2)
- MR = movement range (Sections 6.3.3, 7.2.3.2)
- M_D = moment demand at any location of a Type II shaft generated by the column's overstrength moment and associated shear (Section 6.2.5.3)
- M_D^R = moment demand on the right side of superstructure-to-column joint (Section 4.4.3.1)
- M_D^L = moment demand on the left side of superstructure-to-column joint (Section 4.4.3.1)
- M_{dl}^R = dead load plus added dead load moments on the right side of superstructure-to-column joint (Sections 4.4.3.1, 5.4.4)
- M_{dl}^L = dead load plus added dead load moments on the left side of superstructure-to-column joint (Sections 4.4.3.1, 5.4.4)
- $M_{dl}^{col-top}$ = dead load plus added dead load moments on the top of column (Section 4.4.3.1)
- $M_{eq}^{col-top}$ = seismic moment at the top of column (Sections 4.4.3.1, 4.4.3.2)
- M_{eq}^R = earthquake moment distributed to the right side of superstructure-to-column joint (Sections 4.4.3.1, 5.4.4)
- M_{eq}^L = earthquake moment distributed to the left side of superstructure-to-column joint (Sections 4.4.3.1, 5.4.4)
- M_{max} = maximum moment demand in Type II shaft (Section 6.2.5.3)
- M_n = nominal moment capacity based on the specified minimum concrete and steel strengths when the concrete strain reaches 0.003 (Section C4.4.4)
- M_{ne} = nominal moment capacity based on expected material properties and a concrete strain of 0.003 (Sections 5.3.6.2, 5.4.2, 6.2.5.3)
- $M_{ne,slab}$ = nominal moment capacity of a slab superstructure based on expected material properties and a concrete strain of 0.003 (Section 9.5)

- $M_{ne}^{\text{sup}(R)}$ = nominal moment capacity of the adjacent right superstructure span based on expected material properties and a concrete strain of 0.003 (Section 5.4.4)
- $M_{ne}^{\text{sup}(L)}$ = nominal moment capacity of the adjacent left superstructure span based on expected material properties and a concrete strain of 0.003 (Section 5.4.4)
- M_o^{col} = overstrength moment of a seismic critical member (Sections 4.4.2.1, 4.4.2.2, 6.2.5.3, 7.4.1, 7.4.2, 7.6.4)
- $M_o^{\text{col-top}}$ = column overstrength moment on the top of column (Section 4.4.3.1)
- $M_{o(x)}^{\text{col}}$ = component of the column overstrength moment demand about the X axis (Sections 6.2.3.2, C6.2.4.2)
- $M_{o(y)}^{\text{col}}$ = component of the column overstrength moment demand about the Y axis (Sections 6.2.3.2, C6.2.4.2)
- M_p = idealized plastic moment capacity calculated by $M-\phi$ analysis (Sections C4.4.4, C6.2.3.2)
- M_p^{col} = idealized plastic moment capacity of a seismic critical member calculated by $M-\phi$ analysis (Sections 4.4.2.1, 4.4.4, 5.3.2, 5.3.6.2, 9.5)
- $M_{p/s}^{\text{col-top}}$ = effective secondary prestress moment on the top of column (Section 4.4.3.1)
- $M_{p/s}^R$ = effective secondary prestress moment (after losses have occurred) on the right side of superstructure-to-column joint (Sections 4.4.3.1, 5.4.4)
- $M_{p/s}^L$ = effective secondary prestress moment (after losses have occurred) on the left side of superstructure-to-column joint (Sections 4.4.3.1, 5.4.4)
- $M_{p(x)}^{\text{pile}}$ = component of the pile plastic moment capacity at the pile cap connection due to total average axial load about the X axis (Section C6.2.4.2)
- $M_{p(y)}^{\text{pile}}$ = component of the pile plastic moment capacity at the pile cap connection due to total average axial load about the Y axis (Section C6.2.4.2)
- M_u = ultimate moment capacity of a seismic critical member (Section C4.4.4)
- M_y = moment of a seismic critical member corresponding to the first reinforcing bar yield (Sections 3.4.2, C4.4.4, 5.3.6.2)
- $M-\phi$ = moment curvature analysis (Sections 3.4.2, C5.2.2, 5.3.6.2, 5.4.2)
- m_i = tributary mass of bent or column i (Section 7.1.2)
- m_j = tributary mass of bent or column j (Section 7.1.2)
- N = blow count per foot for the California Standard Penetration Test (Section Appendix Figure B.10)
- N_A = abutment support length normal to centerline of bearing (Section 6.3.3)
- N_H = support length normal to the centerline of bearing (Section 7.2.3.2)
- N_p = total number of piles in the pile group (Sections 6.2.3.2, C6.2.4.2)
- $(N_1)_{60}$ = penetration resistance corrected for overburden pressure and hammer efficiency (Section 6.1.2)

- n = number of individual interlocking spirals or hoops in the core of the sections (Section 5.3.7.3); total number of piles at distance $d_{(i)x}$ or $d_{(i)y}$ from the centroid of the pile group (Section 6.2.3.2)
- P = absolute value of the net axial force normal to the shear plane (Section 7.6.4); axial compressive force (Section C3.4.2)
- P_b = beam axial force (including prestressing) at the center of a moment-resisting joint (Section 7.4.2)
- P_c = axial force on a seismic critical member including the effects of overturning (Sections 5.3.3, 5.3.7.2, 6.2.2.2, 7.4.2)
- P_{dl} = axial load due to dead load (Sections 4.4.4, 5.3.3); superstructure dead load reaction at the abutment plus the weight of the abutment and its footing (Section 6.3.4)
- P_{dl}^{sup} = superstructure axial load reaction at the abutment (Sections 4.3.1, 6.3.4)
- P_p = total axial load on the pile group including column axial load (dead load + EQ load due to any overturning effects), footing weight, and overburden soil weight (Sections 6.2.3.2, C6.2.4.2)
- p_c = nominal principal compression stress in a joint (Sections 6.2.2.2, 7.4.2)
- p_t = nominal principal tension stress in a joint (Sections 6.2.2.2, 7.4.2, 7.4.5.1)
- R_A = abutment displacement coefficient (Sections C4.2.1, 6.3.1.3)
- R_D = displacement reduction factor for damping ratios exceeding 5% (Section 3.2.1.5)
- R_{sk} = skew reduction factor (Section 6.3.1.2)
- R_{Rup} = site to rupture plane distance (Section Appendix B)
- S = cap beam short stub length (Section 7.4.4.2)
- S_d = 5% damped spectral displacement (Section 3.2.1.5)
- S'_d = spectral displacement modified for higher levels of damping (Section 3.2.1.5)
- S_a = design spectral acceleration coefficient at the structure period (Section 4.2.1)
- s = spacing of shear/transverse reinforcement measured along the longitudinal axis of the structural member (Sections 5.3.7.3, 5.3.7.5, C5.3.8.2)
- s_u = undrained shear strength (Section 6.1.2, Appendix Figure B.10)
- T_i = natural period of frame i (Sections 7.1.3, C7.2.3.2)
- T_j = natural period of frame j (Sections 7.1.3, C7.2.3.2)
- T_c = total tensile force in column longitudinal reinforcement associated with M_o^{col} (Sections 6.2.2.2, 7.4.2)
- $T_{(i)}^{pile}$ = axial tension force demand on a pile (Sections 6.2.2.2, 6.2.3.2)
- T_{jv} = net tensile force in moment resisting footing joints (Section 6.2.2.2)
- V = shear capacity of hollow structural section (Section C7.2.3.3)
- V_c = nominal shear capacity provided by concrete (Section 5.3.7)
- V_D = shear demand at any location of a Type II shaft generated by the column's overstrength moment and associated shear (Section 6.2.5.3)

- V_D^R = shear demand on the right side of superstructure-to-column joint (Sections 4.4.3.1, 4.4.3.2)
- V_D^L = shear demand on the left side of superstructure-to-column joint (Sections 4.4.3.1, 4.4.3.2)
- V_{dl}^R = dead load plus added dead load shears on the right side of superstructure-to-column joint (Sections 4.4.3.2, 5.4.4)
- V_{dl}^L = dead load plus added dead load shears on the left side of superstructure-to-column joint (Sections 4.4.3.2, 5.4.4)
- $V_{dl}^{col-top}$ = dead load plus added dead load shear on the top of column (Section 4.4.3.2)
- V_e = maximum elastic lateral force (Section C4.2)
- V_{eq}^L = seismic shear on the left side of superstructure-to-column joint (Sections 4.4.3.2, 5.4.4)
- V_{eq}^R = seismic shear on the right side of superstructure-to-column joint (Sections 4.4.3.2, 5.4.4)
- $V_{eq}^{col-top}$ = shear force at the top of column due to seismic (Sections 4.4.3.1, 4.4.3.2)
- V_{max} = maximum shear demand in Type II shaft (Section 6.2.5.3)
- V_n = nominal shear capacity (Sections 5.3.7, 5.4.3, 6.2.5.3)
- $V_n^{sup(R)}$ = nominal shear capacity of the adjacent right superstructure span (Section 5.4.4)
- $V_n^{sup(L)}$ = nominal shear capacity of the adjacent left superstructure span (Section 5.4.4)
- V_o^{col} = overstrength shear force (Sections 4.4.2.2, 5.3.7, 6.2.3.2, 6.2.5.3, 7.4.1, 7.6.4)
- $V_o^{col-top}$ = overstrength shear on the top of column (Section 4.4.3.2)
- $V_{o(x)}^{col}$ = component of column overstrength shear demand along the X axis (Section C6.2.4.2)
- $V_{o(y)}^{col}$ = component of column overstrength shear demand along the Y axis (Section C6.2.4.2)
- V_p = inelastic lateral force (Section C4.2)
- $V_{p/s}^L$ = effective secondary prestress shear (after losses have occurred) on the left side of superstructure-to-column joint (Sections 4.4.3.2, 5.4.4)
- $V_{p/s}^R$ = effective secondary prestress shear (after losses have occurred) on the right side of superstructure-to-column joint (Sections 4.4.3.2, 5.4.4)
- $V_{p/s}^{col-top}$ = effective secondary prestress shear on the top of column (Section 4.4.3.2)
- V_{piles} = sum of lateral capacity of the piles (Section 6.3.4)
- V_p^{col} = plastic shear capacity corresponding to the idealized plastic moment capacity of a SCM (Section C4.4.2.2)
- V_s = nominal shear capacity provided by shear reinforcement (Sections 5.3.7, 5.3.7.3, 5.3.7.4)
- V_{ww} = shear capacity of one wingwall (Section 6.3.4)
- ν = poisson's ratio (Section 3.3.6)
- ν_c = permissible concrete shear stress (Section 5.3.7.2)

- v_{jv} = nominal shear stress in the vertical direction within a moment resisting joint (Sections 6.2.2.2, 7.4.2)
- v_s = shear wave velocity (Section 6.1.2, Appendix B)
- v_n = shear wave velocity of layer n (Section Appendix B)
- v_{s30} = shear wave velocity for the upper 30 m of the soil profile (Section Appendix B)
- W = tributary weight of the structure (Section 4.2.1)
- w = unit weight of concrete (Section 3.3.6)
- w_{abut} = abutment width along the skew direction (Section 6.3.1.2)
- w_{bw} = width of backwall for seat abutment (Section 6.3.1.2)
- w_{cap} = width of drop cap (Sections 9.3, C9.5)
- w_{dia} = width of diaphragm abutment (Section 6.3.1.2)
- Z = depth to rock parameter (Section Appendix B)
- $Z_{1.0}$ = depth to 1.0 km/s shear wave velocity (Section Appendix B)
- $Z_{2.5}$ = depth to 2.5 km/s shear wave velocity (Section Appendix B)
- α = adjustment factor for shear key force demand (Section 6.3.4)
- Δ = deflection (Section 4.4.4); deflection at pile cut-off elevation (Section C6.2.6)
- Δ_{abut} = abutment displacement at idealized yield (Section 6.3.1.2)
- Δ_b = displacement due to flexibility of capacity protected members (Section 3.5.1)
- Δ_c = local displacement capacity (Section C5.2.2)
- Δ_{col} = displacement due to the elastic and plastic deformation of the column (Section 3.5.1)
- Δ_C = frame or bent displacement capacity in the local principal/critical axis of member (Sections 3.5.1, 5.2.2); support length of abutment or in-span hinge (Section 3.5.1)
- Δ_{cr+sh} = relative displacement due to creep and shrinkage (Sections 6.3.3, 7.2.3.2)
- Δ_d = local member displacement demand (Section 5.3.7.2)
- Δ_D = frame or bent displacement demand in the local principal/critical axis of member (Sections 3.5.1, 4.2, 4.4.1, 4.4.4, 6.3.1.3)
- Δ_{D1} and Δ_{D2} = earthquake displacement demands on either side of an in-span hinge (Section 7.2.3.2)
- Δ_{eff} = effective abutment longitudinal displacement (Sections 6.3.1.2, 6.3.1.3)
- Δ_{eq} = displacement demand for the frame adjacent to the abutment (Section 6.3.3); relative longitudinal earthquake displacement demand at an expansion joint (Section 7.2.3.2)
- Δ_f = displacement due to foundation flexibility (Section 3.5.1)
- Δ_{gap} = width of expansion gap at seat abutment (Sections 6.3.1.2, C6.3.1.2)
- $\Delta_{max,elastic}$ = displacement corresponding to maximum elastic lateral force (Section C4.2)
- $\Delta_{max,inelastic}$ = displacement corresponding to maximum inelastic lateral force (Section C4.2)
- Δ_p = idealized plastic displacement capacity due to rotation of the plastic hinge (Section C5.2.2)

- $\Delta_{p/s}$ = relative displacement due to pre-stress shortening (Sections 6.3.3, 7.2.3.2)
- Δ_{p^*} = plastic displacement (Section 3.5.1)
- Δ_r = relative lateral offset between the point of contra-flexure and the base of the plastic hinge (Section 4.4.4)
- Δ_s = shaft displacement at the point of maximum moment (Section 4.4.4)
- Δ_{temp} = relative displacement due to thermal expansion and contraction (Sections 6.3.3, 7.2.3.2)
- Δ_Y^{col} = idealized yield displacement of a seismic critical member (Sections 3.5.1, C5.2.2)
- $\Delta_{Y(i)}$ = frame or bent displacement at the instant a plastic hinge forms in the i-th seismic critical member (Sections 4.4.1, 5.3.7.2)
- ϵ_{cc} = compressive strain at maximum compressive stress of confined concrete (Section 3.3.6)
- ϵ_{co} = unconfined concrete compressive strain at the maximum compressive stress (Section 3.3.6)
- ϵ_{sp} = ultimate compressive strain (spalling strain) of unconfined concrete (Section 3.3.6)
- ϵ_{cu} = ultimate compression strain for confined concrete (Section 3.3.6)
- ϵ_{ps} = tensile strain for 7-wire low relaxation prestress strand (Section 3.3.4)
- $\epsilon_{ps,EE}$ = tensile strain in prestress steel at the essentially elastic limit state (Section 3.3.4)
- $\epsilon_{ps,u}^R$ = reduced ultimate tensile strain in prestress steel (Section 3.3.4)
- ϵ_{sh} = tensile strain at the onset of strain hardening for A706 reinforcement (Section 3.3.3)
- ϵ_{su} = ultimate tensile strain for A706 reinforcement (Section 3.3.3)
- ϵ_{su}^R = reduced ultimate tensile strain for A706 reinforcement (Sections 3.3.3, 5.4.2)
- ϵ_y = nominal yield strain for A706 reinforcement (Section 3.3.3)
- ϵ_{ye} = expected yield strain for A706 reinforcement (Section 3.3.3)
- θ = angular difference between the local and global directions (Section C4.2.2); abutment skew angle (Sections 6.3.1.2, 6.3.3, C7.2.3.2)
- θ_p = plastic hinge rotation capacity (Section C5.2.2)
- λ = load reduction factor for a pile group (Section 6.2.3.2)
- μ_c = local displacement ductility capacity (Section C5.2.2)
- μ_D = displacement ductility demand (Sections 4.4.1, 5.3.7.2)
- μ_d = local displacement ductility demand (Section 5.3.7.2)
- ρ_c = axial load ratio due to dead load and overturning (Section 5.3.3)
- ρ_{dl} = axial load ratio due to dead load (Sections 5.3.3, 5.3.8.2)
- ρ_l = percent longitudinal reinforcement ratio (Section 5.3.8.2)
- ρ_s = volumetric ratio of transverse reinforcement (Sections 5.3.7.2, C5.3.8.2, 7.4.5.1, 8.3.1.4)
- ρ_s^T = volumetric ratio of column transverse reinforcement extended into a T joint region (Section 7.4.5.2)

ρ_s^{knee} = volumetric ratio of column transverse reinforcement extended into a Knee joint region
(Section 7.4.5.3)

ϕ = load and resistance factor (Sections 3.6, 5.3.7, 5.4.4, 6.2.5.3, 7.6.4)

ϕ_p = idealized plastic curvature capacity (Section C5.2.2)

ϕ_u = ultimate curvature capacity defined as the curvature when the concrete reaches the ultimate compression strain ε_{cu} or the longitudinal reinforcing steel reaches the reduced ultimate strain ε_{su}^R
(Sections C3.5.1, 5.2.2, C5.2.2, 5.3.6.2)

ϕ_y = idealized yield curvature defined by an elastic- perfectly plastic $M-\phi$ curve of the cross section
(Sections C5.2.2, 5.3.6.2)

ϕ_y = yield curvature corresponding to the first yield of the reinforcement (Sections 3.4.2, 5.3.6.2)

2.3 ACRONYMS AND INITIALISMS

ABC = Accelerated Bridge Construction (Section C1.1)

AASHTO = American Association of State Highway and Transportation Officials (Sections C1.1, C8.3.1.1, C8.3.2)

AASHTO-CA BDS = current California approved AASHTO LRFD Bridge Design Specifications, and the corresponding California Amendments to AASHTO LRFD Bridge Design Specifications (Sections C1.1, 5.4.2, 5.4.3, 6.2.2.1, 6.2.2.6, C6.2.2.6, C6.2.4.3, C6.2.5.1, C6.3.4, C6.3.5.1, C6.3.5.2, 7.2.2, 7.3.1, 7.6.4, 8.4.1.2, 8.4.2)

ASTM = American Society for Testing and Materials (Sections 2.1, C3.3.1, 3.3.3, C3.3.3, C3.3.4, 5.3.4, C6.1.2)

BDA = Bridge Design Aids (Sections C7.2.3.3, C7.2.3.4, 9.1, C9.5)

Caltrans = California Department of Transportation (Sections 1.1, 1.2.3, C1.2.3, 1.2.4, C1.4, C1.5, 3.2.1, C3.2.1.1, C3.3.3, C3.3.6, C4.2, C5.2.2, C5.3.6.2, 6.2.3.2, 6.2.4.1, C8.1, Appendix B)

CIDH = Cast-in-drilled-hole (Sections 5.4.5, C6.2.5.1, 6.2.5.3, 6.2.6)

CIP = Cast-in-place (Sections C1.1, C7.2.1.2, 7.2.2, C9.6)

CISS = Cast-in-steel-shell pile (Sections 5.3.4, 5.4.5, C6.2.5.1, 6.2.5.3)

CPM = Capacity Protected Member (Sections 1.4, 5.4.1, 5.4.2, 5.4.3, 8.2.3.1)

CQC3 = Complete Quadratic Combination 3 (Sections 4.2.2, C4.2.2)

DES = Caltrans' Division of Engineering Service (Section 1.1)

DS = Design Spectrum (Sections C1.4, 3.2.1); Damage State (Sections C1.3, C3.5.1)

DSH = Design Seismic Hazards (Sections 1.4, C1.4, 3.1, 3.2, C4.3.1)

EDA = Elastic Dynamic Analysis (Sections 3.5.2, 4.2, C4.2, C4.2.1, 4.2.2, C4.2.3, 4.3.1, 6.3.1.1, C6.3.1.1, C7.2.3.3, C7.2.3.4)

EQC = Earthquake Committee (Sections 1.5, C1.5)

ERE = Earthquake Resisting Element (Sections 1.4, 3.1, C3.1)

ESA = Equivalent Static Analysis (Sections C3.5.1, 3.5.2, 4.2, C4.2, 4.2.1, C4.2.1, C4.2.3, 4.3.1, 5.2.2, C5.2.2, 6.3.1.1, C6.3.1.1, C7.2.3.3)

FEE = Functional Evaluation Earthquake (Sections 1.3, 2.1, 3.2.1.2, C3.2.1.2, 4.4.1, 7.5.2)

ISA = Inelastic Static Analysis (Sections 5.2.2, C5.2.2, 6.2.4.2, 6.3.1.1, C6.3.1.1)

METS = Caltrans Materials Engineering and Testing Services (Section C3.3.3)

MTD = Caltrans Bridge Memo to Designers (Sections C1.1, C1.2.1, C1.2.2, C1.5, C3.2.2, C3.2.3, C4.2.2, C4.2.3, C6.2.3.2, C6.2.5.1, C6.3.4, C7.5.1, C7.6.3.4, C9.6)

NTHA = Nonlinear Time History Analysis (Sections 3.5.2, 4.2, C4.2, C4.2.1, 4.2.3, C4.2.3, 4.3.1, C6.3.1.1, Appendix A)

OEEAR = Office of Earthquake Engineering, Analysis and Research (Sections 1.5, C1.5, C4.2)

PC = Precast (Sections C1.1, 7.2.1.2, 7.2.2)

PGA = Peak Ground Acceleration (Sections 7.2.1.2, C7.2.1.2, Appendix A)

PSDC = Project Specific Seismic Design Criteria (Sections 1.1, C1.1, C1.2.1, C1.3, 2.1, 3.2.1.5, C3.2.1.5, 3.2.4, 3.3.3, C3.3.3, 5.3.1, C5.3.8.2, 6.2.4.3, C7.2.3.2, C7.5.1)

P/S = Prestressed (Concrete or Strand) (Sections 4.4.3.1, 6.3.3, 7.2.3.2)

PTFE = Polytetrafluoroethylene (spherical bearings) (Sections 6.3.3, C6.3.3, 7.2.3.1, 7.5.1, C7.5.1)

SBE = State Bridge Engineer (Sections 1.5, C1.5)

SC = Structure Construction (Sections 1.5, C1.5)

SCM = Seismic Critical Member (Sections 1.4, C1.4, 2.1, 3.1, C3.3.1, 3.3.3, C3.3.3, 3.4.2, 3.4.4, 4.4.1, C4.4.1, 4.4.2, 4.4.2.1, C4.4.2.1, 4.4.2.2, C4.4.2.2, 4.4.4, C5.2.2, 5.3.1, C5.3.1, 5.3.2, C5.3.2, 5.3.3, C5.3.3, 5.3.6.2, 5.3.4, C5.3.4, C5.3.5, C5.3.6.2, 5.3.7.1, 5.3.7.2, 5.3.7.5, 5.3.8.1, C5.3.8.1, 5.3.8.2, C5.3.8.2, 5.3.9.1, 5.3.9.2, 5.3.9.3, 5.4.1, C5.4.1, 5.4.4, 6.2.5.4, 7.2.1.2, 7.3.1, C7.3.3, C7.4.2, 7.4.5.2, C7.4.5.2, 7.4.5.3, C7.4.5.3, 8.2.2.1, C8.2.2.1, 8.2.2.2, 8.2.2.3, C8.2.2.3, 8.3.1.1, 8.3.1.3, 8.3.1.4, 8.4.1.2, 8.4.2, 9.2, C9.2, 9.3, 9.4, C9.4, 9.5, C9.5, Appendix A)

SDC = Seismic Design Criteria (Sections 1.1, C1.1, C1.2.2, C1.4, 1.5, C1.5, 2.1, C3.2.1.1, C4.2, 4.3.2, C5.3.8.2, C6.1.1, 6.2.1, C7.5.1, C8.3.2, 9.1, Appendix A, Appendix B)

SEE = Safety Evaluation Earthquake (Sections 1.3, 2.1, 3.2.1.1, C3.2.1.1, 4.4.1)

SQM = Structure Quality Management (Sections 1.5, C1.5)

SRSS = Square Root of Sum of Squares (Section 4.2.2)

Sup = Superstructure (Sections 5.4.4, 7.6.2)

TSC = Technical Steering Committee (Sections 1.5, C1.5)



GENERAL REQUIREMENTS

3.1 EARTHQUAKE RESISTING ELEMENTS

Earthquake Resisting Elements (EREs) shall be used within the earthquake resisting system to dissipate energy or increase the bridge damping during the design seismic hazards. Where applicable, EREs shall undergo inelastic deformation without causing structure collapse.

Standard EREs include plastic hinges in seismic critical members (SCMs), and soil behind abutment backwalls, as shown in Figure 3.1-1.

Earthquake resisting elements should be accessible for inspection and repair.

Devices that may prevent plastic hinging/dissipation of energy shall not be attached to, bored into, or cast into the potential plastic hinge locations of a SCM.

3.2 DESIGN SEISMIC HAZARDS

The Design Seismic Hazards (DSH) shall include ground shaking, surface fault rupture, and their associated effects such as liquefaction, lateral spreading, and tsunami.

3.2.1 Ground Shaking

Ground shaking shall be characterized for design by the Design Spectrum (DS). The design spectrum shall be based on the most current Caltrans approved U.S. Geological Survey Seismic Hazard Maps. A qualified geo-professional shall provide final design spectrum recommendations.

3.2.1.1 Safety Evaluation Earthquake

The Design Spectrum for Safety Evaluation Earthquake (SEE) shall be taken as a spectrum based on a 5% probability of exceedance in 50 years (or 975-year return period).

C3.1

Refer to Figure C3.1-1 for locations of EREs within earthquake resisting systems.

Nonstandard EREs include seismic isolation bearings and energy dissipation devices such as dampers, which limit the inertial forces transferred between the superstructure and substructure.

Type I shafts may hinge below the ground line. However, their use is not subject to the exception procedure because of their robust ductile behavior coupled with the fact that the foundation soil could be excavated to access and repair the plastic hinge.

Devices such as barrier rails, may disrupt the performance of plastic hinges.

C3.2.1

The design spectrum reflects the shaking hazard at or near the ground surface.

For Design Spectrum development, refer to Appendix B.

C3.2.1.1

The SEE represents a rare ground motion that may occur during the life of the structure. Larger ground motions are possible.

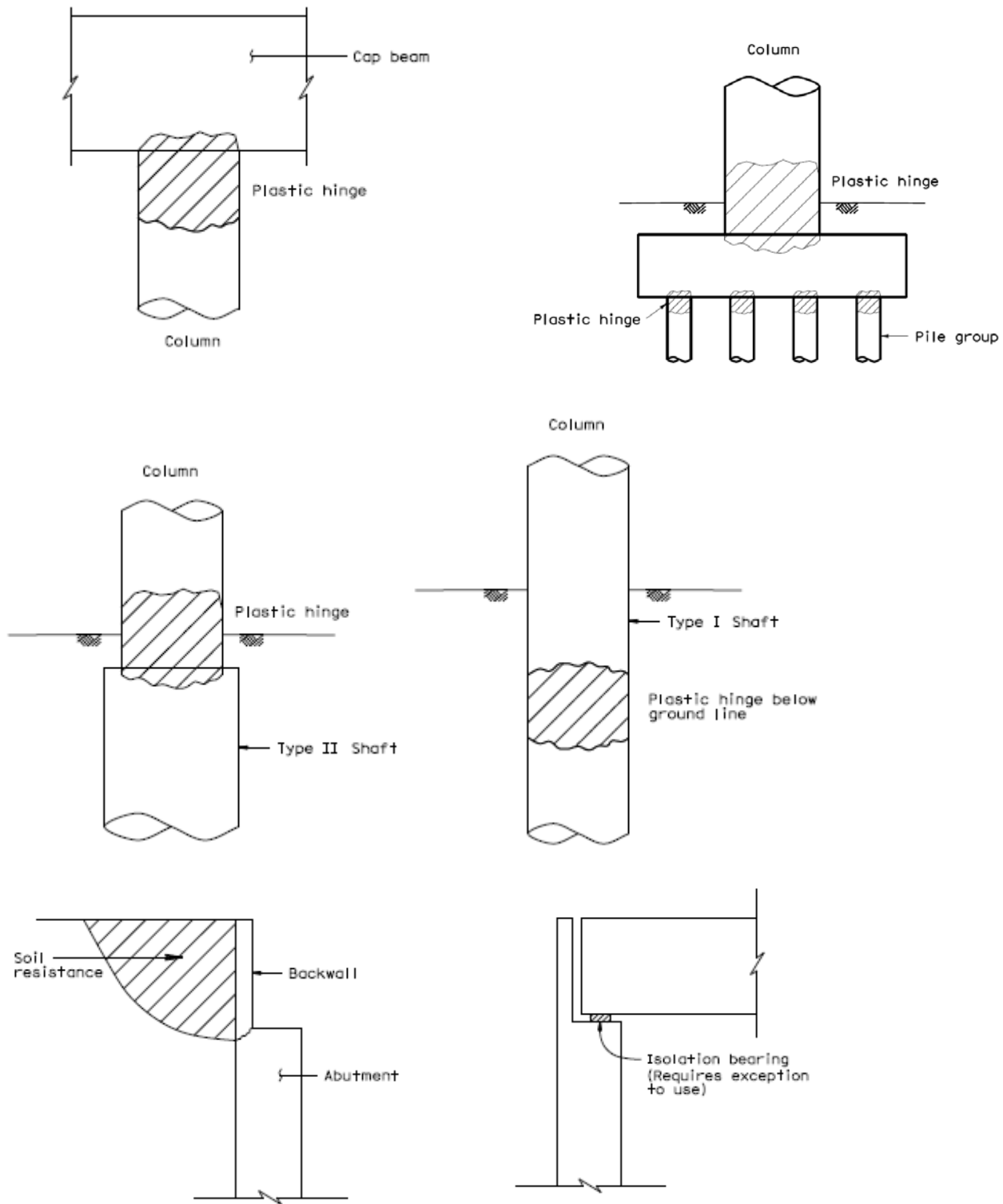


Figure 3.1-1: Examples of Earthquake Resisting Elements

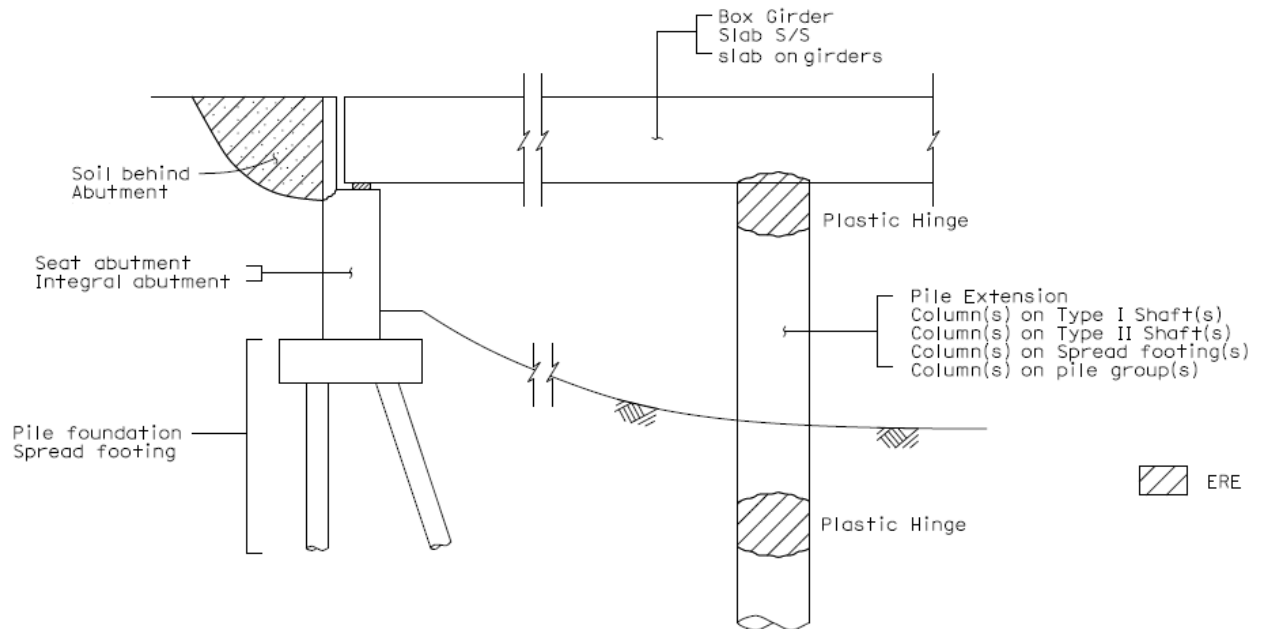


Figure C3.1-1: Locations of Standard Earthquake Resisting Elements in Earthquake Resisting Systems

The design spectrum based on a 5% probability of exceedance in 50 years is, for all practical purposes, equivalent to that based on a 7 % probability of exceedance in 75 years. The latter matches the 75 years design life of a bridge.

A web-based design tool is available for use in the specification of the design spectrum: (http://dap3.dot.ca.gov/ARS_Online/). The technical references link at this website contains links to several helpful documents including Caltrans' fault database and a link to the USGS Interactive Deaggregation website.

Several aspects of design spectrum development require special knowledge related to the determination of fault location (utilization of original source mapping where appropriate) and interpretation of the site profile and geologic setting for incorporation of site effects.

A detailed discussion of the development of the probabilistic design spectra as well as possible adjustment factors is given in Appendix B.

The deterministic criterion used in previous editions of the SDC as one of the parameters in the definition of design spectrum was dropped in the current SDC 2.0. Recent seismic source models for California include a much broader range of considered rupture scenarios making selection of M_{\max} somewhat arbitrary. Furthermore, performance based earthquake engineering relies on probabilistic estimates of demand and capacity, therefore deterministic measures are unsuitable. In the case of a rare earthquake on a fault considered to be inactive, SDC compliant bridges have ample reserve capacity to prevent collapse.

3.2.1.2 Functional Evaluation Earthquake

The Design Spectrum for Functional Evaluation Earthquake (FEE) shall be taken as a spectrum based on a 20 % probability of exceedance in 50 years (or 225-year return period).

C3.2.1.2

The FEE represents an earthquake that has a significant chance of occurring during the lifespan of a bridge.

The FEE can be obtained using the USGS Interactive Deaggregation website (<https://earthquake.usgs.gov/hazards/interactive/>). The FEE spectrum is constructed by running the deaggregation tool for 20% exceedance in 50-year hazard for each of the ten available spectral periods. Intermediate periods can be obtained using linear interpolation.

3.2.1.3 Horizontal Ground Motion and Directionality Effects

In determining seismic displacement demands, uncertainty in directionality effects of horizontal ground motion shall be accounted for as specified in Sections 4.2.1, 4.2.2, and 4.2.3.

C3.2.1.3

Except for cases of near-fault shaking (generally, site to fault distances less than 15.5 miles (25 km)), earthquake ground shaking hazard has a random orientation and may be equally probable in all horizontal directions. The method for obtaining the maximum demands on bridge members due to the directionality of ground shaking depends on the analysis method and complexity of the bridge. Refer to Section 4.2 for analysis methods.

3.2.1.4 Effects of Vertical Ground Excitation

The effects of vertical ground excitation shall be estimated by applying an equivalent static vertical load to the superstructure as specified in Section 7.2.2.

3.2.1.5 Damping

A 5% damped elastic response spectrum shall be used for determining seismic demand. A PSDC shall be required to use damping ratios other than 5%.

C3.2.1.5

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. If allowed via the PSDC procedure, a reduction to the 5% damped spectral displacement may be applied as shown in Equation C3.2.1.5-1.

$$S'_d = (R_D) \times (S_d) \quad (\text{C3.2.1.5-1})$$

where:

$$R_D = \frac{1.5}{[40c + 1]} + 0.5 \quad (\text{C3.2.1.5-2})$$

S'_d = spectral displacement modified for higher levels of damping

S_d = 5% damped spectral displacement

R_D = displacement reduction factor for damping ratios exceeding 5%

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

The following characteristics are typically good indicators that higher damping may be anticipated:

- Total length less than 300 feet,
- Three spans or less,
- Abutments designed for sustained soil mobilization,
- Skew less than or equal to 20 degrees, and
- Continuous superstructure without in-span hinges or expansion joints.

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.

3.2.2 Liquefaction

The liquefaction potential within the project site shall be investigated. Bridges subject to liquefaction shall be analyzed and designed as specified in Section 6.2.4.3.

3.2.3 Fault Rupture

The potential for fault rupture below or near the bridge shall be considered.

C3.2.2

When locations are identified as being susceptible to liquefaction and/or lateral spreading, the geo-professional should provide recommendations that include a discussion of the following:

- Potential horizontal and vertical ground displacements
- Possible remediation strategies including ground improvement, avoidance, and/or structural modification

For additional information on quantifying the impacts of soil liquefaction and lateral spreading, see MTD 20-14 and 20-15.

C3.2.3

Preliminary investigation of fault rupture hazard includes the identification of nearby active surface faults that may cross beneath a bridge or proposed bridge, per MTD 20-10. If a fault trace underlies a structure or the structure falls within the specified fault zone, then the geo-professional will provide the following recommendations:

- Location and orientation of fault traces or zones with respect to structures,
- Expected horizontal and vertical displacements, and
- Description of additional evaluations or investigations that could refine the above information.

Refer to Caltrans Geotechnical Manual for additional information on fault rupture.

3.2.4 Additional Seismic Hazards

Other seismic hazards that are identified at a project site shall be assessed in accordance with the PSDC procedure.

3.3 MATERIAL PROPERTIES

3.3.1 Expected Material Properties

The capacity of reinforced and prestressed concrete components to resist all seismic demands except shear, shall be based on expected material properties (i.e., expected compressive strength of unconfined concrete, f'_{ce} and expected yield strength of reinforcement, f_{ye}).

Seismic shear capacity shall be based on the specified material strengths (i.e., specified minimum yield strength for steel reinforcement, f_y , and specified compressive strength of unconfined concrete, f'_c).

C3.3.1

Use of expected material strengths instead of specified strengths provides a realistic estimate for design strength (Unanwa and Mahan, 2014). An expected concrete compressive strength, f'_{ce} recognizes the typically conservative nature of concrete batch design, and the expected strength gain with age. Also, given that the specified minimum yield strength f_y for ASTM A706 Grade 60 steel can range between 60 ksi and 78 ksi while that of ASTM A706 Grade 80 steel can range between 80 ksi and 98 ksi, an expected reinforcement yield strength, f_{ye} better represents the actual strength than the specified minimum of 60 ksi or 80 ksi. The possibility that the yield strength may be less than f_{ye} in seismic critical members (SCMs) will conservatively impact capacity protected components. The possibility that the yield strength may be less than f_{ye} in capacity protected members is accounted for in the overstrength magnifier specified in Section 4.4.2.1.

3.3.2 Nonlinear Reinforcing Steel Models

Reinforcing steel shall be modeled with a stress-strain relationship that exhibits an initial linear elastic portion, a yield plateau, and a strain hardening range in which the stress increases with the strain, as shown in Figure 3.3.2-1. The yield point shall be defined by the expected yield strength of the steel, f_{ye} .

C3.3.2

The strain-hardening curve can be modeled as a parabola or other non-linear relationship and should terminate at the ultimate tensile strain, ϵ_{su} . The ultimate strain is set at the strain at peak stress. The length of the yield plateau is a function of the steel strength and bar size.

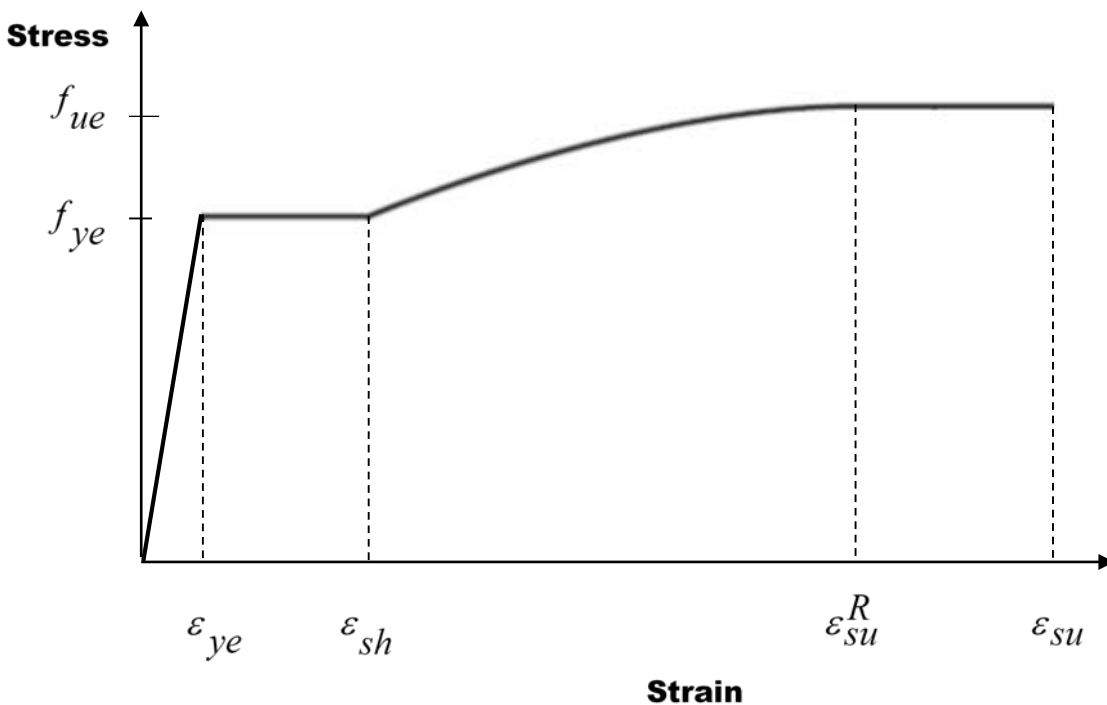


Figure 3.3.2-1 Steel Stress-Strain Model

3.3.3 Reinforcing Steel

ASTM A706 reinforcing steel shall be used for seismic applications, as follows:

- ASTM A706 Grade 60 – for all applications, including use in SCMs and capacity protected members. Hooks, headed bar terminations, and splices shall be allowed.
- ASTM A706 Grade 80 – for straight bars in capacity protected members only. A PSDC shall be required to use Grade 80 reinforcing steel for hooks, headed bar terminations, splices, and couplers.

The properties of ASTM Grades 60 and 80 reinforcing steel, as specified in Table 3.3.3-1, shall be used. Additional test data, if available, may be used in-lieu of those specified in Table 3.3.3-1 subject to the PSDC procedure.

For design purposes, the reduced ultimate tensile strain, ϵ_{su}^R instead of the ultimate tensile strain, ϵ_{su} shall be used.

C3.3.3

Grade 80 bars are not to be used in SCMs until definitive data from ongoing research become available.

Use of Grade 80 reinforcing steel for headed bar terminations, hooks, and couplers may be permitted on a project specific basis (PSDC) based on availability of specific project data.

The properties for A706 Grade 60 reinforcing steel were established on the basis of data from a collection of mill certificates and augmented by testing conducted by Caltrans Material Engineering and Testing Services (METS).

With the exception of the reduced ultimate tensile strain, ϵ_{su}^R , the properties for A706 Grade 80 reinforcing steel are based on a number of monotonic pull tests obtained through a Caltrans-funded research project (Overby et al., 2015).

The reduced ultimate strain instead of the ultimate tensile strain is used for design in order to decrease the probability of fracture of the reinforcement. For Grade 60 steel, the ultimate tensile strain was reduced by up to thirty-three percent to obtain the reduced ultimate strain. For Grade 80 reinforcement, a value of 0.084 for ϵ_{su}^R was proposed by Overby et al. (2015) based on the 5th percentile of the test results. The specified ϵ_{su}^R value of 6% for Grade 80 reinforcement was adopted in order to be conservative.

Table 3.3.3-1 Properties of ASTM A706 Grades 60 and 80 Reinforcing Steel

Property	Grade 60	Grade 80
Modulus of elasticity, E_s	29,000 ksi	29,000 ksi
Specified minimum yield strength, f_y	60 ksi	80 ksi
Expected yield strength, f_{ye}	68 ksi	85 ksi
Specified minimum tensile strength, f_u	80 ksi	98 ksi
Expected tensile strength, f_{ue}	95 ksi	112 ksi
Nominal yield strain, ϵ_y	0.0021	0.0028
Expected yield strain, ϵ_{ye}	0.0023	0.0033
Ultimate tensile strain, ϵ_{su}	0.120 - #10 bars and smaller 0.090 - #11 bars and larger	0.095
Reduced ultimate tensile strain, ϵ_{su}^R	0.090 - #10 bars and smaller 0.060 - #11 bars and larger	0.060
Strain at onset of strain hardening, ϵ_{sh}	0.0150 - #8 bars 0.0125 - #9 bars 0.0115 - #10 & #11 bars 0.0075 - #14 bars 0.0050 - #18 bars	0.0074

3.3.4 Nonlinear Prestressing Steel Model

Prestressing steel shall be modeled with a nonlinear stress strain model, as shown in Figure 3.3.4-1.

The curves in Figure 3.3.4-1 may be approximated by Equations 3.3.4-1 to 3.3.4-4.

Approximate yield prestress steel strain, $\varepsilon_{ps,EE}$

$$\varepsilon_{ps,EE} = \begin{cases} 0.0076 & \text{for } f_u = 250 \text{ ksi} \\ 0.0086 & \text{for } f_u = 270 \text{ ksi} \end{cases}$$

Reduced ultimate prestress steel strain, $\varepsilon_{ps,u}^R$

$$\varepsilon_{ps,u}^R = 0.03$$

250 ksi Strand:

$$\varepsilon_{ps} \leq 0.0076 : f_{ps} = 28500\varepsilon_{ps} \quad (3.3.4-1)$$

$$\varepsilon_{ps} \geq 0.0076 : f_{ps} = 250 - (0.25/\varepsilon_{ps}) \quad (3.3.4-2)$$

270 ksi Strand:

$$\varepsilon_{ps} \leq 0.0086 : f_{ps} = 28500\varepsilon_{ps} \quad (3.3.4-3)$$

$$\varepsilon_{ps} \geq 0.0086 : f_{ps} = 270 - \frac{0.04}{\varepsilon_{ps} - 0.007} \quad (3.3.4-4)$$

where:

ε_{ps} = tensile strain for 7-wire low relaxation prestress strand

f_{ps} = tensile stress for 7-wire low relaxation prestress steel strand (ksi)

C3.3.4

Figure 3.3.4-1 is an idealized stress-strain model for 7-wire low-relaxation prestressing strand.

For material properties pertaining to high strength bars, refer to ASTM A722 - Standard Specification for High-Strength Steel Bars for Prestressed Concrete.

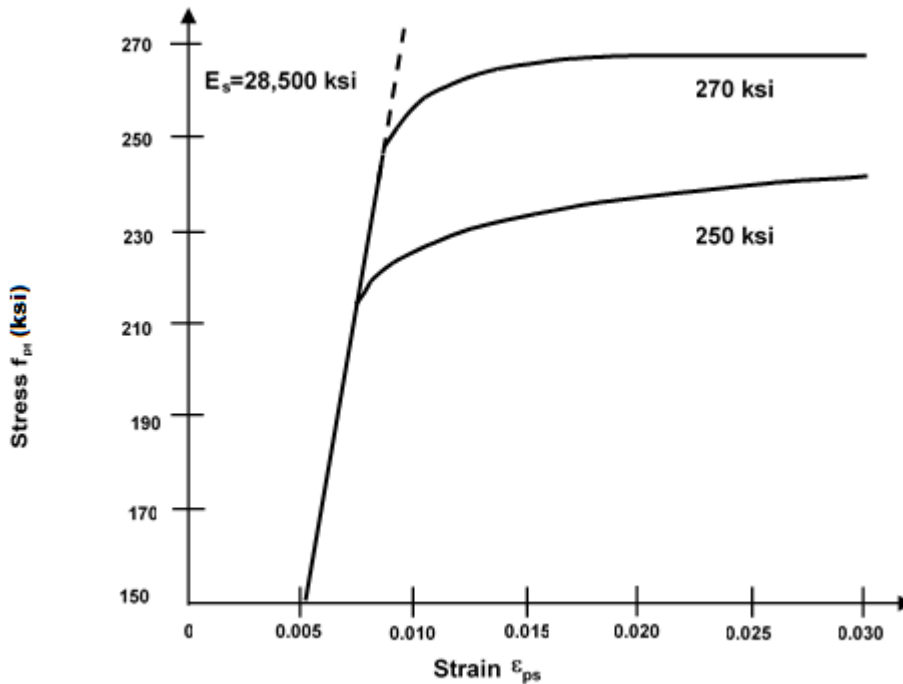


Figure 3.3.4-1 Prestressing Strand Stress Strain Model

3.3.5 Nonlinear Concrete Models

Mander's stress-strain model, as shown in Figure 3.3.5-1, shall be used to determine the capacity of concrete members.

C3.3.5

In Figure 3.3.5-1, the initial ascending curve may be represented by the same equation for both the confined and unconfined model since the confining steel has no effect in this range of strains. The confined concrete model continues to ascend until the confined compressive strength, f'_{cc} is reached. This segment is followed by a descending curve dependent on the parameters of the confining steel. The ultimate strain, ϵ_{cu} is the point where strain energy equilibrium is reached between the concrete and the confinement steel.

Additional information on Mander's stress-strain model is available in Mander et al. (1988a).

3.3.6 Normal Weight Concrete

Properties of normal weight concrete shall be taken as specified in Table 3.3.6-1.

Values of confined compressive strain, ϵ_{cc} and ultimate compression strain for confined concrete, ϵ_{cu} shall be defined by Mander's constitutive stress strain model for confined concrete.

C3.3.6

Typical values for the ultimate compression strain, ϵ_{cu} range from 0.012 to 0.05, depending on the amount of transverse confinement reinforcement (Priestley et al. 1996).

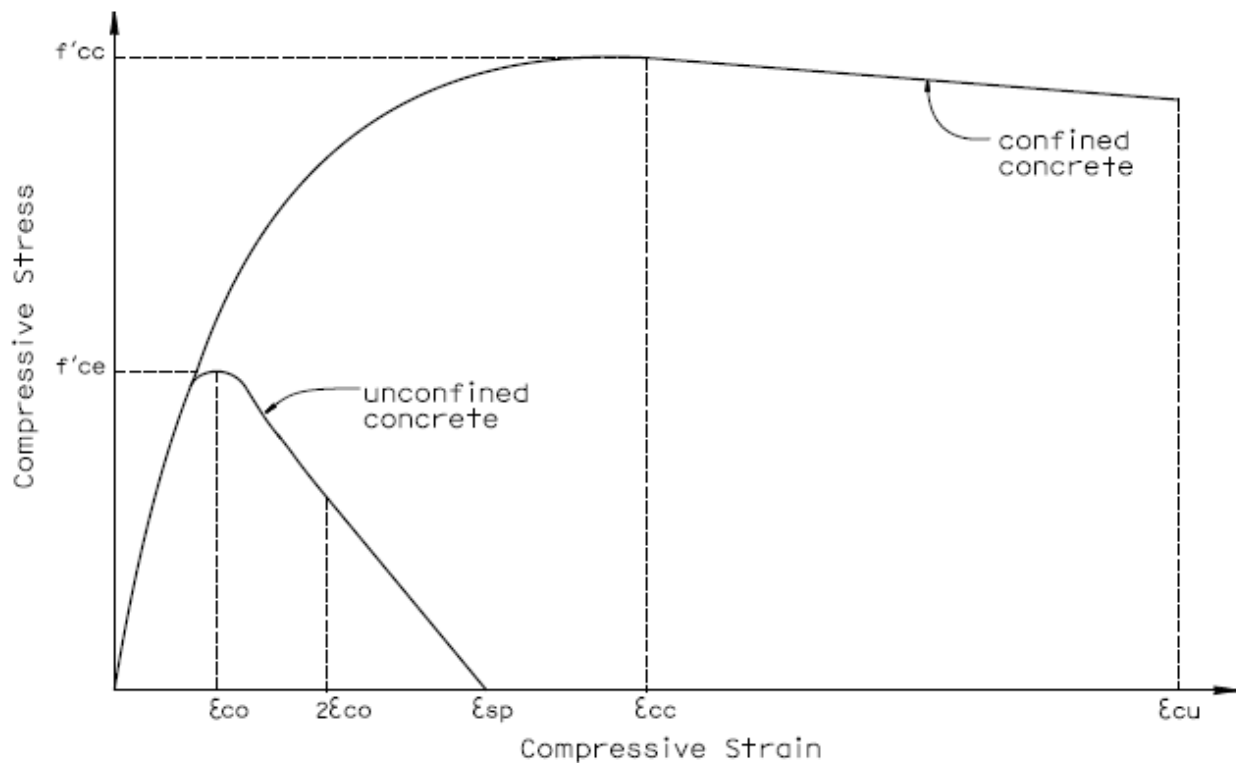


Figure 3.3.5-1 Concrete Stress Strain Model (Mander, 1988a)

Table 3.3.6-1 Properties of Normal Weight Concrete

Property	Design Value	Equation No.
Modulus of Elasticity	$E_c = 33w^{1.5} \sqrt{f'_{ce}}$ <p>where:</p> E_c = modulus of elasticity of concrete (psi) w = unit weight of concrete (lb/ft ³) f'_{ce} = expected compressive strength of unconfined concrete (psi)	(3.3.6-1)
Shear Modulus	$G_c = \frac{E_c}{2(1 + \nu_c)}$	(3.3.6-2)
Poisson's Ratio	$\nu = 0.2$	(3.3.6-3)
Expected Concrete Compressive Strength	$f'_{ce} = 1.3f'_c$ but not less than 5000 psi	(3.3.6-4)
Unconfined concrete compressive strain at the maximum compressive stress	$\epsilon_{co} = 0.002$	(3.3.6-5)
Ultimate unconfined compressive strain (spalling)	$\epsilon_{sp} = 0.005$	(3.3.6-6)
Confined compressive strain	ϵ_{cc} - defined by Mander's model	
Ultimate compression strain for confined concrete	ϵ_{cu} - defined by Mander's model	

3.4 EFFECTIVE SECTION PROPERTIES

3.4.1 General

Section properties, flexural rigidity $E_c I$ and torsional rigidity, $G_c J$ shall reflect the cracking that occurs before yield is reached.

3.4.2 Effective Moment of Inertia for Seismic Critical Members

The effective moment of inertia, I_{eff} for SCMs shall be determined from the secant slope of the Moment-Curvature ($M-\phi$) curve between the origin and the point designating the first reinforcing bar yield as:

$$E_c I_{eff} = \frac{M_y}{\phi_y} \quad (3.4.2-1)$$

where:

M_y = moment of the section at first yield of the reinforcing steel (lb-in.)

ϕ_y = yield curvature (rad/in)

I_{eff} = effective moment of inertia (in.⁴)

C3.4.1

Concrete members display nonlinear response before reaching their idealized yield limit state.

C3.4.2

$M-\phi$ curve is obtained from cross-section analysis as specified in Section 5.3.6.2.

For preliminary design, I_{eff} may be estimated as:

$$I_{eff} = \left\{ A + \frac{\frac{P}{f'_c A_g} - 0.05}{0.2} (B - A) \right\} I_g \leq B I_g \quad (C3.4.2-1)$$

in which:

$$A = 0.32 + 12 \left(\frac{A_{st}}{A_g} - 0.01 \right) \quad (C3.4.2-2)$$

$$B = 0.42 + 9 \left(\frac{A_{st}}{A_g} - 0.01 \right) \quad (C3.4.2-3)$$

where:

P = axial compressive force (lb)

f'_c = specified minimum compressive strength of unconfined concrete (psi)

A_g = gross cross section area (in.²)

A_{st} = total area of longitudinal reinforcement in the section (in.²)

Equations C3.4.2-1 to C3.4.2-3 were derived from the effective stiffness charts developed by Nuncio-Cantera and Priestley (1991) for axial load ratios, $(P / f'_c A_g)$ less than or equal to 0.25.

3.4.3 Effective Moment of Inertia for Superstructures and Cap Beams

The effective moment of inertia for reinforced concrete sections may be estimated between $0.5I_g - 0.75I_g$, where I_g is the gross section moment of inertia.

For prestressed concrete sections, I_g shall be used.

3.4.4 Effective Torsional Moment of Inertia

The effective torsional moment of inertia, J_{eff} for SCMs shall be determined as:

$$J_{eff} = 0.2J_g \quad (3.4.4-1)$$

where:

J_g = gross torsional moment of inertia

The torsional moment of inertia for superstructures shall not be reduced.

3.5 GLOBAL DISPLACEMENT CRITERIA

3.5.1 General

Each frame or bent shall satisfy:

- Ordinary Standard Bridge

$$\Delta_C \geq \Delta_D \quad (3.5.1-1)$$

- Recovery Standard Bridge

$$\Delta_C \geq 1.4 \Delta_D \quad (3.5.1-2)$$

where:

Δ_C = frame or bent displacement capacity in the local principal/critical axis of member

Δ_D = frame or bent displacement demand in the local principal/critical axis of member

Δ_D and Δ_C shall be measured in the same local principal/critical axis.

C3.4.3

The lower bound represents lightly reinforced sections and the upper bound represents heavily reinforced sections.

I_{eff} in reinforced concrete superstructures is dependent on the extent of cracking and effect of the cracking on the element's stiffness.

C3.4.4

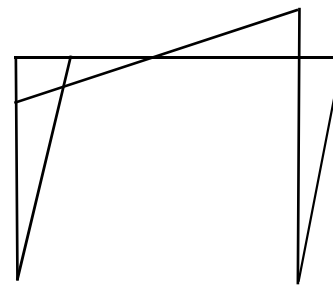
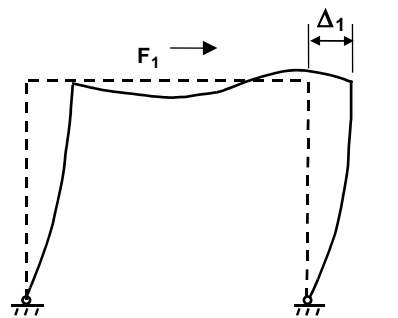
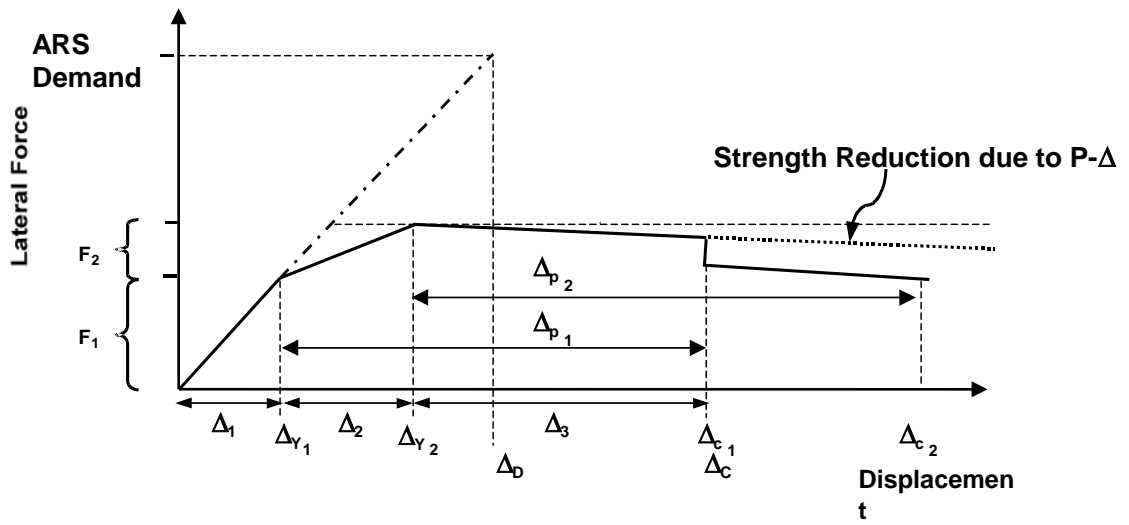
The torsional stiffness of concrete members can be greatly reduced after the onset of cracking.

C3.5.1

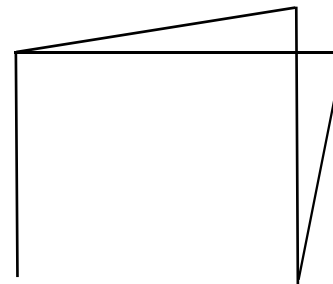
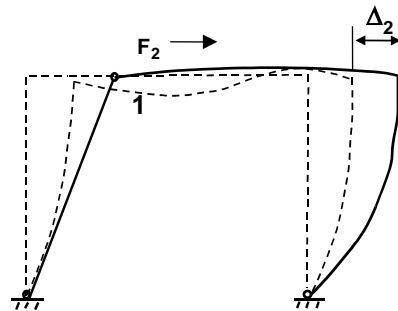
Figure 3.5.1-1 strictly represents the case where the bridge is assumed to have two principal directions defined by the direction along the chord connecting the ends of the frame and the direction perpendicular to it, as in Equivalent Static Analysis (ESA) where a curved bridge is straightened out.

A hinge that exceeds the ultimate curvature capacity, ϕ_u is vulnerable to shear failure and a consequent loss of axial capacity. It should be noted that as a structure develops plastic hinges, the displacement when the first hinge reaches ϕ_u defines Δ_C , even if that hinge was not the first to form.

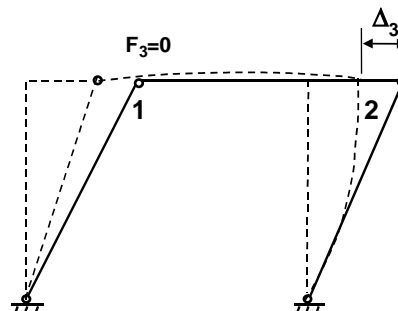
The factor of 1.4 in the displacement criterion of Equation 3.5.1-2 is intended to ensure that Recovery bridges perform as specified in Section 1.3 even when subjected to seismic hazard levels greater than the design seismic hazards. The factor was obtained



Moment Diagram 1



Moment Diagram 2



Idealized Frame

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

Figure 3.5.1-1 Global Force Deflection Relationship

Δ_D shall be obtained by performing analyses as specified in Section 4.2. Δ_D shall correspond to the maximum displacement as shown in Figure 3.5.1-1.

Δ_C shall be obtained by performing analysis as specified in Section 5.2.2. Δ_C shall correspond to the lateral capacity of the bridge or bent when any plastic hinge reaches its ultimate curvature capacity, ϕ_u as shown in Figure 5.3.6.2-1.

Abutments and in-span hinges shall also satisfy Equations 3.5.1-1 and 3.5.1-2, where Δ_C and Δ_D shall be taken as:

Δ_C = support length of the abutment or in-span hinge as specified in Sections 6.3.3 and 7.2.3.2

Δ_D = frame/bridge displacement demand in the longitudinal direction measured at the abutment or in-span hinge

The analytical model for determining the displacement demands and capacity shall include the contribution of foundation flexibility, Δ_f as shown in Figures 3.5.1-2 and 3.5.1-3 and the flexibility of capacity protected members, Δ_b , as shown in Figure 3.5.1-3. In Figure 3.5.1-2, Δ_{P^*} = portion of the plastic displacement capacity which, when added to Δ_Y^{col} and Δ_f , equals the displacement demand.

3.5.2 Local Principal/Critical Axes

The local principal/critical axes for a frame or bent shall be taken as:

For Equivalent Static Analysis (ESA):

- (a) the direction along a chord connecting the ends of the frame
- (b) the direction perpendicular to the chord defined in (a)

For Elastic Dynamic Analysis (EDA):

- (a) the direction along a chord connecting the ends of the frame
- (b) the direction along the centerline of a bent
- (c) the direction perpendicular to the centerline of the bent defined in (b)

by analyzing a sample of mostly single column bridges across Northern and Southern California with fixed column bases for the 2475-year and 975-year return period design spectra and averaging the resulting ratios of the displacement demands produced by the 2475-year return period DS to that produced by the 975-year return period DS (Yoon et al., 2018). Limited analyses on bridges supported on deep foundations indicate that the factor of 1.4 may result in a conservative design for bridges on deep foundations relative to those on shallow foundations. An adjustment to the factor of 1.4 may be warranted when the analysis results of a more comprehensive bridge database become available.

The displacement capacity, Δ_C is measured when all plastic hinges within the structure have formed and any one of the plastic hinges reaches its ultimate curvature capacity.

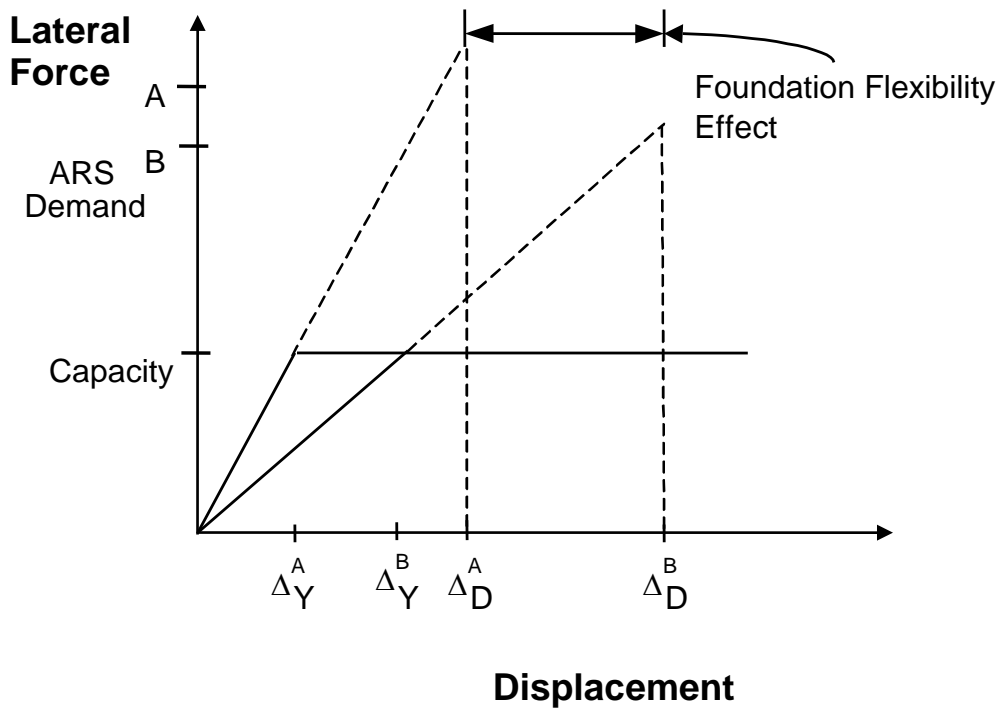
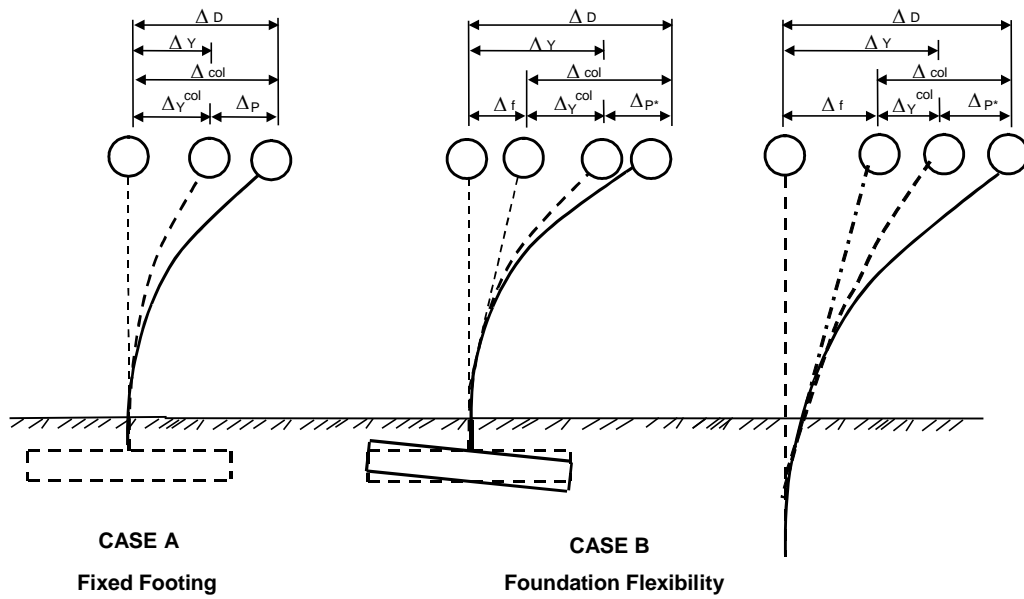


Figure 3.5.1-2 Effects of Foundation Flexibility on the Force-Displacement Curve of a Single Column Bent

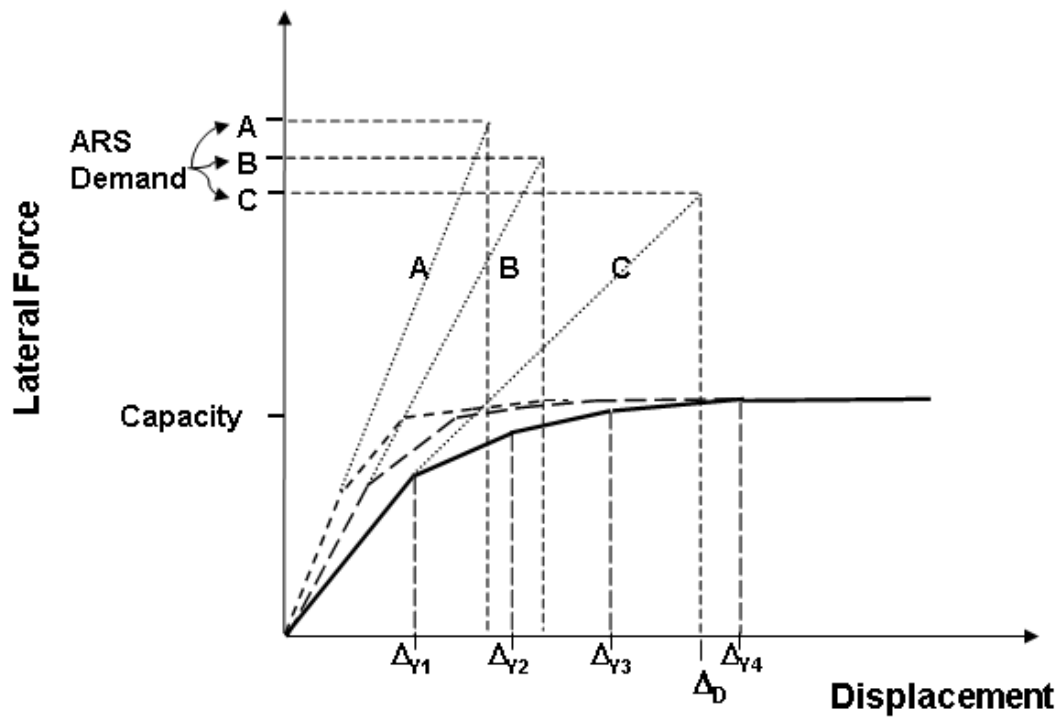
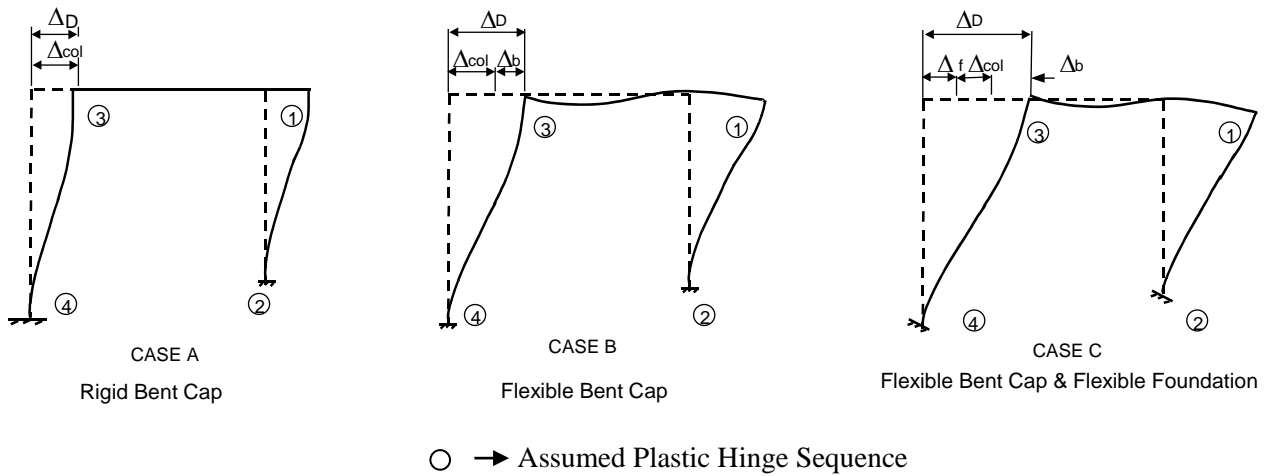


Figure 3.5.1-3 Effects of Bent Cap and Foundation Flexibility on Force-Deflection Curve of a Bent

For Nonlinear Time History Analysis (NTHA):

- (a) the direction along the centerline of a bent
- (b) the direction perpendicular to the centerline of the bent defined in (a)

3.6 LOAD AND RESISTANCE FACTORS

For all seismic-related calculations, the following factors shall be used:

- resistance factor for flexure, $\phi = 1.0$
- resistance factor for shear, $\phi = 1.0$
- load factor for Dead Load = 1.0
- load factor for Live load = 0



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SEISMIC DEFORMATION DEMANDS AND ANALYSIS METHODS

4.1 GENERAL

Seismic deformation demands shall be determined using the methods specified in the following sections.

4.2 ANALYSIS METHODS

Equivalent Static Analysis (ESA), Elastic Dynamic Analysis (EDA), or Nonlinear Time History Analysis (NTHA) shall be used to determine displacement demands, as specified in Table 4.2-1.

Equal displacement approximation of structural response shall apply to ESA and EDA.

When NTHA method is used, the designer shall also perform an ESA of the same bridge as a rough check on the analysis results. The results obtained by NTHA shall supersede those obtained by ESA provided the reduction in displacement demands are within 50% of the ESA values.

C4.1

Seismic deformation demands are characterized by the displacements, rotations, curvatures, or strain experienced by the structure. Displacement ductility and curvature ductility are the most frequently used measures of inelastic response (Priestley et al., 1996). The former relates to overall structural response while the latter relates to the response of an individual section.

C4.2

The equal displacement approximation (Veletsos and Newmark, 1960) assumes that the maximum displacement resulting from the inelastic response of a structure is approximately equal to the displacement obtained from an analysis using the linear elastic response spectrum, as shown in Figure C4.2-1.

The threshold for applicability of the equal displacement approximation is taken conservatively by Caltrans as an initial period greater than or equal to 0.7 seconds.

Forces and moments resulting from linear-elastic analyses (i.e., ESA and EDA) are unrealistic and should not be used for design. Only the displacement estimates are useable.

Nonlinear Time History Analysis is included for routine bridge design in the current version of the *SDC* as the method can, in general, be used to analyze any bridge. However, to guard against the use of improper NTHA models and ensure that analyses results are credible, NTHA results are checked against those obtained by the simplified, although approximate ESA. If the displacement demand obtained by NTHA is less than 50% of those obtained by ESA, the NTHA result may be used if the designer provides a justification acceptable to the Office Chief of OEEAR.

Table 4.2-1 Applicability of Methods for Displacement Demand Analysis

PARAMETER	ANALYSIS METHOD		
	ESA	EDA	NTHA
Maximum bridge length	1000 ft	3000 ft ^{*1}	No restriction
Maximum skew angle	30°	No restriction	No restriction
Maximum bearing difference between supports	5°	20°	No restriction

*1 The maximum bridge length requirement shall not apply when EDA is used for viaducts with repeating frame systems and geometry.

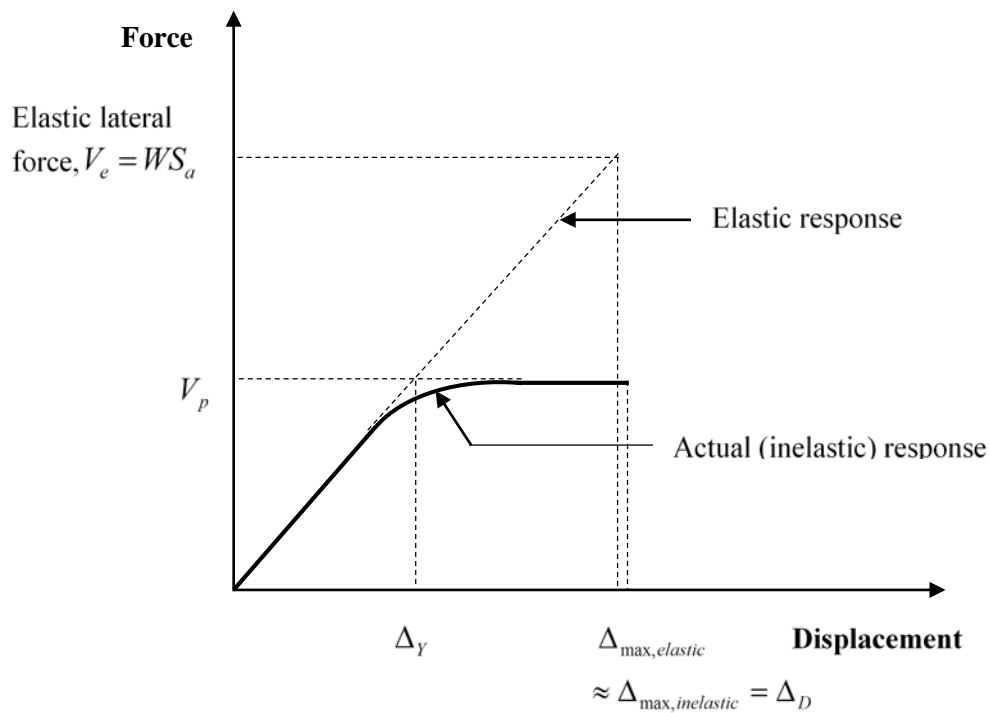


Figure C4.2-1 Equal Displacement Approximation

4.2.1 Equivalent Static Analysis

Equivalent Static Analysis (ESA) may be used to determine displacement demands where the structural response can be idealized by a single mode, subject to the limitations specified in Table 4.2-1.

The displacement demand, Δ_D shall be determined as:

$$\Delta_D = \frac{WS_a}{K} \quad (4.2.1-1)$$

where:

S_a = design spectral acceleration coefficient at the structure period

W = tributary weight of the structure

K = effective stiffness of the bent or frame

The effective stiffness shall be taken as the slope of the line passing through the origin and the point representing formation of the first structural plastic hinge on the force - displacement curve.

The design spectrum shall be applied along the principal directions of the bridge. The resulting displacement demands need not be combined.

For the special case of single span bridges, the displacement demand in the longitudinal direction shall be determined as follows:

- (a) Use the tributary weight of the superstructure, W and the effective abutment longitudinal stiffness, K_{eff} to determine the structure period, T using Equation C4.2.1-1
- (b) With the period, determine the spectral acceleration, S_a from the Design Spectrum
- (c) Determine the displacement demand from Equation 4.2.1-1

C4.2.1

In ESA, the seismic excitation is assumed to be an equivalent static horizontal force applied to individual frames. The horizontal force is applied at the vertical center of mass of the superstructure and distributed horizontally in proportion to the mass distribution. The lateral force-displacement curve is obtained from a pushover analysis of the bridge.

The period, T of a bent or frame is calculated as:

$$T = 2\pi\sqrt{(W / gK)} \quad (C4.2.1-1)$$

where:

g = acceleration due to gravity

For straight non-skewed bridges, the displacements obtained using ESA are generally close to those obtained using EDA or NTHA. However, for horizontally curved or skewed bridges, the 2-D displacements obtained by ESA are generally more conservative than the 3-D displacements obtained by EDA and NTHA.

Effective abutment longitudinal stiffness, K_{eff} is determined as specified in Section 6.3.1, including the effectiveness assessment using abutment displacement coefficient, R_A .

Determination of displacement demand in the transverse direction of single span bridges is not required as the bridge translation in that direction is limited upon failure of the shear key.

4.2.2 Elastic Dynamic Analysis

For EDA, the design spectrum shall be used to perform a linear elastic multi-modal spectral analysis. The number of modes considered in the analysis shall be sufficient to capture at least 90% mass participation in each of the longitudinal and transverse directions. A minimum of three elements per column and four elements per span shall be used for developing the analytic model.

For multi-frame bridges, two dynamic analyses, shall be performed to capture the upper bound of the assumed nonlinear response of the bridge, as follows:

- (a) an individual frame model with no abutment participation (i.e., a discrete tension model)
- (b) a continuous global model with all frames locked together (i.e., a global compression model). Abutment stiffness shall be included in the global compression model, as specified in Section 4.3.1.

The design spectrum shall be applied simultaneously in two directions along a set of orthogonal axes. The total response shall be determined by using either of the following methods:

- Complete Quadratic Combination 3 (CQC3)
- Square Root of Sum of Squares (SRSS)

For the SRSS method, the design spectrum shall be rotated to determine the total response. A minimum of four rotations at 30° intervals shall be used.

C4.2.2

The response of multi-frame bridges is complicated by the interaction of frames that vibrate with different frequencies and mode shapes. Models (a) and (b) represent situations where the joints open and close, respectively, bounding the bridge response. Depending on software capabilities, model (a) could be one of several stand-alone frame models, or a global model made discontinuous by the use of releases between frames.

The CQC3 method (Menun and Der Kiureghian, 1998) does not require rotations of the design spectrum or structure axes in order to produce the maximum response as it uses a formula to determine the critical orientation of the design spectrum that produces the maximum response. Refer to MTD 20-17 for additional information on Elastic Dynamic Analysis.

Generally, realistic results are obtained if the analysis tool is capable of producing displacement results along the principal axes of each member. If displacements in the local transverse (Δ_T) and longitudinal (Δ_L) directions are not directly available from the analysis tool, they may be approximated from the global X and Y displacements as follows (see Figure C4.2.2-1):

$$\Delta_L = |\Delta_X \cos \theta| + |\Delta_Y \sin \theta| \quad (\text{C4.2.2-1})$$

$$\Delta_T = |\Delta_X \sin \theta| + |\Delta_Y \cos \theta| \quad (\text{C4.2.2-2})$$

where:

θ = angular difference between the local and global directions (see Figure C4.2.2-1).

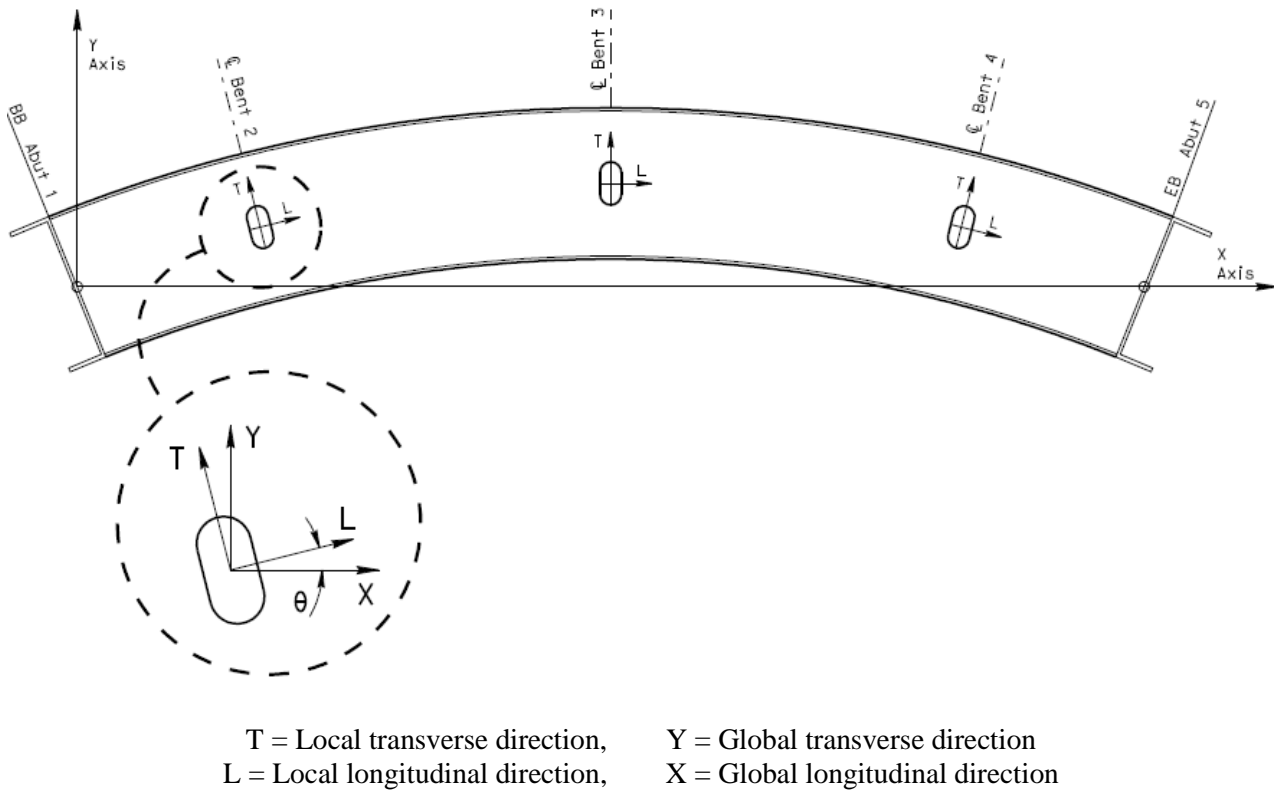


Figure C4.2.2-1 Local and Global Coordinate Systems

4.2.3 Nonlinear Time History Analysis

Nonlinear Time History Analysis (NTHA) shall model soil-foundation-structure interaction, gaps and impact on gap closure, multiple support excitations, isolation/damper devices, and nonlinear soil, material, and hysteretic behavior.

General damping of bridge components shall be modeled using an average Rayleigh damping ratio of 3% specified at two periods, namely (a) the fundamental period, and (b) the period corresponding to attainment of 80% mass modal participation.

The input ground motions shall be applied at the bridge supports and shall consist of at least seven acceleration or displacement time histories based on the fault type, earthquake intensity, fault proximity, and geology of the area.

C4.2.3

Nonlinear Time History Analysis (NTHA) is the most comprehensive method for capturing the seismic demands of bridge members. However, it requires more resources than ESA and EDA when all three analysis methods are applicable. If geometric non-linearity is included in the NTHA model, the $P-\Delta$ requirement of Section 4.4.4 need not be checked.

Unlike the ESA and EDA methods, NTHA is capable of capturing the effects of nonlinear behavior such as yielding of structural components, soil-foundation-structure interaction, opening and closing of expansion joints, geometric nonlinearity, and nonlinear behavior of restrainers and abutments.

All ground motions shall be matched to the site's design spectrum using the time domain method.

Each set of ground motions shall be rotated in 45° increments for at least a total of three angular directions (i.e., 0°, 45°, and 90°) to maximize the structural response.

For ground motions representing near-fault scenarios and fault crossing where the orientation of the motions has been specified by the geoprofessional, the motions shall not be rotated or spectrum-matched.

4.3 STRUCTURAL MODELING

4.3.1 Global Models

Except as provided in Section 4.3.2, a "global" (or "structural system") model of the bridge shall be used to capture the displacement demand of the entire bridge system when EDA or NTHA is used. This analytical model shall include the effects of adjacent components, subsystems, and boundary conditions.

For bridges longer than 3000 feet, multiple partial models of the bridge shall be permitted.

Each partial model shall include a boundary frame or an abutment beyond each end of the frame under consideration, as shown in Figure 4.3.1-1.

A global model shall include soil-foundation-structure interaction to determine the maximum displacement demand.

For NTHA, the entire soil-foundation system shall be modeled explicitly with soil springs attached to the structural foundation elements, as illustrated in Figure 6.2.4.2-2. The entire force-deflection curve of the soil shall be used in the model.

For ESA and EDA, the entire soil-foundation system shall be modeled either:

- (a) explicitly as described above for NTHA but with the initial slope of the force-deflection curve used for the soil stiffness, or

Methods of developing seismic ground motions for time history analysis include:

- (a) Using synthetic motions based on site characteristics
- (b) Creating idealized motions by matching existing records to a target acceleration response spectrum
- (c) Using seed records from the Next Generation Attenuation (NGA) database and site conditions to create site-specific motions.

C4.3.1

The boundary frames approximate the interaction between adjacent models but their analytical results are ignored. A spring may be attached to the unconnected end of the boundary frames to represent the stiffness of the remaining structure.

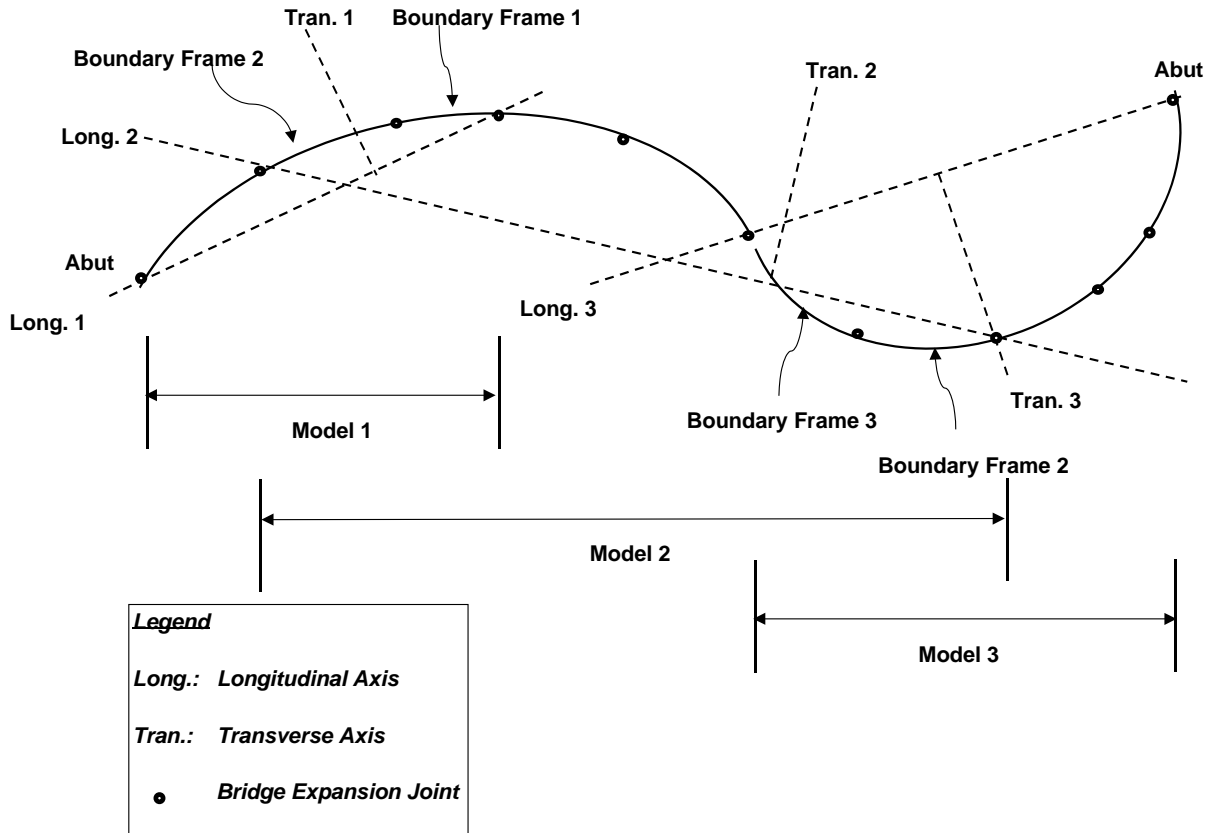


Figure 4.3.1-1 Multi-Frame Modeling Techniques

(b) by use of an equivalent 3-D stiffness matrix. An equivalent 3-D stiffness matrix shall be determined from the nonlinear static pushover analysis of the soil-foundation system that uses the explicit soil-structure foundation model. The equivalent stiffness shall be the idealized initial stiffness of the force-deflection curve of the pushover analysis.

Abutment stiffness shall be included in the global model as follows:

EDA and ESA Methods

Information on abutment longitudinal modeling is provided in Section 6.3.1.

- For bridges with three or less number of frames, one-half the abutment longitudinal stiffness shall be used at each abutment.
- For bridges with four or more number of frames, abutment longitudinal stiffness shall be ignored.
- Abutment transverse stiffness shall be as specified in Section 6.3.2.

NTHA Method

- Abutment longitudinal stiffness represented by a bilinear or nonlinear model, contact/gap elements, and other nonlinear elements shall be modeled in the analysis, regardless of the number of frames.
 - Abutment transverse stiffness shall be estimated using the shear key stiffness, as shown in Figure 4.3.1-2
- The shear key is usually designed to fail under the design seismic hazards (DSH) unless it is specifically designed to sustain DSH level loading. After the shear key fails, the lateral resistance is provided by the friction between the superstructure and abutment, and the coefficient of friction is assumed to be 0.3. The initial slope is based on an assumed 2 inch gap typically present between the superstructure and the shear key.

In Figure 4.3.1-2, P_{dl}^{sup} is the superstructure axial load resultant at the abutment.

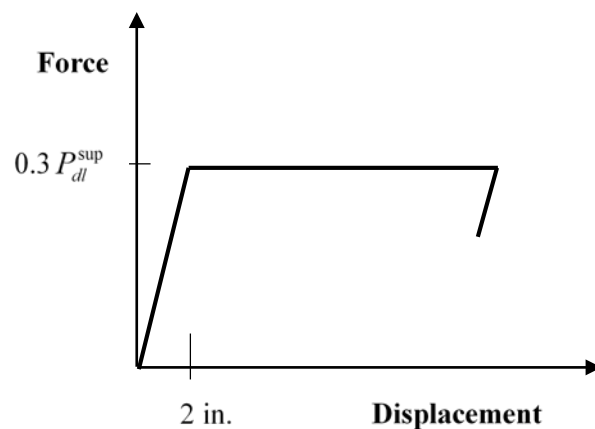


Figure 4.3.1-2 Abutment Transverse stiffness for Nonlinear Time History Analysis

4.3.2 Stand-Alone Models

A stand-alone or discrete frame model may be used to determine the upper bound to the displacement demand.

Each frame shall satisfy all applicable SDC requirements.

4.3.2.1 Transverse Stand-Alone Models

Transverse stand-alone frame models shall assume lumped mass at the top of bents or columns. Hinge spans shall be modeled as rigid elements with half of their mass lumped at the adjacent column as shown in Figure 4.3.2.1-1. The transverse analysis of end frames shall include an estimate of the abutment stiffness as specified in Section 6.3.2. The transverse displacement demand at each bent in a frame shall include the effects of rigid body rotation around the frame's center of rigidity.

4.3.2.2 Longitudinal Stand-Alone Models

Longitudinal stand-alone frame models shall include the short side of hinges with a concentrated dead load, and the entire long side of hinges supported by rollers at their ends as shown in Figure 4.3.2.2-1.

Abutment longitudinal stiffness shall be included as specified in Section 4.3.1.

C4.3.2

In situations where the software required for the analysis of a multi-frame bridge is unavailable or inadequate to analyze multi frame bridges, the bridge may be analyzed by performing stand-alone analysis on the individual frames.

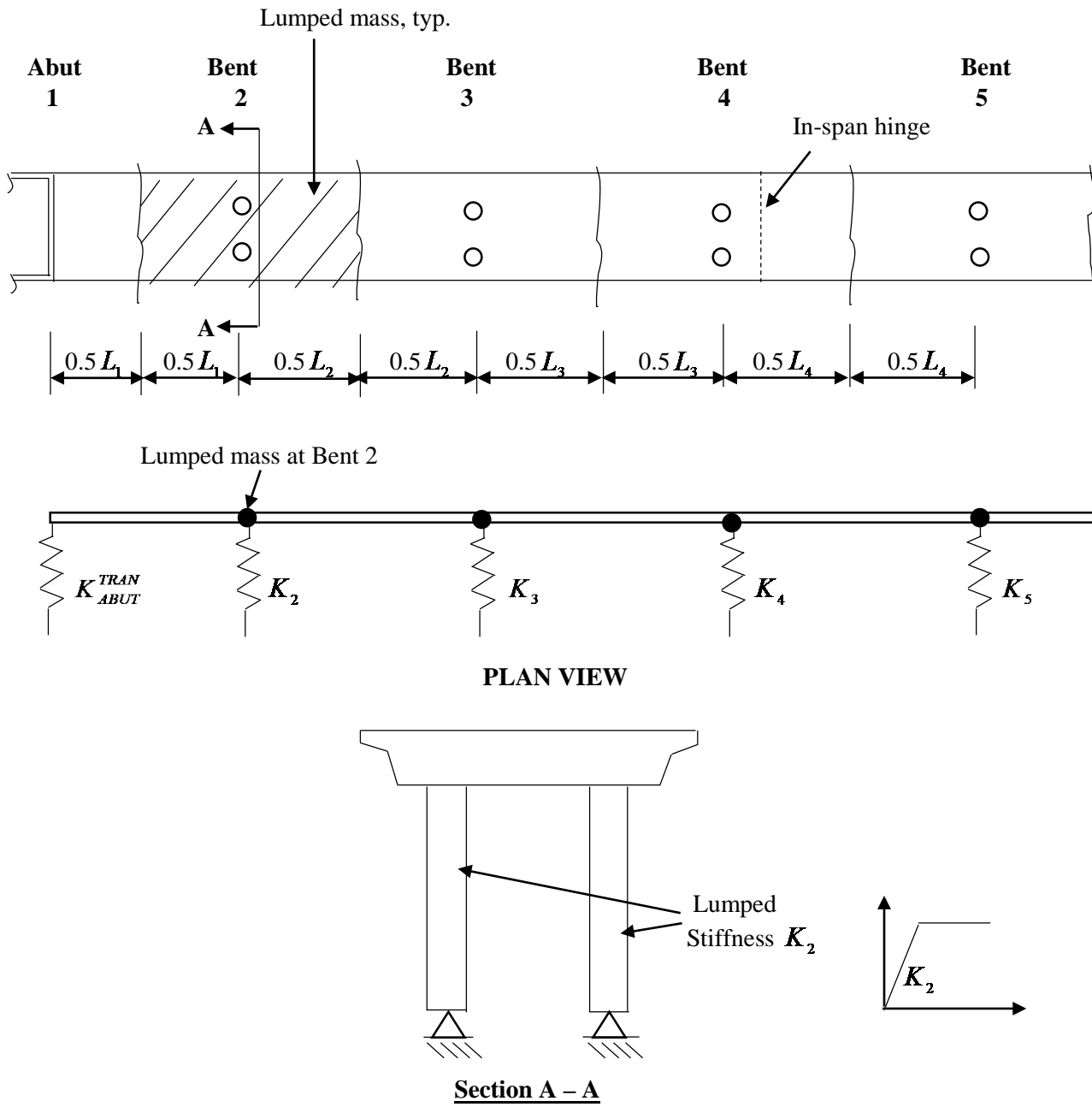
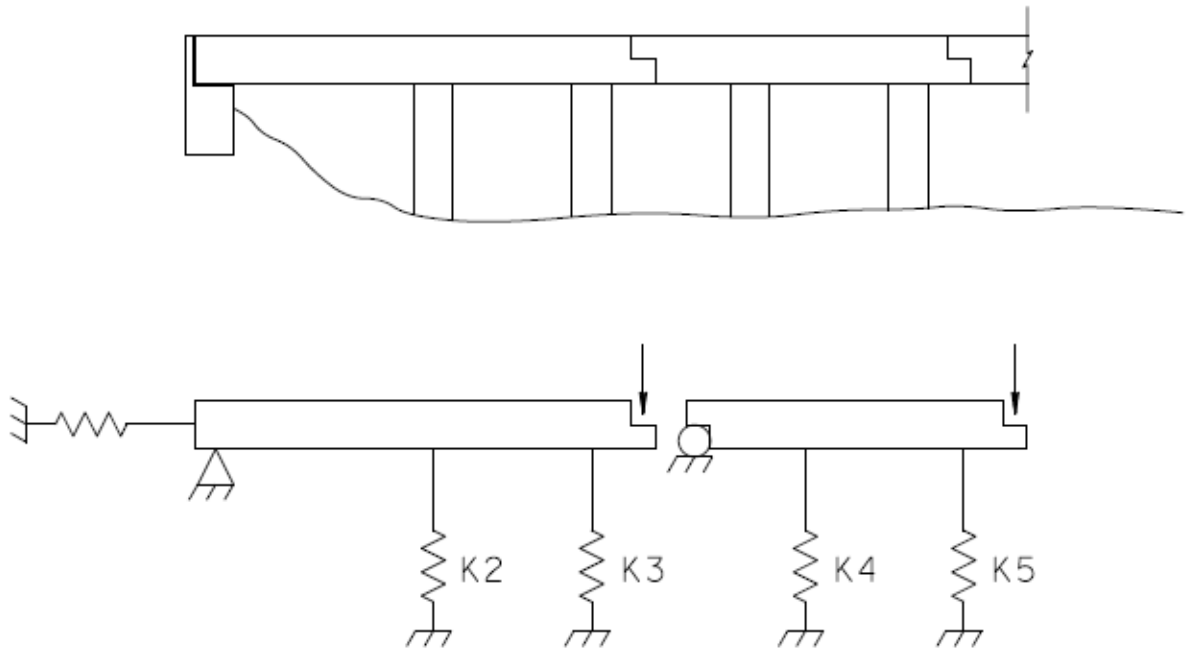


Figure 4.3.2.1-1 Transverse Stand-Alone Model

**Note:**

The abutment longitudinal stiffness is taken as zero when ESA and EDA methods of analysis are used for bridges with four or more frames.

Figure 4.3.2.2-1 Longitudinal Stand-Alone Model

4.4 DEFORMATION DEMANDS

4.4.1 Displacement Ductility Demand

Displacement ductility demand, μ_D shall be used to measure the ductility of a SCM. μ_D shall be defined as:

$$\mu_D = \frac{\Delta_D}{\Delta_{Y(i)}} \quad (4.4.1-1)$$

where:

Δ_D = frame or bent displacement demand in the local principal/critical axes of a member

$\Delta_{Y(i)}$ = frame or bent displacement at the instant a plastic hinge forms in the i-th SCM.

The displacement ductility demand values, μ_D for SCMs shall not exceed the values specified in Table 4.4.1-1.

For pile groups in Class S2 soil, plastic hinging shall not be permitted in the piles except at the top of piles if the pile-to-cap connection is fixed.

C4.4.1

The maximum displacement ductility demand values in Table 4.4.1-1 are based on laboratory testing of mostly single bridge columns, fixed at the base (Lehman and Moehle, 2000; Schoettler et al., 2015). Adjustments for multi-column bents and bridges affected by soil-foundation-structure interaction reflect the complexity of these systems.

The closer the μ_D value is to the maximum limit specified in Table 4.4.1-1, the more economical the design, and vice versa.

For pile groups with a pinned pile-to-cap connection in Class S2 soil, μ_D is limited to 1.0 because plastic hinging is not allowed in the piles.

Seismic critical members with flexible foundations will have low displacement ductility demands because of larger Δ_y . The minimum flexural strength requirement of Section 5.3.6.1 or the P - Δ requirements of Section 4.4.4 may govern the design of frames where foundation flexibility increases the period of the structure and lowers the design spectral acceleration.

It is recognized that as the framing system becomes more complex and boundary conditions are included in the demand model, an increased percentage of the calculated global displacement is significantly affected by the flexibility of components other than the SCMs within the frame. For such systems, including Type I shafts, global ductility demand values are usually well below the maximum values specified in Table 4.4.1-1. The value of μ_D for such members may not exceed 3.5.

Table 4.4.1-1 Displacement Ductility Demand Values, μ_D

Plastic Hinge Location	Seismic Critical Member	Limiting Displacement Ductility Demand		
		Ordinary Standard Bridge	Recovery Standard Bridge	
			SEE	FEE
Substructure	Column in a single-column bent supported on a footing or Type II shaft	4.0	2.5	1.5
	Column in a multi-column bent supported on a footing or Type II shaft	5.0	3.5	2.0
	Pile extensions or Type I shaft, plastic hinge at cap/superstructure soffit	5.0	3.5	2.0
Foundation	Pile extensions or Type I shaft, plastic hinge below ground	3.5	2.5	1.5
	Pile groups in Class S2 soil, fixed pile to cap connection, plastic hinge on top of pile	2.5	1.5	1.0
	Pile groups in Class S2 soil, pinned pile to cap connection	1.0	1.0	1.0
	Pile groups in Class S1 soil	*	*	*

* Lateral analysis not required for foundations in Class S1 soil. See Section C6.2.3.1.

4.4.2 Overstrength Demand

Capacity protected members shall be designed to resist the overstrength demands imparted by SCMs and sacrificial elements.

4.4.2.1 Overstrength Moment Demand

The overstrength moment of a SCM, M_o^{col} shall be taken as:

$$M_o^{col} = 1.2M_p^{col} \quad (4.4.2.1-1)$$

where:

M_p^{col} = idealized plastic moment capacity of a SCM

4.4.2.2 Overstrength Shear Demand

The shear demand transferred to adjacent capacity protected members by SCMs shall be the shear force, V_o^{col} associated with the overstrength moment, M_o^{col} . All potential plastic hinge locations shall be considered to determine the maximum possible shear demand.

4.4.3 Superstructure and Bent Cap Seismic Demand

4.4.3.1 Seismic Moment Demand

The superstructure and bent cap moment demands on the right and left sides of the superstructure-to-column joint along the longitudinal direction, M_D^R and M_D^L , respectively, shall be taken as (see Figure 4.4.3.1-1):

$$M_D^R = M_{dl}^R + M_{p/s}^R + M_{eq}^R \quad (4.4.3.1-1)$$

$$M_D^L = M_{dl}^L + M_{p/s}^L + M_{eq}^L \quad (4.4.3.1-2)$$

where:

M_D^R , M_D^L = moment demands on the right and left sides of superstructure-to-column joint, respectively

C4.4.2.1

The use of overstrength moments of SCMs to design capacity-protected members is intended to ensure that fusing occurs in the SCM as well as to account for:

- Unexpected material strength variations between the SCM and adjacent members
- Actual moment capacities greater than the idealized plastic moment capacity.

C4.4.2.2

The shear force associated with the overstrength moment, V_o^{col} may be determined from:

$$V_o^{col} = 1.2V_p^{col} \quad (C4.4.2.2-1)$$

where:

V_p^{col} = plastic shear corresponding to the idealized plastic moment capacity of a SCM

C4.4.3.1

The overstrength force demand in the superstructure corresponds to the maximum force demand in the earthquake resisting system. When the maximum force demand is reached during a seismic event, the following additional forces are present: dead loads, secondary forces from post-tensioning (i.e., secondary prestress effects).

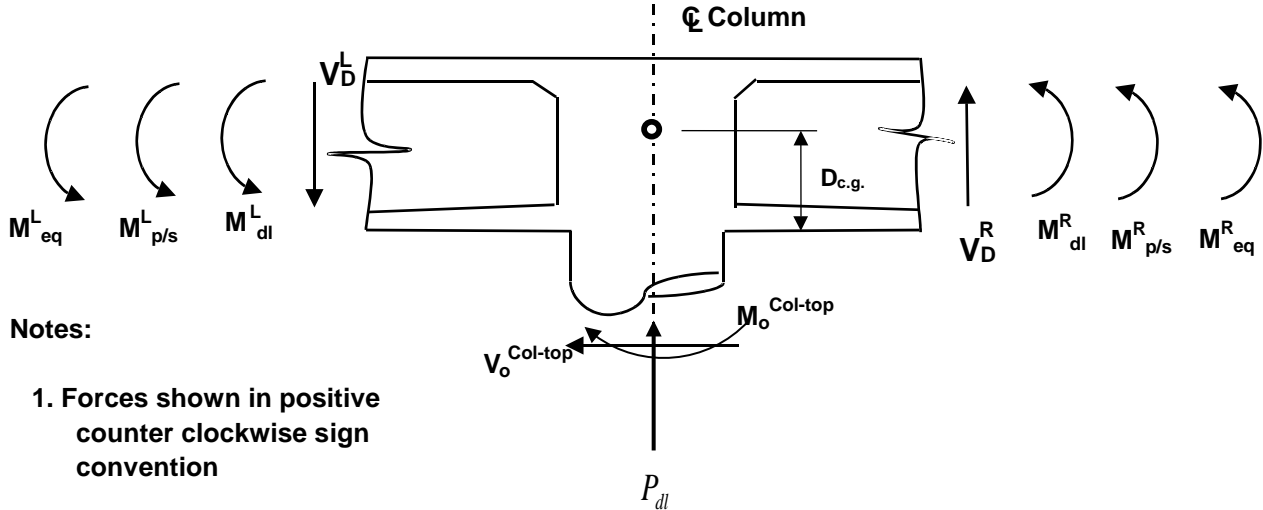


Figure 4.4.3.1-1 Superstructure Demand Generated by Column Overstrength Moment

M_{dl}^R, M_{dl}^L = dead load plus added dead load moments on the right and left sides of superstructure-to-column joint, respectively

$M_{p/s}^R, M_{p/s}^L$ = effective secondary prestress moments (after losses have occurred) on the right and left sides of superstructure-to-column joint, respectively

M_{eq}^R, M_{eq}^L = earthquake moments on the right and left sides of superstructure-to-column joint induced by $M_{eq}^{col-top}$ and $(V_{eq}^{col-top} \times D_{c.g})$

$D_{c.g}$ = distance from the top of column to the center of gravity of the superstructure/bent cap

The seismic moment on the top of column, $M_{eq}^{col-top}$ shall be determined as:

$$M_{eq}^{col-top} = M_o^{col-top} - M_{dl}^{col-top} - M_{p/s}^{col-top} \quad (4.4.3.1-3)$$

where:

Several analysis techniques may be used to distribute the column earthquake moment and shear demands to the superstructure. If a moment distribution or a frame analysis method is used, the superstructure can be modeled as a continuous beam. A moment of magnitude equal to $M_{eq}^{col-top} + (V_{eq}^{col-top} \times D_{c.g})$ associated with each column can be applied at each superstructure/column joint and then distributed to obtain the moments and shears in the superstructure.

Due to the uncertainty in the magnitude and redistribution of secondary prestress moments and shears ($M_{p/s}$ and $V_{p/s}$) at the extreme seismic limit state, these forces should be included in the superstructure when the overall demand is increased, and disregarded when the overall demand is decreased.

$M_o^{col-top}$ = column overstrength moment on the top of column

$M_{dl}^{col-top}$ = dead load plus added dead load moments on the top of column

$M_{p/s}^{col-top}$ = effective secondary prestress moment on the top of column

The seismic moment, $M_{eq}^{col-top}$ shall be distributed into the superstructure considering longitudinal bridge movement to the right and to the left.

Effective superstructure width as defined in Section 7.2.1.1 shall be used in computing the seismic demand distributed to the superstructure.

4.4.3.2 Seismic Shear Demand

The superstructure and bent cap shear demands on the right and left sides of the superstructure-to-column joint, V_D^R and V_D^L , respectively, shall be determined as:

$$V_D^R = V_{dl}^R + V_{p/s}^R + V_{eq}^R \quad (4.4.3.2-1)$$

$$V_D^L = V_{dl}^L + V_{p/s}^L + V_{eq}^L \quad (4.4.3.2-2)$$

where:

V_D^R, V_D^L = shear demands on the right and left sides of superstructure-to-column joint, respectively

V_{dl}^R, V_{dl}^L = dead load plus added dead load shears on the right and left sides of superstructure-to-column joint, respectively

$V_{p/s}^R, V_{p/s}^L$ = effective secondary prestress shears (after losses have occurred) on the right and left sides of superstructure-to-column joint, respectively

V_{eq}^R, V_{eq}^L = seismic shears on the right and left sides of superstructure-to-column joint induced by $M_{eq}^{col-top}$ and $V_{eq}^{col-top}$

The seismic shear on the top of column, $V_{eq}^{col-top}$ shall be determined as:

$$V_{eq}^{col-top} = V_o^{col-top} - V_{dl}^{col-top} - V_{p/s}^{col-top} \quad (4.4.3.2-3)$$

where:

$V_o^{col-top}$ = overstrength shear on the top of column

$V_{dl}^{col-top}$ = dead load plus added dead load shear on the top of column

$V_{p/s}^{col-top}$ = effective secondary prestress shear on the top of column

4.4.4 P-Δ Effects

The effects of gravity loads acting through lateral displacements shall be included in the design.

For SCMs meeting the ductility demand limits specified in Section 4.4.1, P-Δ effects may be ignored if Equation 4.4.4-1 is satisfied.

$$P_{dl} \Delta_r \leq 0.25 M_p^{col} \quad (4.4.4-1)$$

where:

P_{dl} = axial load attributed to dead load

Δ_r = relative lateral offset between the point of contra-flexure and the base of the plastic hinge, as shown in Figure 4.4.4-1. For Type I shafts:

$$\Delta_r = \Delta_D - \Delta_s \quad (4.4.4-2)$$

where:

Δ_s = shaft displacement at the point of maximum moment

C4.4.4

P-Δ effects can only be accurately captured with nonlinear time history analysis. In lieu of such analysis, Equation 4.4.4-1 is used to establish a conservative limit for lateral displacements induced by axial load.

The moment demand at the point of maximum moment in the shaft is shown in Figure 4.4.4-1. 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 4.4.4-1). The idealized plastic moment M_p is always less than M_u in a well-confined column.

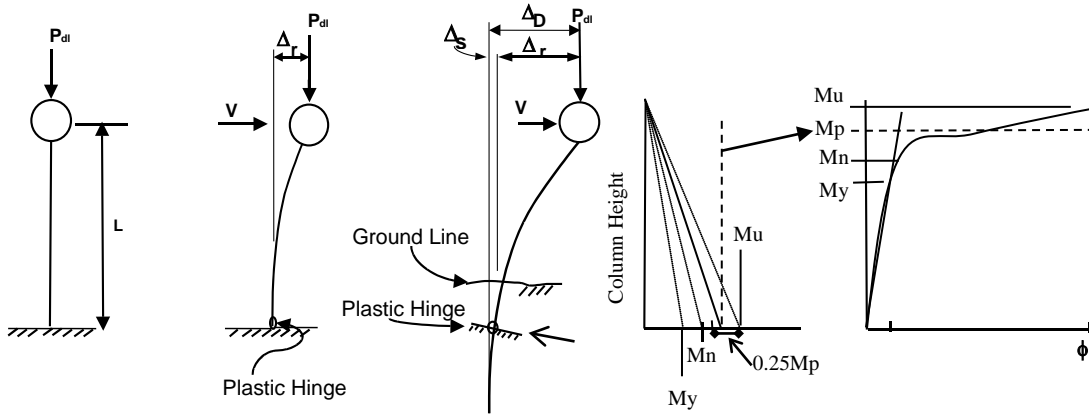


Figure 4.4.4-1 P-Δ Effects on Bridge Columns



SEISMIC DEFORMATION CAPACITIES

5.1 GENERAL

Deformation capacities of bridge members, frames, and bents shall be determined as specified in the following sections.

5.2 GLOBAL DISPLACEMENT CAPACITY

5.2.1 General

Global displacement capacity, Δ_C shall be determined along the directions specified in Section 3.5.2.

5.2.2 Inelastic Static Analysis

Except as provided herein, Inelastic Static Analysis (ISA) shall be used to determine the displacement capacity, Δ_C of a frame or bent, provided the seismic response of the structure is dominated by a single translational mode of vibration. Δ_C shall correspond to the lateral capacity of the bridge or bent when any plastic hinge reaches its ultimate curvature capacity, ϕ_u . ISA shall be performed with the dead load applied while a lateral static load or displacement is applied at the center of gravity of the superstructure or the bent cap. Nonlinear soil-foundation-structure interaction shall be taken into account by means of soil springs. Abutment stiffness shall be included, as specified in Sections 6.3.1 and 6.3.2.

For bridges where Equivalent Static Analysis (ESA) may be used to determine displacement demand as specified in Sections 4.2 and 4.2.1, “Local Displacement Capacity” equations may be used as the ISA.

C5.2.2

Inelastic Static Analysis is commonly referred to as “pushover” analysis. ISA is a piecewise linear analysis of a bridge frame or bent as successive plastic hinges form in the earthquake resisting system. ISA captures the overall nonlinear behavior of the earthquake resisting system by applying a lateral static load at the center of gravity of the superstructure or the bent, and monotonically increasing the load until the displacement capacity criterion specified in Section C3.5.1 is reached. Because the analytical model accounts for the redistribution of internal forces as plastic hinges form, ISA provides a more realistic seismic response of the bridge than elastic analysis.

Traditionally, Caltrans has used ISA to determine displacement capacity and ESA to determine displacement demand.

Local Displacement Capacity

Local displacement capacity, Δ_c is defined as a member’s displacement capacity attributed to its elastic flexibility, Δ_Y^{col} and its plastic hinge deformation capacity, Δ_p .

For the special case of single span bridges, the displacement capacity shall be greater than or equal to the larger of the minimum support length of the abutment as specified in Section 6.3.3 and the displacement demand as specified in Section 4.2.1.

The local displacement capacity of a member is based on its rotation capacity, which in turn is based on the curvature capacity of its plastic hinges. Δ_c is independent of the effects of adjacent members and subsystems.

Fixed-Free and Fixed-fixed SCMs are idealized as one and two cantilever segments, as shown in Figures C5.2.2-1 and C5.2.2-2. Δ_c is determined from:

$$\Delta_c = \Delta_Y^{col} + \Delta_p \quad (C5.2.2-1)$$

$$\Delta_Y^{col} = \frac{L^2}{3} \phi_Y \quad (C5.2.2-2)$$

$$\Delta_p = \theta_p \left(L - \frac{L_p}{2} \right) \quad (C5.2.2-3)$$

$$\theta_p = L_p \phi_p \quad (C5.2.2-4)$$

$$\phi_p = \phi_u - \phi_Y \quad (C5.2.2-5)$$

where:

L = distance from the point of maximum moment to the point of contra-flexure (in.)

L_p = equivalent plastic hinge length (in.)

Δ_p = idealized plastic displacement capacity due to rotation of the plastic hinge (in.)

Δ_Y^{col} = idealized yield displacement at the formation of the plastic hinge (in.)

ϕ_p = idealized plastic curvature capacity (assumed constant over L_p) (rad/in)

ϕ_u = curvature capacity when the concrete strain reaches ϵ_{cu} or the longitudinal reinforcing steel reaches the reduced ultimate strain ϵ_{su}^R (rad/in)

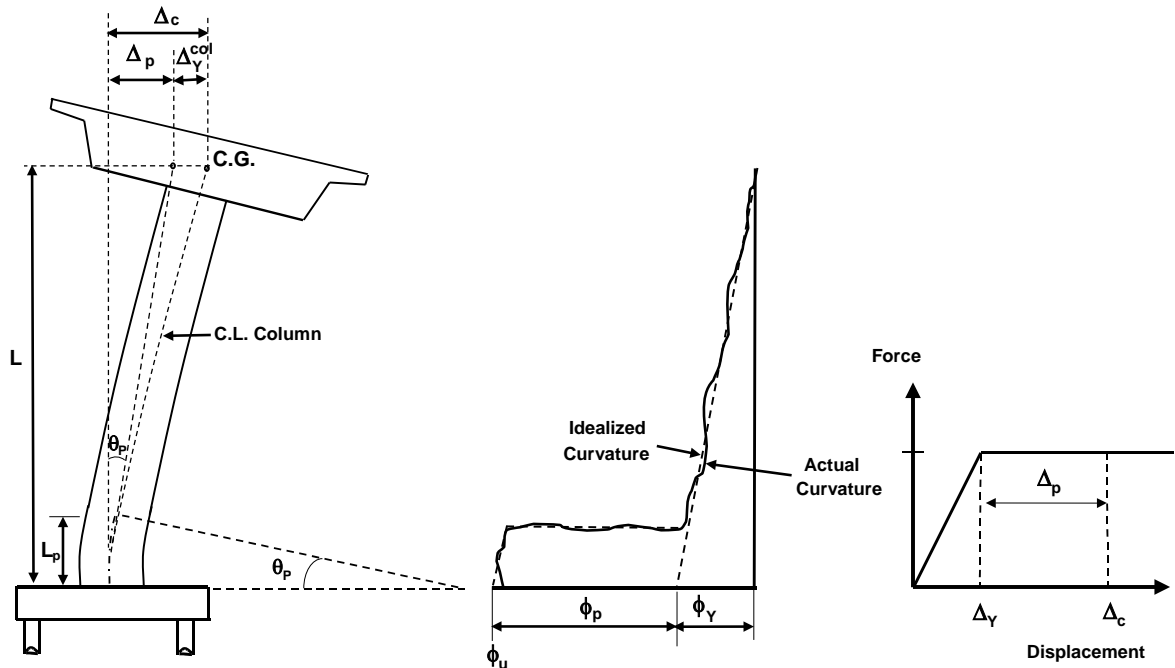


Figure C5.2.2-1 Local Displacement Capacity of a Typical Cantilever Column with Fixed Base

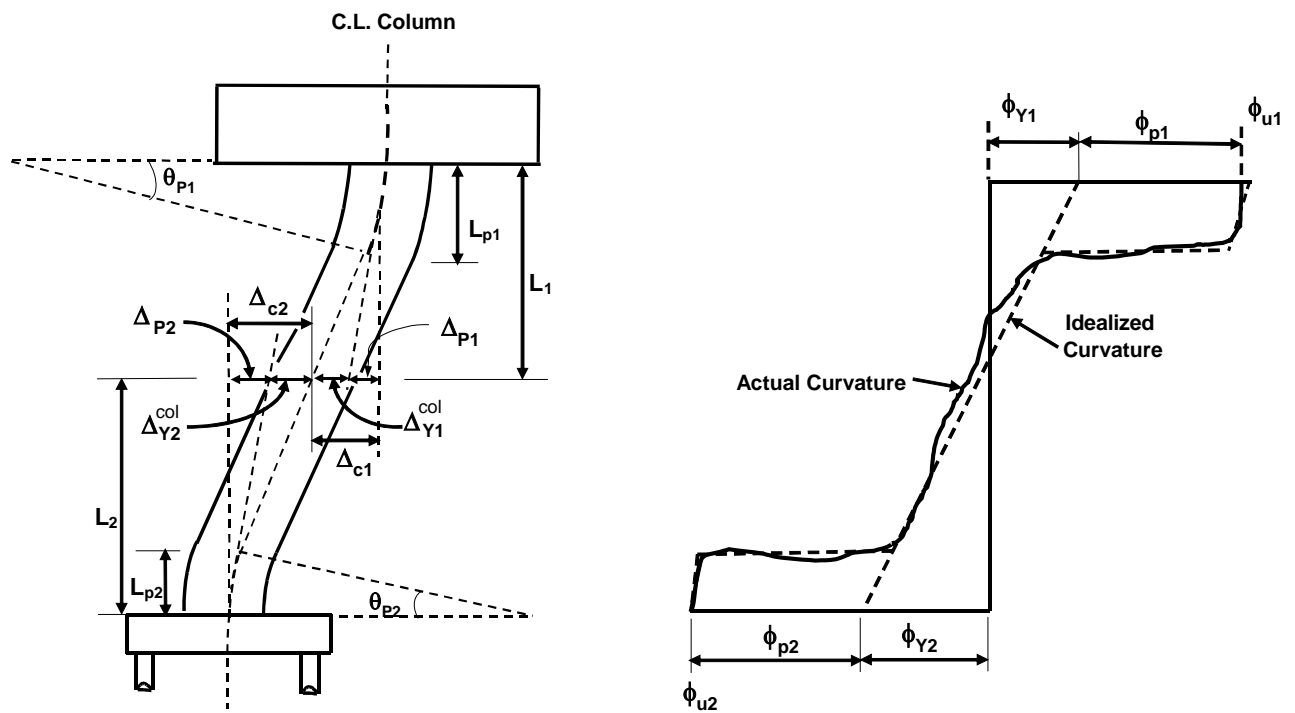


Figure C5.2.2-2 Local Displacement Capacity of a Typical Fixed-Fixed Column

ϕ_Y = idealized yield curvature defined by an elastic-perfectly-plastic M - ϕ curve of the cross section (rad/in)

θ_p = plastic hinge rotation capacity (radian)

For Type I shafts, Equation C5.2.2-3 becomes

$$\Delta_p = \theta_p L \quad (\text{C5.2.2-6})$$

where:

L is the portion of the shaft length measured from the center of the plastic hinge to the center of gravity of the superstructure. For fixed-fixed column, the dimensions L_1 and L_2 in Figure C5.2.2-2 should be substituted for L in Equations C5.2.2-2 and C5.2.2-3.

Traditionally, the local displacement ductility capacity of a SCM, μ_c has been defined as:

$$\mu_c = \frac{\Delta_c}{\Delta_Y} \quad (\text{C5.2.2-7})$$

Local displacement ductility capacity is based on an equivalent member that approximates a fixed base cantilever element as shown in Figure C5.2.2-3.

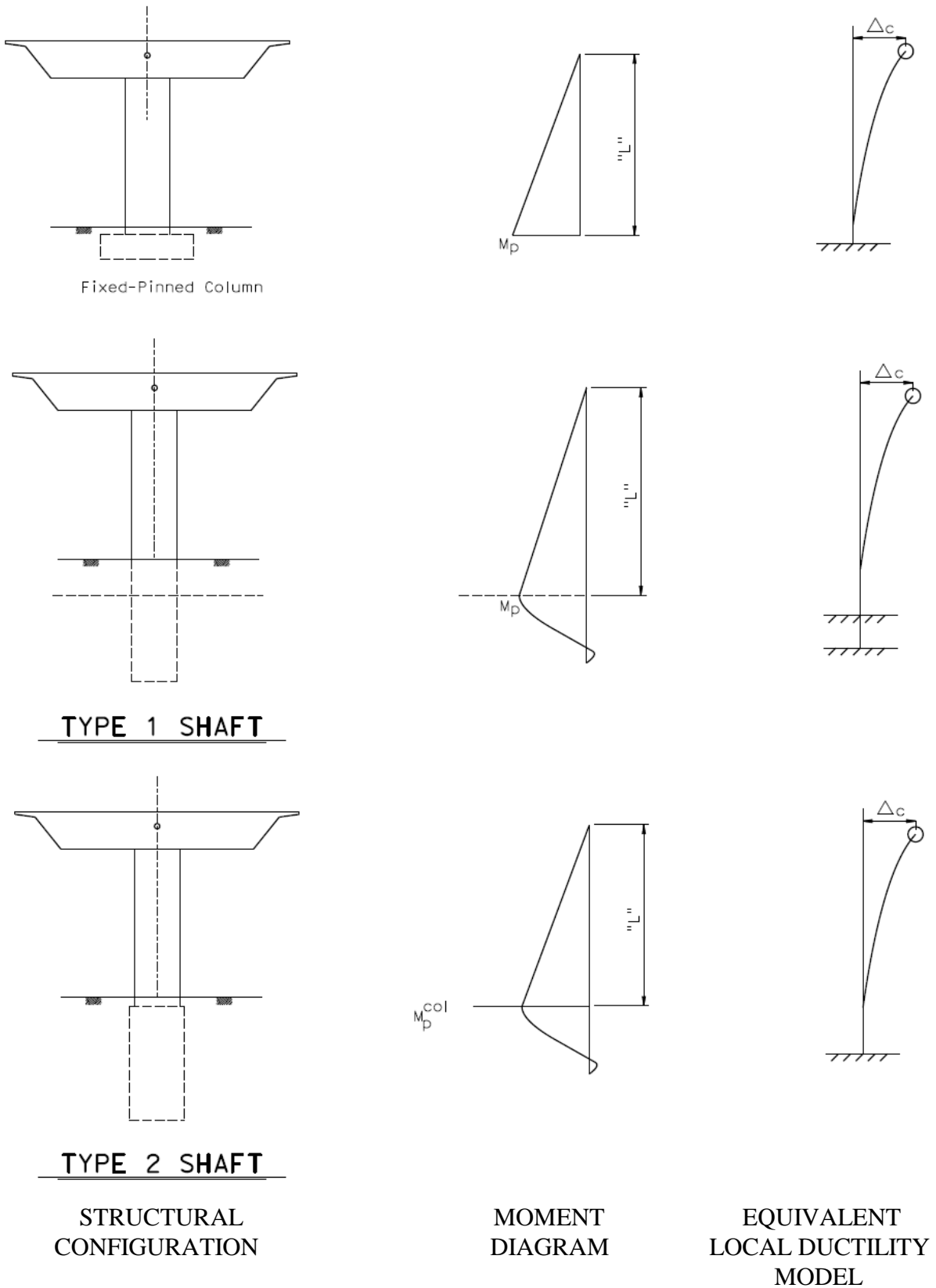


Figure C5.2.2-3 Local Ductility Assessment

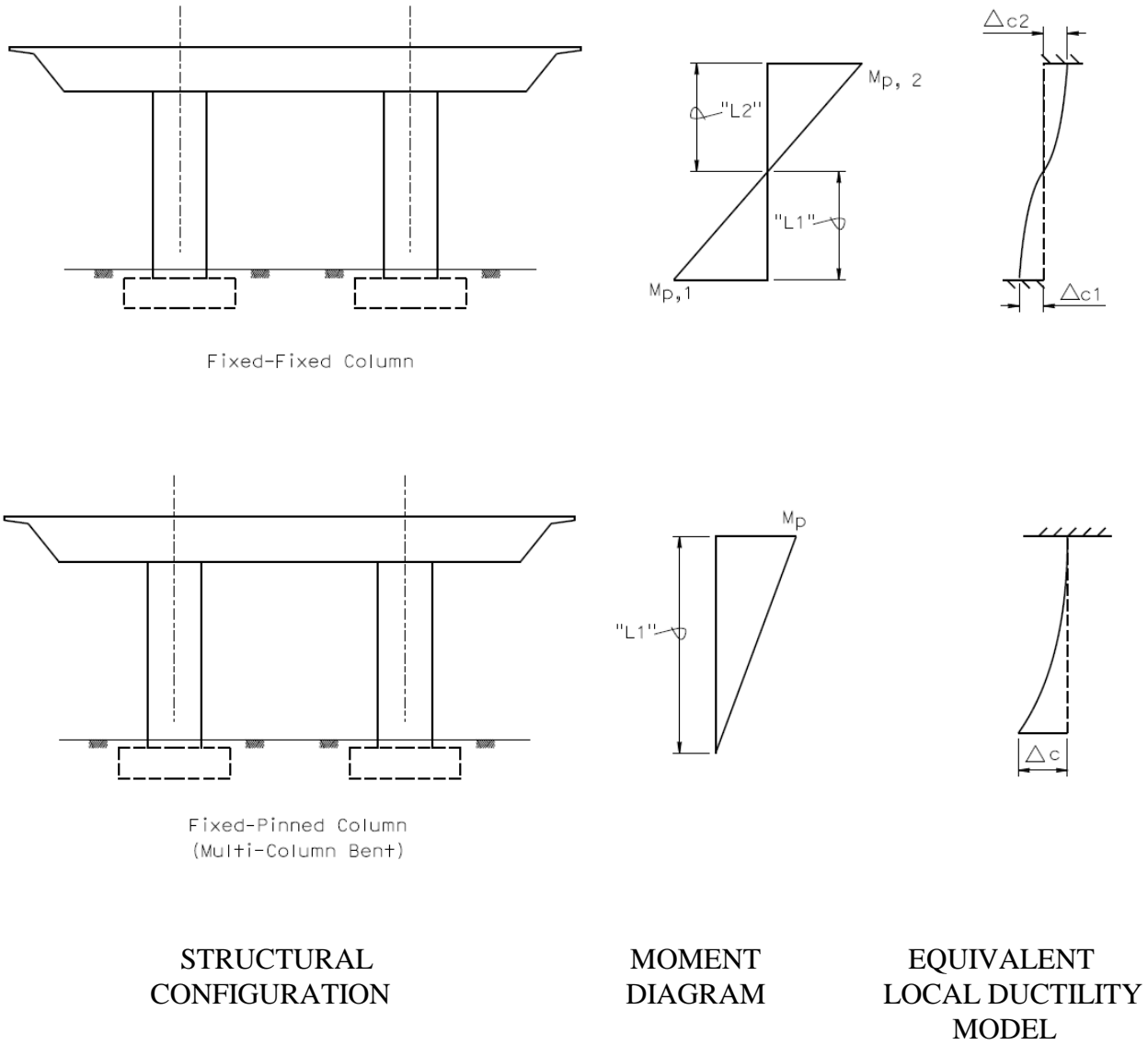


Figure C5.2.2-3 Local Ductility Assessment (continued)

5.3 SEISMIC CRITICAL MEMBERS

5.3.1 General

Columns, Type I shafts, Pile/Shaft groups in Class S2 soils, and pile-extensions shall be classified as seismic critical members (SCMs). Any member intended to function as a SCM as specified in Section 1.4 but not listed above shall require a PSDC.

Expected locations of plastic hinges in SCMs shall be designed and detailed for ductile response.

The requirement of ductile response shall apply to SCMs regardless of the seismicity level of the bridge site.

5.3.2 Plastic Hinge Region

The plastic hinge region, L_{pr} shall be taken as the larger of:

- 1.5 times the cross sectional dimension in the direction of bending
- The region of a SCM where the moment exceeds 75% of the maximum plastic moment, M_p^{col}
- $0.25 \times$ (Length of SCM from the point of maximum moment to the point of contra-flexure)

5.3.3 Axial Load Limits

The axial load ratio for SCMs in compression shall satisfy:

$$\rho_{dl} = \frac{P_{dl}}{f'_c A_g} \leq 0.15 \quad (5.3.3-1)$$

and

$$\rho_c = \frac{P_c}{f'_c A_g} \leq 0.22 \quad (5.3.3-2)$$

where:

C5.3.1

A non seismic critical member may be designed as a seismic critical member in order to meet a nonstandard design requirement. For example, if capacity protected members such as outrigger bent cap beams and “C” bent cap beams are expected to experience a moderate or significant damage, they should be designed and detailed as SCMs.

Ductile design and detailing of SCMs also applies in areas of low seismicity where the bridge response might be in the essentially elastic range.

C5.3.2

The plastic hinge region, L_{pr} is the region of the SCM that is expected to form a plastic hinge. Plastic hinge regions require enhanced lateral confinement.

C5.3.3

The analytical models describing the ductile response of SCMs have been calibrated by many column tests generally using a 10% axial load ratio.

Although tests with axial loads up to $0.35 f'_c A_g$ (Benzoni et al., 1996) have shown that the ductility capacity of confined columns will diminish with increasing axial loads, there are not enough data points to reliably establish the column ductile behavior under axial loads greater than $0.15 f'_c A_g$. It is expected that future research will extend the knowledge base beyond $0.15 f'_c A_g$ and to SCMs other than columns.

ρ_{dl} = axial load ratio due to dead load

ρ_c = axial load ratio due to dead load and overturning

P_{dl} = axial force due to dead load

A_g = gross cross section area of a SCM

P_c = axial force due to dead load and overturning

The value of f_c' for use in determining the axial load limits specified above shall not exceed 5000 psi.

5.3.4 Equivalent Plastic Hinge Length

The equivalent plastic hinge length, L_p shall be taken as:

Case (A)

- Plastic hinge at ends of columns supported on footings or Type II shafts
- Plastic hinge at the boundaries of steel pipe in columns/shafts with steel pipes (casing or CISS)

$$L_p = 0.08L + 0.15f_{ye}d_{bl} \geq 0.3f_{ye}d_{bl} \quad (5.3.4-1)$$

where:

f_{ye} = expected yield strength for ASTM A706 reinforcement (ksi)

d_{bl} = nominal bar diameter of longitudinal column reinforcement (in.)

L = length of a SCM from the point of maximum moment to the point of contraflexure (in.)

Case (B)

- Plastic hinge at the top of horizontally-isolated flared columns
- Plastic hinge at ends of steel-jacketed columns

$$L_p = G + 0.3f_{ye}d_{bl} \quad (5.3.4-2)$$

where:

In transverse frame analysis, axial forces in multi-SCM bents can vary from the dead load state when subjected to seismic overturning forces due to framing action of the cap beam. Overturning effects are negligible in longitudinal frame analysis and may be disregarded.

C5.3.4

The equivalent plastic hinge length, L_p is the length of a SCM over which the plastic curvature is assumed constant for estimating plastic rotation.

G = gap between the isolated flare and soffit of the bent cap, or the gap between the steel jacket and soffit of the bent cap or top of the footing (in.)

In Equation 5.3.4-2, f_{ye} and d_{bl} have the units of ksi and inches, respectively.

For columns flared only in one direction, Equation 5.3.4-1 shall be used for the “non-flared” direction.

Case (C)

- Plastic hinge in Type I shafts
- Plastic hinge occurring at least a distance, D_c away from boundaries of steel pipe in columns/shafts with steel pipes (casing or CISS)

$$L_p = D^* + 0.08H_{o-max} \quad (5.3.4-3)$$

where:

D^* = diameter for circular shaft or the least cross section dimension for oblong shafts

H_{o-max} = length of shaft/column from point of maximum moment to point of contraflexure above ground considering the plastic hinge at the point of maximum moment.

When vertical flare isolation is used in combination with horizontally isolated flare, the equivalent plastic hinge length shall be taken as the lesser of L_p calculated using Equations 5.3.4-1 and 5.3.4-2, where G shall be taken as the length from the bent cap soffit to the bottom of the vertical flare isolation region.

5.3.5 Confined Core

Seismic critical members shall be detailed with either a single solid circular core or multiple solid interlocking cores.

C5.3.5

Single circular cores or multiple interlocking cores with hoop transverse reinforcement provide confining pressure to the core concrete and an inward restraint against buckling of the longitudinal bars (Priestley et al., 1996, Mander et al., 1988b) and are used for both circular and rectangular shaped SCMs.

5.3.6 Flexural Capacity

5.3.6.1 Minimum Flexural Capacity

Each bent or frame shall have a minimum plastic moment capacity (based on expected material properties) to resist a lateral force of ten percent of the tributary weight of the bent or frame applied as a static lateral load at the center of gravity of the superstructure.

5.3.6.2 Moment-Curvature Analysis

The plastic moment capacity of all SCMs shall be determined by moment-curvature ($M - \phi$) analysis. The $M - \phi$ curve shall be idealized with an elastic perfectly plastic response as shown in Figure 5.3.6.2-1. The elastic portion of the idealized curve shall pass through the point marking the first reinforcing bar yield. The idealized plastic moment capacity shall be obtained by balancing the areas between the actual and the idealized $M - \phi$ curves beyond the first reinforcing bar yield point, as shown in Figure 5.3.6.2-1.

In Figure 5.3.6.2-1:

M_p^{col} = idealized plastic moment capacity of a SCM

M_y = moment of a SCM corresponding to the first reinforcing bar yield

ϕ_y = yield curvature corresponding to the first reinforcing bar yield

ϕ_y = idealized yield curvature

ϕ_u = ultimate curvature capacity

M_{ne} = nominal moment capacity based on expected material properties and a concrete strain of 0.003

For the transverse analysis of multi-column bents, additional column axial force due to overturning effects shall be considered when determining the plastic moment capacity of the columns.

C5.3.6.2

Moment curvature curve is derived from the curvatures associated with a range of moments for a cross section based on the principles of strain compatibility and equilibrium of forces.

$M - \phi$ curves are used to represent material inelasticity either in concentrated zero length plastic hinges or in distributed plastic zones.

Although ultimate displacements reported in laboratory tests are traditionally based on the state when a 20 % reduction in the maximum lateral resistance is attained (Park and Paulay, 1975; Priestley et al. 1996), the ultimate curvature reported by Caltrans $M - \phi$ curve should not be reduced by 20%. The $M - \phi$ analysis is based on Mander's stress-strain model (Mander et al., 1988a, 1988b) in which the ultimate compressive strain of concrete was defined as the strain corresponding to the first fracture of the confinement steel. Tests results (Saini and Saiidi, 2014) have shown that ultimate displacements based on main reinforcing bar fracture and damage to the core concrete are about 20% greater than those obtained using the theoretical Mander's failure criterion.

In transverse frame analysis, axial forces in multi-SCM bents can vary from the dead load state when subjected to seismic overturning forces due to framing action of the cap beam. Overturning effects are negligible in longitudinal frame analysis and may be disregarded.

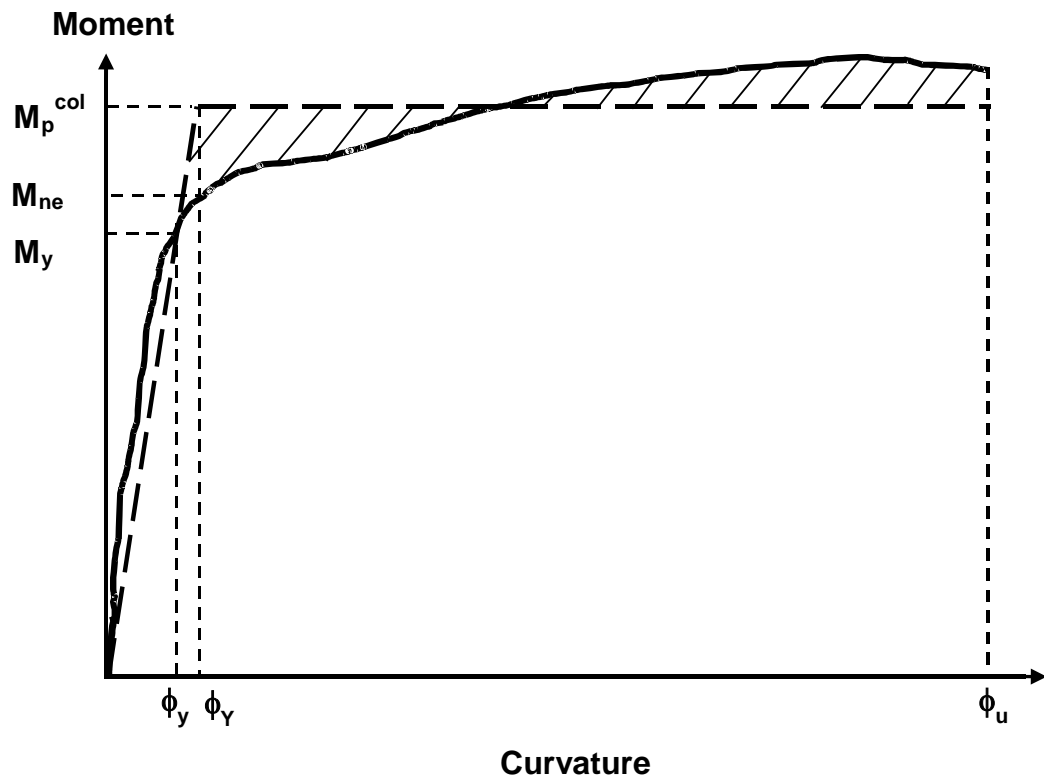


Figure 5.3.6.2-1 Moment - Curvature Curve

5.3.7 Shear Capacity

5.3.7.1 General

The shear capacity of SCMs shall satisfy:

$$\phi V_n \geq V_o^{col} \quad (5.3.7.1-1)$$

$$V_n = V_c + V_s \quad (5.3.7.1-2)$$

where:

V_n = nominal shear capacity (kip)

ϕ = resistance factor for shear = 1.0

V_o^{col} = overstrength shear associated with the overstrength moment of a SCM, as specified in Section 4.4.2.2

V_c = nominal shear capacity provided by concrete

V_s = nominal shear capacity provided by shear reinforcement

5.3.7.2 Nominal Concrete Shear Capacity C5.3.7.2

The nominal concrete shear capacity for SCMs, V_c shall be taken as:

$$V_c = v_c A_e \quad (5.3.7.2-1)$$

where:

$$A_e = 0.8A_g \quad (5.3.7.2-2)$$

- Inside the plastic hinge region

$$v_c = (F1)(F2)\sqrt{f'_c} \leq 4\sqrt{f'_c} \quad (5.3.7.2-3)$$

- Outside the plastic hinge region

$$v_c = 3(F2)\sqrt{f'_c} \leq 4\sqrt{f'_c} \quad (5.3.7.2-4)$$

$$0.3 \leq F1 = \frac{\rho_s f_{yh}}{0.15} + 3.67 - \mu_d \leq 3 \quad (5.3.7.2-5)$$

In Equation 5.3.7.2-5, the value of “ $\rho_s f_{yh}$ ” shall be limited to 0.35 ksi.

$$F2 = 1 + \frac{P_c}{2000A_g} < 1.5 \quad (5.3.7.2-6)$$

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

In Equations 5.3.7.2-3 to 5.3.7.2-6:

v_c = permissible concrete shear stress (psi)

f'_c = specified compressive strength of unconfined concrete (psi)

Figure C5.3.7.2-1 shows how the value of F1 varies over a range of displacement ductility demand ratios, μ_d .

The volumetric ratio of transverse reinforcement, ρ_s is defined in Section 5.3.8.2.

f_{yh} = specified minimum yield strength of transverse reinforcement (ksi)

ρ_s = volumetric ratio of transverse reinforcement

P_c = axial force on a SCM including the effects of overturning (lb)

A_g = gross cross sectional area of SCM (in.²)

$$\mu_d = \text{local displacement ductility demand} = \frac{\Delta_d}{\Delta_{Y(i)}}$$

Δ_d may be obtained by subtracting the displacements due to foundation and bent cap flexibilities from the global displacement, Δ_D as illustrated in Figures 3.5.1-2 and 3.5.1-3. The global displacement ductility demand, μ_D may be used in lieu of μ_d , provided a significant portion of the global displacement is attributed to the deformation of the SCM.

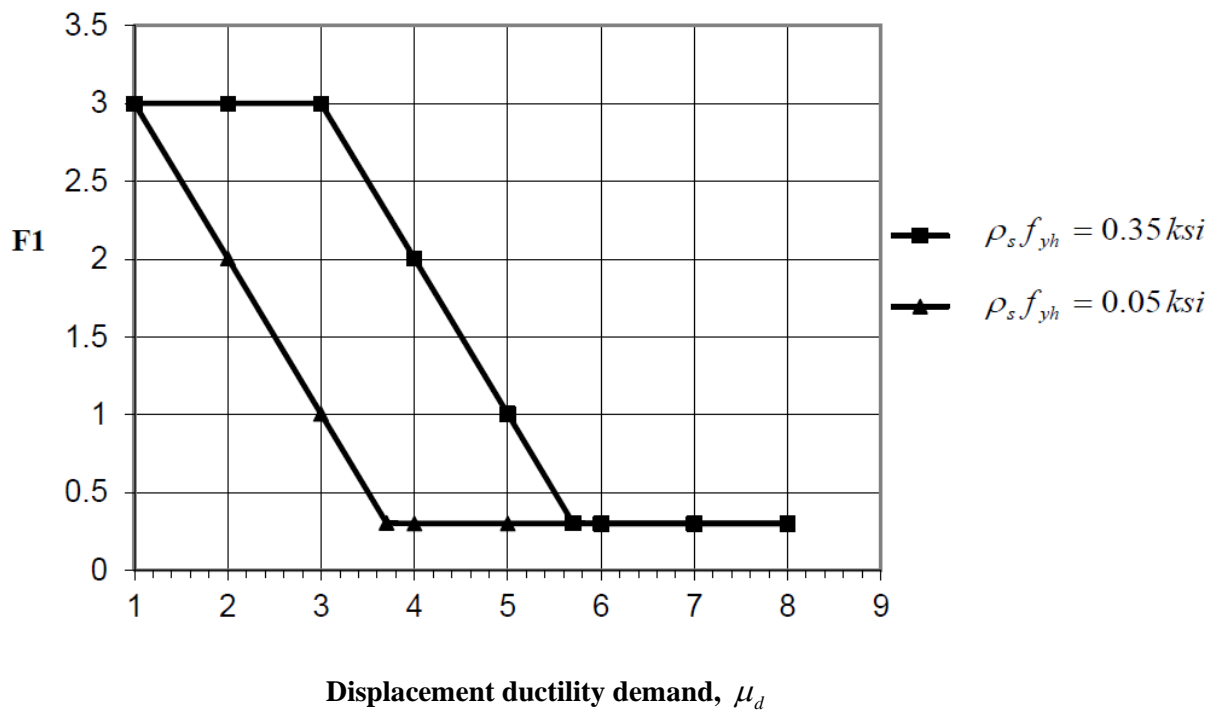


Figure C5.3.7.2-1 Concrete Shear Factors

5.3.7.3 Nominal Shear Reinforcement Capacity

The nominal shear reinforcement capacity for confined circular or interlocking core sections, V_s shall be taken as:

$$V_s = \left(\frac{A_v f_{yh} D'}{s} \right) \quad (5.3.7.3-1)$$

where:

$$A_v = n \left(\frac{\pi}{2} \right) A_b \quad (5.3.7.3-2)$$

A_v = area of shear reinforcement perpendicular to the flexural tension reinforcement (in.²)

n = number of individual interlocking spirals or hoops in the core of the sections

D' = cross-sectional dimension of confined concrete core measured between the centerlines of the peripheral hoop or spiral (in.)

s = spacing of shear reinforcement (in.)

A_b = area of individual reinforcing steel bar (in.²)

5.3.7.4 Maximum Shear Reinforcement

The nominal shear capacity provided by the reinforcing steel, V_s shall satisfy:

$$V_s \leq 8\sqrt{f'_c} A_e \quad (5.3.7.4-1)$$

where, f'_c and V_s have the units of psi and lb, respectively.

5.3.7.5 Minimum Shear Reinforcement

The area of shear reinforcement for each individual core of SCMs confined by hoops/spirals or interlocking hoops/spirals shall satisfy:

$$A_v \geq 0.025 \frac{D' s}{f_{yh}} \quad (5.3.7.5-1)$$

5.3.8 Transverse Reinforcement Requirements

5.3.8.1 General

Transverse reinforcement shall be provided in SCMs to satisfy plastic shear and curvature demands as specified in Sections 5.3.7.1, 3.5.1, and 4.4.1.

Transverse reinforcement shall be either ultimate butt-spliced hoops or continuous spiral (where allowed) as specified in Section 8.2.2.3.

Combination of spiral reinforcement with hoops shall not be used in a SCM.

C5.3.8.1

Sufficient amount of transverse reinforcement (i.e., confinement and shear reinforcement) is provided in SCMs in order to meet plastic shear demands, curvature demands resulting from imposed displacements, and to prevent buckling of the longitudinal compression reinforcement (Mander et al., 1988a).

In large diameter SCMs, hoops are preferred over spirals because of the larger bars and larger spacing used in hoops to facilitate construction. The discrete nature of hoops provides an advantage in SCMs since the failure of one hoop does not lead to a premature plastic hinge failure. When a plastic hinge forms, the confinement steel is exposed as a result of loss of cover concrete. With additional deformation, the strain in the transverse reinforcement increases. Any break at a single location in spiral reinforcement may cause a considerable length of the spiral to become ineffective and lead to plastic hinge failure. Also, the process of sampling and testing hoop splices for QA/QC purposes is easier than that of spiral splices.

Refer to Section 8.2.1 for types of reinforcing bar splices.

5.3.8.2 Minimum Transverse Reinforcement Inside the Plastic Hinge Region

For Ordinary Standard bridges, the minimum transverse reinforcement in terms of volumetric ratio, $\rho_{s,min}$ provided in the plastic hinge region shall be as specified in Table 5.3.8.2-1. SCMs with parameters outside those specified in Table 5.3.8.2-1 shall satisfy a minimum local displacement ductility capacity of 3.0.

For Recovery Standard bridges, $\rho_{s,min}$ shall be taken as 0.01.

C5.3.8.2

For SCMs with circular or interlocking core sections, the transverse reinforcement volumetric ratio, ρ_s is given by:

$$\rho_s = \frac{4A_b}{D' s} \quad (C5.3.8.2-1)$$

where:

A_b = area of individual confinement steel bar (in.²)

D' = cross-sectional dimension of confined concrete core measured between the centerlines of the peripheral hoop or spiral (in.)

s = spacing of transverse reinforcement measured along the longitudinal axis of the SCM (in.)

Table 5.3.8.2-1 Minimum Transverse Reinforcement Volumetric Ratio (Ordinary Standard Bridges)

Aspect Ratio, L/D_c	Diameter of SCM, D_c (ft.)	Longitudinal Reinforcement Ratio, ρ_l (%)	Axial Load Ratio, ρ_{dl} (%)	Minimum Transverse Reinforcement, $\rho_{s,min}$
≤ 8.0	$3 \leq D_c \leq 6$	≤ 2.3	≤ 10	0.006
			$10 < \rho_{dl} \leq 15$	0.007
	$6 < D_c \leq 11$	≤ 2.15	≤ 10	0.007
			$10 < \rho_{dl} \leq 15$	0.008

For the analysis of bridges with rectangular-shaped SCMs having ties and cross ties, ρ_s is given by:

$$\rho_s = \frac{A_v}{D_c' s} \quad (C5.3.8.2-2)$$

where:

A_v = sum of area of the ties and cross ties running in the direction perpendicular to the axis of bending (in.²)

D_c' = confined column cross-section dimension, measured out-to-out of ties, in the direction parallel to the axis of bending (in.).

The percent longitudinal reinforcement ratio, ρ_l is given by:

$$\rho_l = (100A_{st} / A_g) \quad (C5.3.8.2-3)$$

The axial load ratio due to dead load, ρ_{dl} is specified in Section 5.3.3.

A minimum volumetric ratio of transverse reinforcement, ρ_s ensures adequate confinement and dependable rotational capacity in the plastic hinge regions regardless of the displacement demand imparted to the SCM.

In previous editions of the *SDC*, a minimum local displacement ductility capacity of 3.0 was specified. The minimum volumetric ratios of transverse reinforcement specified in Table 5.3.8.2-1 were calibrated (Unanwa and Mahan, 2018) to provide a minimum local displacement ductility capacity equal to or greater than 3.0. The minimum value of ρ_s of 0.01 for Recovery Standard bridges is intended to meet the enhanced performance criteria for Recovery bridges specified in Section 1.3, based on previous bridge performance experience.

For SCMs with parameters outside those specified in Table 5.3.8.2-1, a minimum local displacement ductility capacity of 3.0 may be difficult to achieve. In such cases, an acceptable minimum value of ρ_s may be established following the PSDC procedure.

5.3.8.3 Transverse Reinforcement Outside the Plastic Hinge Region

The volume of transverse reinforcement required outside the plastic hinge region, shall be greater than or equal to 50 % of the transverse reinforcement provided within the plastic hinge region.

5.3.8.4 Transverse Reinforcement for Columns Supported on Type II Shafts

The volumetric ratio of transverse reinforcement for columns supported on Type II shafts shall satisfy the requirements specified in Sections 5.3.8.2 and 5.3.8.3.

5.3.8.5 Transverse Reinforcement for Type I Shafts with Larger Shaft Reinforcement

Type I shafts designed with larger longitudinal reinforcement below the ground (i.e., shaft) than that used above the ground (i.e., column) shall have a volumetric ratio of transverse reinforcement in the top $4D_{c,max}$ segment of the shaft greater than or equal to 75 % of the transverse reinforcement required at the base of the column.

5.3.9 Longitudinal Reinforcement Requirements

5.3.9.1 Maximum Longitudinal Reinforcement

The maximum area of longitudinal reinforcement in a SCM, $A_{st,max}$ shall be taken as:

$$A_{st,max} = 0.04A_g \quad (5.3.9.1-1)$$

5.3.9.2 Minimum Longitudinal Reinforcement

The minimum area of longitudinal reinforcement in a SCM, $A_{st,min}$ shall be taken as:

$$A_{st,min} = 0.01A_g \quad (5.3.9.2-1)$$

5.3.9.3 Minimum Size of Longitudinal Reinforcement within Interlocking Hoops

The longitudinal bar size in the interlocking portion of a SCM (i.e., “B” bars in Figure 5.3.9.3-1) shall be chosen to correspond to the longitudinal bar size outside the interlocking portion (i.e., “A” bars in Figure 5.3.9.3-1) as shown in Table 5.3.9.3-1.

The longitudinal reinforcing bars in the interlocking portion of a SCM shall have a maximum lateral spacing of 12 inches and need not be anchored in the footing or the bent cap unless the bars are required for the flexural capacity of the SCM.

Table 5.3.9.3-1 Minimum Longitudinal Bar Sizes within Interlocking Hoops

Bar size used outside the interlocking portion (A bars)	Minimum bar size required inside the interlocking portion (B bars)
#10	#6
#11	#8
#14	#9
#18	#11

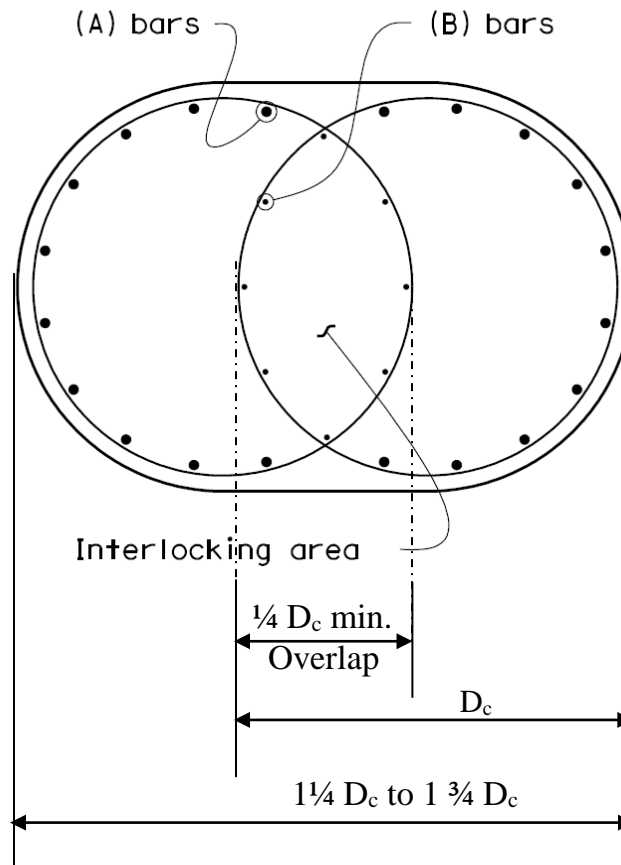


Figure 5.3.9.3-1 Longitudinal Reinforcement within Interlocking Hoops

5.4 CAPACITY PROTECTED MEMBERS

5.4.1 General

Footings, Type II shafts, bent cap beams, beam-column joints, superstructures, pile/shaft groups in Class S1 soil, abutment stemwalls, abutment seats, seismic expansion joints, in-span hinge seats, and column/shaft isolation casings shall be classified as capacity protected members (CPMs).

Capacity protected members shall be designed to remain essentially elastic when subjected to the overstrength moment and shear demands of adjoining SCMs and sacrificial elements.

C5.4.1

Maintaining essentially elastic behavior does not guarantee that the bridge will provide immediate or limited service after a seismic event.

Capacity protected members are designed and detailed to resist the overstrength moment and shear of the adjoining seismic critical members, even in locations of low seismicity where the bridge might respond elastically to the design seismic hazards.

Capacity-protected members should have some ductility to provide insurance against unexpected propagation of damage from the adjoining SCMs.

The overstrength moment and shear demands of SCMs are defined in Sections 4.4.2.1 and 4.4.2.2.

5.4.2 Flexural Capacity

The flexural capacity of CPMs shall be taken as the nominal moment capacity based on expected material properties, M_{ne} multiplied by the resistance factor of 1.0 for flexure.

M_{ne} shall be determined by either $M-\phi$ analysis as specified in Section 5.3.6.2, or nominal flexural resistance (at Strength limit state) as specified in AASHTO-CA BDS using expected material properties.

M_{ne} using $M-\phi$ analysis shall be based on 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.

5.4.3 Shear Capacity

The shear capacity of CPMs shall be taken as the nominal shear capacity, V_n multiplied by the resistance factor of 1.0 for shear.

The nominal shear capacity shall be calculated in accordance with AASHTO-CA BDS provisions using the specified minimum material properties.

5.4.4 Superstructure and Bent Cap Seismic Capacity

The capacity of the bent cap and superstructure shall be greater than the demand distributed to it on each side of the SCM by the largest combination of dead load moment, secondary prestress moment, and column earthquake moment, as shown in Equations 5.4.4-1 to 5.4.4-4.

$$\phi M_{ne}^{\text{sup}(R)} \geq M_{dl}^R \pm M_{p/s}^R + M_{eq}^R \quad (5.4.4-1)$$

$$\phi M_{ne}^{\text{sup}(L)} \geq M_{dl}^L \pm M_{p/s}^L + M_{eq}^L \quad (5.4.4-2)$$

$$\phi V_n^{\text{sup}(R)} \geq V_{dl}^R \pm V_{p/s}^R + V_{eq}^R \quad (5.4.4-3)$$

$$\phi V_n^{\text{sup}(L)} \geq V_{dl}^L \pm V_{p/s}^L + V_{eq}^L \quad (5.4.4-4)$$

Bent cap reinforcement required for overstrength should be developed beyond the column-to-bent cap joint. Cutting off bent cap reinforcement is discouraged because small changes in the plastic hinge capacity may translate into large changes in the moment distribution along the cap due to steep moment gradients.

where:

$M_{ne}^{sup(R)}$ = nominal moment capacity of the adjacent right superstructure span based on expected material properties and a concrete strain of 0.003

$M_{ne}^{sup(L)}$ = nominal moment capacity of the adjacent left superstructure span based on expected material properties and a concrete strain of 0.003

$V_n^{sup(R)}$ = nominal shear capacity of the adjacent right superstructure span

$V_n^{sup(L)}$ = nominal shear capacity of the adjacent left superstructure span

ϕ = resistance factor = 1.0 for both flexure and shear

The strength of the superstructure should not be considered effective on the side of the column adjacent to a hinge seat.

The effective superstructure width and effective bent cap width as specified in Sections 7.2.1.1, 7.3.1 and 7.3.2 shall be used in determining the seismic capacity.

The strength of the superstructure on the side of the column adjacent to a hinge seat may be considered effective by explicitly modeling that portion in the analysis program, including the load acting on it.

5.4.5 Transverse Reinforcement for Type II Shafts

The minimum volumetric ratio of transverse reinforcement required for a Type II shaft rebar cage between the top of the shaft and the termination of the column cage shall be as specified in Table 5.4.5-1.

C5.4.5

The additional confinement in the top 2 feet of the shaft is required to prevent bond failure and control the opening of tensile splitting cracks in the top of the shaft (Murcia-Delso et al., 2013). The experimental investigation of Murcia-Delso et al. (2013) also shows that structural steel casing can effectively control tensile splitting cracks in the top of the shaft.

If the transverse reinforcement provided in the upper portion of Type II shafts between the top and the termination of the column cage results in transverse reinforcement spacing which violates minimum spacing requirements in the shaft, the bar size and spacing may be increased, or the bars may be bundled and the spacing increased.

Table 5.4.5-1 Minimum Volumetric Ratio of Transverse Reinforcement in Type II Shafts

Location	Minimum Volumetric Ratio of Transverse Reinforcement	
	CISS or CIDH with permanent casing	All other shafts
Top 2 feet of shaft	50 % of that required at the base of the column	Same as that required at the base of the column
Between bottom end of column cage and 2ft below top of Type II shaft		50 % of that required at the base of the column



FOUNDATIONS, ABUTMENTS, AND SOIL-FOUNDATION-STRUCTURE INTERACTION

6.1 SOIL PROFILE CLASSIFICATION

6.1.1 General

Soil shall be classified as either Class S1 or Class S2, depending on its properties as specified in the following sections.

6.1.2 Class S1 Soil

Soils with all the following characteristics shall be classified as Class S1:

- Standard penetration test, $(N_1)_{60} \geq 30$
(Granular soils)
- Undrained shear strength, $s_u > 2000$ psf
(Cohesive soils)
- Shear wave velocity, $v_s > 886$ ft/sec
- Not susceptible to liquefaction, lateral spreading, or scour

where:

$(N_1)_{60}$ = penetration resistance corrected for overburden pressure and hammer efficiency

C6.1.1

Foundation soils combined with the structural components (i.e., piles, footings, pile caps and drilled shafts) and the seismic input loading determine the dynamic response of the foundation subsystem. Typically, the soil response has a significant effect on the overall foundation system response. Therefore, the anticipated foundation subsystem response can be based on the characteristics of the soil within the foundation's zone of influence.

The soil classification adopted in this edition of the SDC is related to that used in previous editions as follows:

- Class S1 - represents competent soils.
- Class S2 represents non-competent soils, including marginal soil, poor soil, soft soil, potentially liquefiable soil and soil susceptible to lateral spreading.

C6.1.2

$(N_1)_{60}$ is the penetration resistance in terms of blow count corrected to 60% of the theoretical free fall energy in accordance with ASTM D1586-11 (ASTM, 2011) and corrected for overburden pressure of 2 ksf.

In order to allow for the simplified design method for piles specified in Section 6.2.3.2, piles shall satisfy the required embedment in Class S1 soil specified in Section 6.2.3.1.

6.1.3 Class S2 Soil

Any soil that does not satisfy the requirements of Class S1 shall be classified as “Class S2.”

Lateral analysis as specified in Section 6.2.4.2 shall be required for foundations in Class S2 soils.

C6.1.3

The lateral and vertical force-deformation responses of Class S2 soil have a significant effect on the foundation response and subsequently on the overall response of the bridge. Therefore, lateral analysis is considered in the design.

6.2 FOUNDATIONS

6.2.1 General

Bridge foundations, including piles, shafts, and footings (pile cap and spread footings) shall be designed to resist seismic loading in accordance with the seismic performance criteria specified in Section 1.3. Foundation components classified as seismic critical members or capacity protected members in accordance with Sections 5.3.1 and 5.4.1, respectively, shall satisfy all applicable requirements of the SDC.

6.2.2 Footings

6.2.2.1 General

The structural design of footings shall include flexure, one-way shear, and two way-shear.

Flexural and shear demands shall be based on the axial load, column overstrength moment, and the associated shear. Flexure and shear demands shall not exceed the factored flexural and shear resistances specified in Section 5 of AASHTO-CA BDS using expected material properties in-lieu of specified material properties.

Footing-to-column moment resisting joints shall be proportioned to satisfy joint shear requirements.

6.2.2.2 Footing Joint Shear

The principal stresses in footing-to-column moment resisting joints shall satisfy:

Principal compression:

$$p_c \leq 0.25 f'_c \quad (6.2.2.2-1)$$

Principal tension:

$$p_t \leq 12 \sqrt{f'_c} \quad (6.2.2.2-2)$$

(In Equations 6.2.2.2-1 and 6.2.2.2-2, f'_c and p_t have units of psi)

where:

$$p_t = \frac{f_v}{2} - \sqrt{\left(\frac{f_v}{2}\right)^2 + v_{jv}^2} \quad (6.2.2.2-3)$$

$$p_c = \frac{f_v}{2} + \sqrt{\left(\frac{f_v}{2}\right)^2 + v_{jv}^2} \quad (6.2.2.2-4)$$

$$v_{jv} = \frac{T_{jv}}{B_{eff}^{ftg} D_{ftg}} \quad (6.2.2.2-5)$$

$$T_{jv} = T_c - \sum T_{(i)}^{pile} \quad (6.2.2.2-6)$$

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

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

$$B_{eff}^{ftg} = \begin{cases} \sqrt{2} D_c & \text{Circular Column} \\ B_c + D_c & \text{Rectangular Column} \end{cases} \quad (6.2.2.2-7)$$

$$f_v = \frac{P_c}{A_{jh}^{fig}} \quad (6.2.2.2-8)$$

where:

P_c = nominal principal compressive stress

P_t = nominal principal tensile stress

f_v = average normal stress in the vertical direction

v_{jv} = nominal shear stress in the vertical direction

D_{fig} = depth of footing

D_c = column cross-sectional dimension parallel to the direction of bending

B_c = column cross-sectional dimension perpendicular to the direction of loading

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

With respect to Figure C6.2.2.2-1:

$$A_{jh}^{fig} = (D_c + D_{fig})(B_c + D_{fig}) \quad (C6.2.2.2-1)$$

- Rectangular column

$$A_{jh}^{fig} = \frac{\pi(D_c + D_{fig})^2}{4} \quad (C6.2.2.2-2)$$

- Circular column

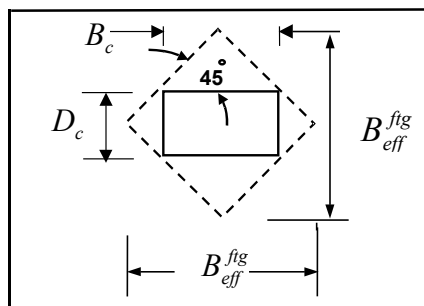
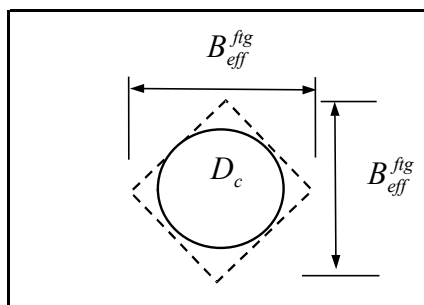
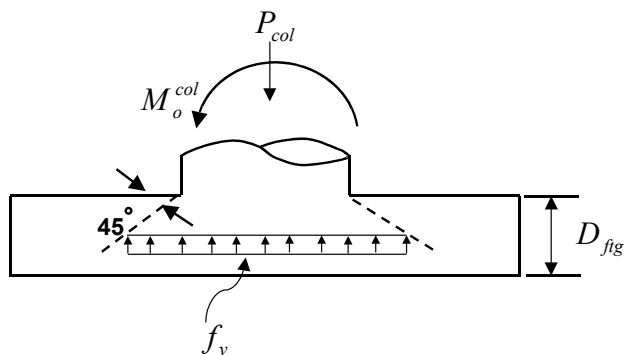


Figure C6.2.2.2-1 Assumed Effective Dimensions for Footing Joint Stress Calculation

6.2.2.3 Effective Footing Width for Flexure

The entire width of the footing shall be considered effective in resisting the column overstrength moment and the associated shear.

6.2.2.4 Rigid Footing Response

Rigid footing behavior and the associated linear distribution of pile forces and deflections may be assumed if

$$\frac{L_{fg}}{D_{fg}} \leq 2.2 \quad (6.2.2.4-1)$$

where:

L_{fg} = cantilever length of the footing or pile cap measured from the face of the column to the edge of the footing

6.2.2.5 Footing Stirrups

Stirrups with 90° hooks shall not be allowed in footings unless the principal tensile stress demand in the joint is less than $3.5\sqrt{f'_c}$, where f'_c has the units of psi.

C6.2.2.4

The rigid footing is commonly assumed in calculating pile or soil reactions. A finite element study on the effects of cap stiffness on pile reactions (Duan and McBride, 1995) shows that rigid cap assumption is valid when the cantilever length to thickness ratio (L_{fg}/D_{fg}) is less than or equal to 2.2.

C6.2.2.5

The type of hooks used for stirrups in footings depends on the column fixity condition and the level of principal tensile stress.

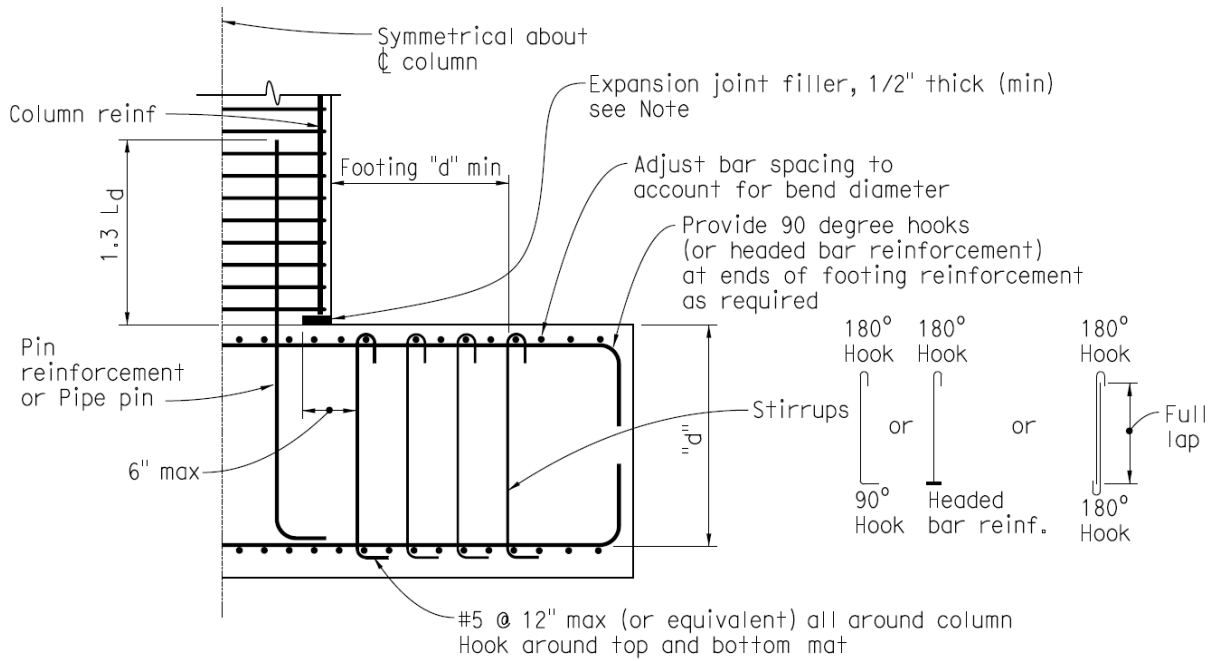
The bar size in the footing mats along with the principal tensile stress level and the spacing of the mat are all critical factors in the choice of the stirrup bar size.

Figures C6.2.2.5-1 and C6.2.2.5-2 illustrate the stirrup terminations that may be used for columns pinned and fixed at the base, respectively. Stirrups with 135° hooks are not recommended due to the difficulty in placing the stirrups during construction.

The principal stress demand may be reduced by increasing the depth of the footing.

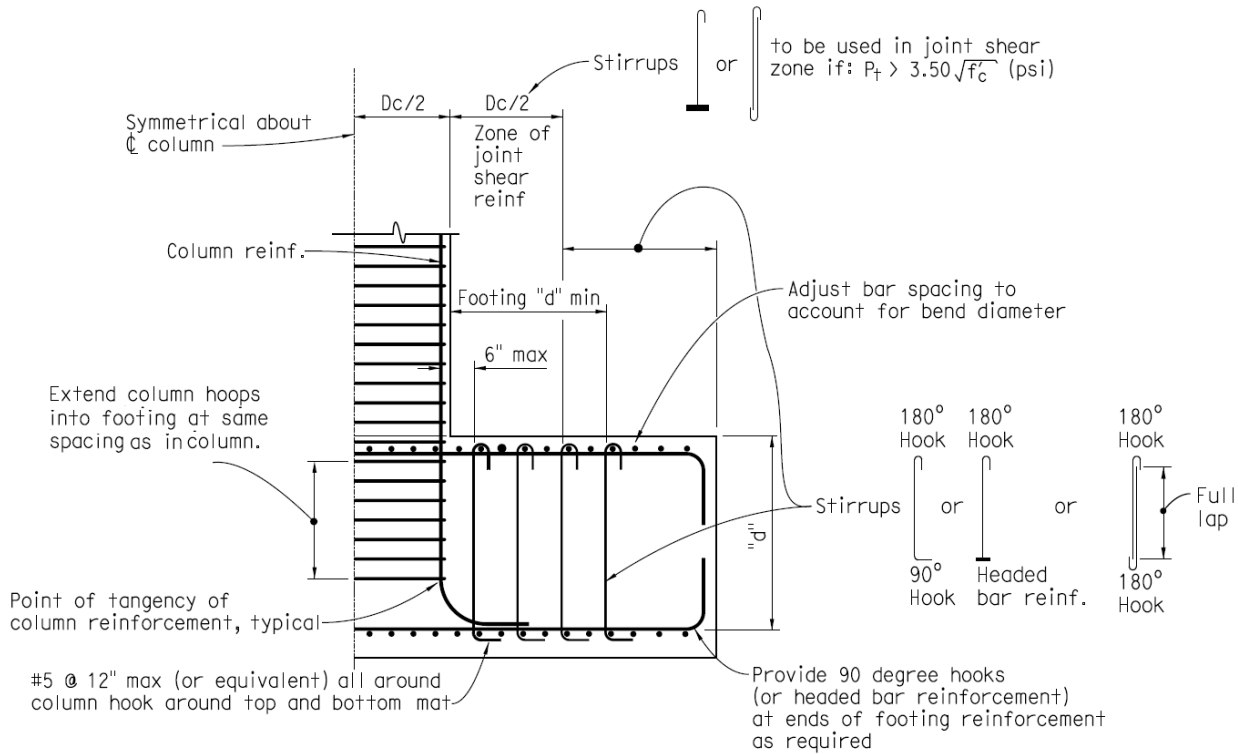
Refer to Section 6.2.2.2 for definition of principal tensile stress.

Use of #18 bars in footings needs a careful review as it affects the choice of the stirrup bar and hook detailing to fit the mat.



Note to Designer: 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 C6.2.2.5-1 Footing Reinforcement – Pinned Column



Note to Designer: When precast construction is used, the 6 in. (maximum) distance for placing the first stirrup shall be measured from the nearest edge of formed holes used for grouting extended main column reinforcement.

Figure C6.2.2.5-2 Footing Reinforcement – Fixed Column

6.2.2.6 Spread Footings

For structural design of spread footings, the distribution of bearing stress under the bottom of the footing shall be assumed linear.

Spread footings shall satisfy the following requirements:

- The bearing stress shall not exceed the factored nominal bearing resistance for seismic (Extreme Event I) provided by the geo-professional
- The sliding force shall not exceed the factored sliding resistance determined in accordance with Section 10 of AASHTO-CA BDS
- The eccentricity of loading for seismic (Extreme Event I) shall satisfy the requirement of Section 10 of AASHTO-CA BDS

6.2.3 Pile Foundations in Class S1 Soil

6.2.3.1 General

Piles founded in Class S1 soils may be analyzed and designed using the simplified model as specified in Section 6.2.3.2 if:

- The pile cap satisfies the stiffness requirement specified in Section 6.2.2.4
- The pile layout is symmetrical about the X- and Y- axes and all piles have the same cross-sectional area
- The upper portion of the pile measured from the pile cut-off elevation or finish grade (whichever is lower) is embedded in a class S1 soil profile.

For granular materials, the upper portion of the pile shall be taken as the greater of 8 pile diameters/widths and 20 feet.

For cohesive materials, the upper portion of the pile shall be taken as the greater of 6 pile diameters/widths and 15 feet.

C6.2.2.6

Spread footings in rock may be analyzed using a model similar to the simplified model for piles as specified in Section 6.2.3.2.

For additional information on spread footings, refer to Section 10 of AASHTO-CA BDS.

C6.2.3.1

Foundations embedded in Class S1 soils are capable of resisting design forces during an earthquake 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. Lateral analysis is typically ignored in the demand and capacity assessments for foundations in Class S1 soils.

The minimum required embedment is associated with all Class 90, 140, and 200 piles and other small diameter piles which include piles with a diameter and/or width of 24 inches or less. For these small diameter piles, a simple correlation related to pile size could not be developed due to many variables such as pile types, sizes, materials, etc. Therefore, a minimum embedment depth is used.

Pile foundations that do not satisfy the above three conditions shall require a comprehensive design as specified in Section 6.2.4.2.

Pile caps shall be designed as footings as specified in Section 6.2.2.

6.2.3.2 Simplified Design for Piles in Class S1 Soil C6.2.3.2

The axial force demands, $C_{(i)}^{pile}$ for compression and $T_{(i)}^{pile}$ for tension, shall not exceed the factored nominal seismic resistance provided by the geotechnical professional.

The axial force demand on an individual pile when the column reaches its overstrength moment, as shown in Figure 6.2.3.2-1, shall be taken as:

$$\left. \begin{array}{l} C_{(i)}^{pile} \\ T_{(i)}^{pile} \end{array} \right\} = \frac{P_p}{N_p} \pm \frac{\lambda M_{o(y)}^{col} d_{(i)x}}{I_{p.g.(y)}} \pm \frac{\lambda M_{o(x)}^{col} d_{(i)y}}{I_{p.g.(x)}} \quad (6.2.3.2-1)$$

where:

$$I_{p.g.(x)} = \sum n d_{(i)y}^2 \quad (6.2.3.2-2)$$

$$I_{p.g.(y)} = \sum n d_{(i)x}^2 \quad (6.2.3.2-3)$$

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

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

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

N_p = total number of piles in the pile group

λ = load reduction factor for a pile group

= 1.0 for pile cap design

= 0.83 for pile design

$M_{o(x)}^{col}$ = component of the column overstrength

moment demand about the X axis

A comprehensive analysis entails a lateral analysis of the foundation and its components with all applicable soil springs.

For calculating axial force demands on piles, the simplified model assumes that the demand due to plastic shear of the column at the pile-to-cap connection is ignored due to the reserve capacity inherent in pile group-supported footing systems in Class S1 soil. The reserve capacity includes contributions from passive soil resistance and side friction on the pile cap, which in combination with the competent properties of Class S1 soil, limits the lateral translation of the piles and results in low moment demand. This assumption is consistent with the observed response of similar foundations during past earthquakes.

Equation 6.2.3.2-1 provides axial force in the piles based on linear variation of axial force in the piles (rigid pile cap) when axial force and overstrength moment at the base of the column are applied to the foundation. The load reduction factor, λ accounts for redundancy in the pile group considering the fact that applied loads can be redistributed among the piles and overloading of any pile does not necessarily result in collapse of the foundation.

For pile cap design, the load reduction factor is 1.0 as caps are capacity protected members and are designed to resist the column overstrength moment. For pile group design, however, a reduction in the design moment demand from M_o to M_p (i.e., $\lambda = 1/1.2 = 0.83$) is adjudged to be reasonable in view of built-in redundancy due to group effects.

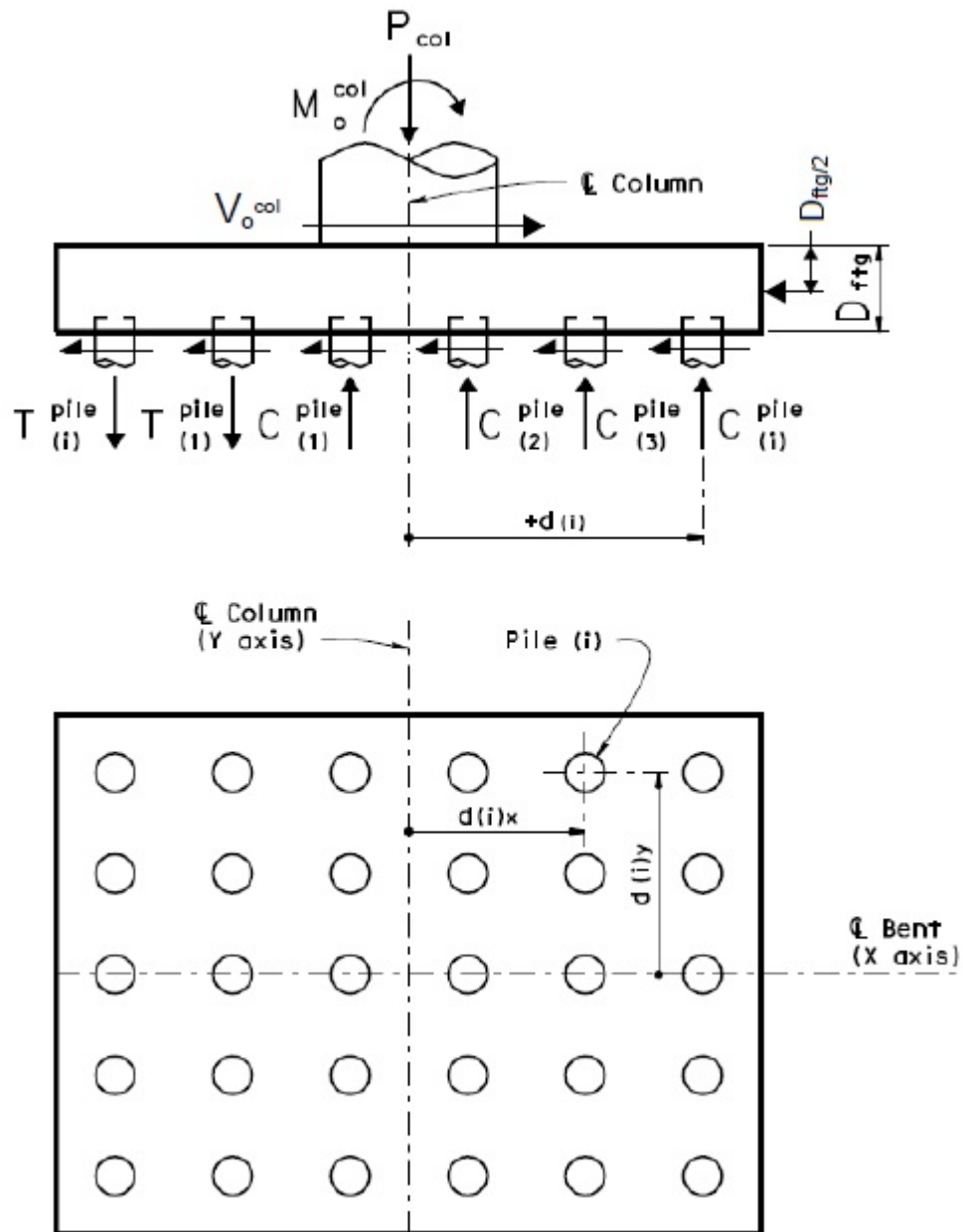


Figure 6.2.3.2-1 Simplified Pile Model for Foundations in Class S1 Soil

$M_{o(y)}^{col}$ = component of the column overstrength moment demand about the Y axis

$d_{(i)x}$ = distance from pile (i) to the centerline of the column along the X axis

$d_{(i)y}$ = distance from pile (i) to the centerline of the column along the Y axis

$I_{p.g.(x)}$ = moment of inertia of the pile group about the X axis

$I_{p.g.(y)}$ = moment of inertia of the pile group about the Y axis

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

Pile groups in Class S1 soil shall resist the shear demand from the column. The factored nominal shear resistance of the piles shall be greater than the plastic shear demand, V_o^{col} at the bottom of the column.

Pre-designed piles with wire confinement as shown in Caltrans ‘Standard Plans,’ may only be used in Class S1 soils.

6.2.4 Pile Foundations in Class S2 Soil

6.2.4.1 General

The foundation system shall be designed to resist the demands imposed by the overstrength moment and shear force of the columns and the lateral displacement of the foundation considering soil-foundation-structure interaction.

The axial force demand shall not exceed the factored nominal seismic resistance provided by the geo-professional.

The piles shall maintain their axial load capacity at the expected lateral displacement.

If the deformation demand creates plastic hinging in the piles, the piles shall be designed as seismic critical members.

The nominal shear resistance of piles in Class S1 soils are provided in Attachment 1 of MTD 5-1. The contribution of passive soil pressure may be added to the shear resistance obtained from Attachment 1 of MTD 5-1.

C6.2.4.1

In Class S2 soils the pile cap rotation may be accompanied by significant lateral displacements.

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.

- Increase the amount of fixity at the pile-to-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 or use larger piles

Piles supporting bents shall be fixed at the pile-to-cap connections. The pile-to-cap connections for piles supporting abutments may be pinned, fixed, or partially fixed.

Concrete piles shall have a minimum confinement reinforcement of #4 spiral or hoop reinforcement.

6.2.4.2 Comprehensive Foundation Design

Pile/shaft foundations in Class S2 soils shall be analyzed using Inelastic Static Analysis (ISA) as specified in Section 5.2.2, including soil-foundation-structure interaction, as illustrated in Figure 6.2.4.2-1. For preliminary design, pile foundations in Class S2 soils may be analyzed using a simplified model. However, ISA shall be performed for the final design of the foundation.

C6.2.4.2

Simplified Model for Pile Foundations in Class S2 Soil

The axial force demand on an individual pile when the column reaches its overstrength moment and the top of piles reach their plastic moment capacity, M_p^{pile} as shown in Figure C6.2.4.2-1, is approximated by:

$$\left. \begin{array}{l} C_{(i)}^{pile} \\ T_{(i)}^{pile} \end{array} \right\} = \frac{P_p}{N_p} \pm \frac{(M_{o(y)}^{col} + V_{o(x)}^{col} D_{fig} + N_p M_{p(y)}^{pile}) d_{(i)x}}{I_{p.g.(y)}} \pm \frac{(M_{o(x)}^{col} + V_{o(y)}^{col} D_{fig} + N_p M_{p(x)}^{pile}) d_{(i)y}}{I_{p.g.(x)}} \quad (C6.2.4.2-1)$$

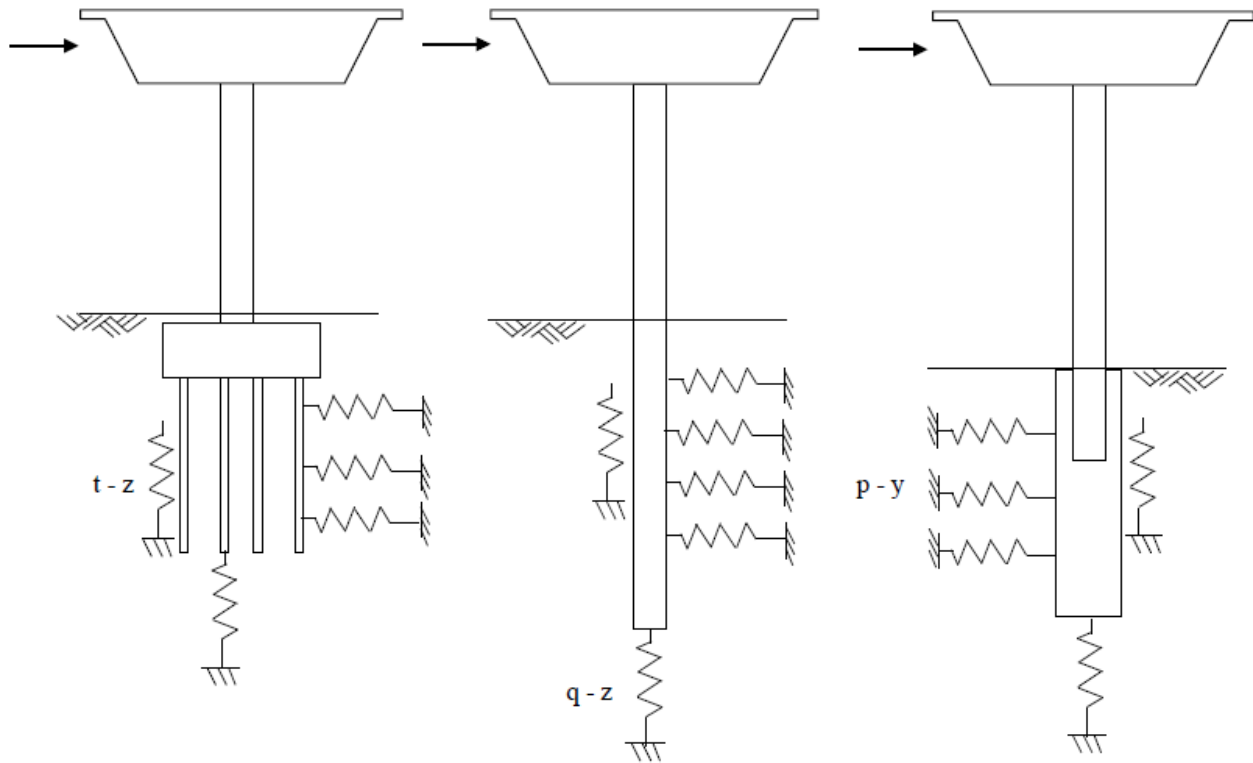
where:

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

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

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

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

**Notes:**

1. All springs are not shown
2. All spring types may not be applicable to a given soil

Figure 6.2.4.2-1 Inelastic Static Analysis of Foundations

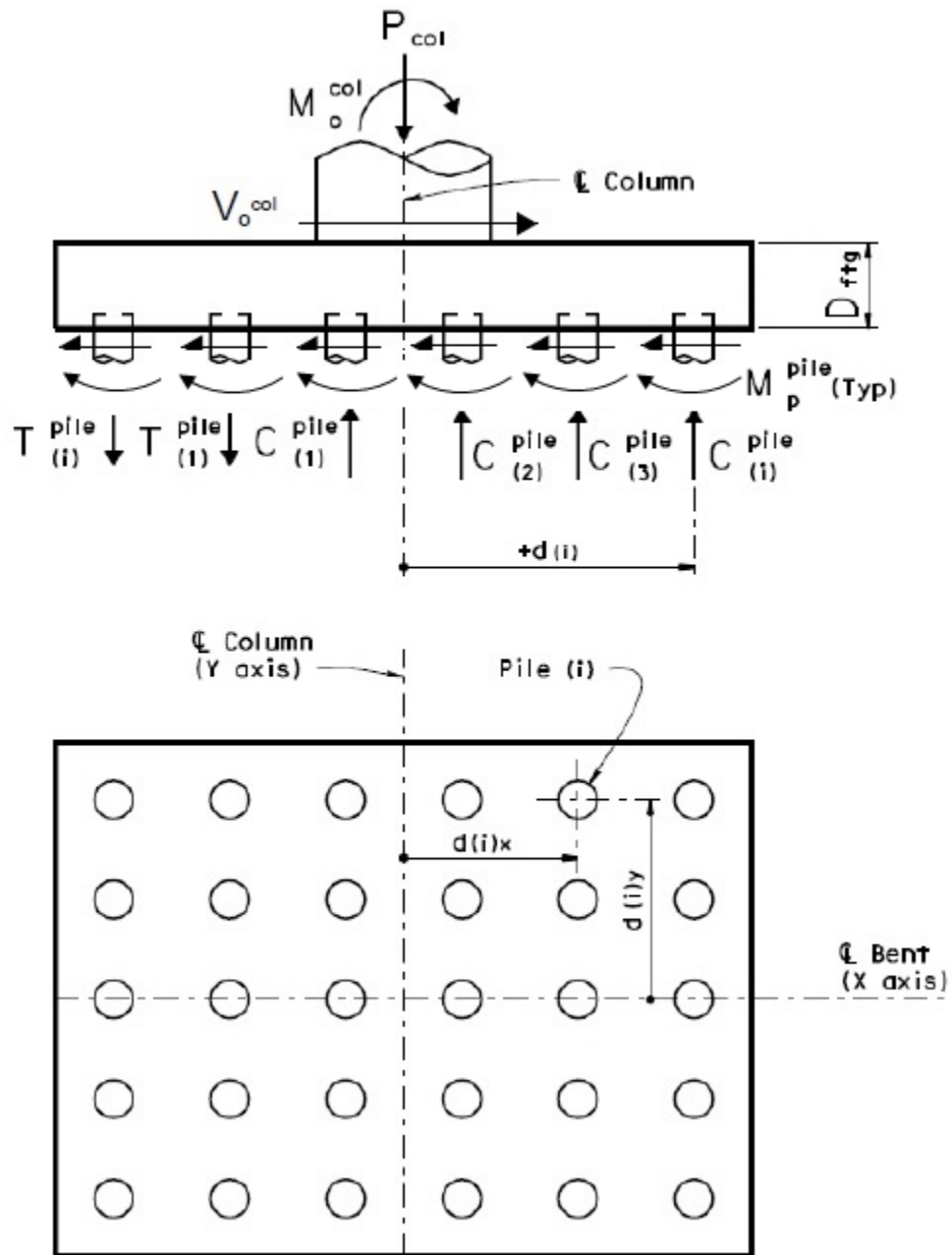


Figure C6.2.4.2-1 Simplified Pile Model for Foundations in Class S2 Soil

6.2.4.3 Scour and Liquefaction Considerations

For soils susceptible to scour and/or liquefaction, the lateral analysis shall be based on the occurrence and non-occurrence conditions using appropriate soil stiffness provided by the geo-professional.

- Case 1 – Potentially Liquefiable Layer Only

For potentially liquefiable layer existing within 50 feet from the ground surface, lateral analysis shall use liquefied soil stiffness (springs) for the liquefiable layer and a reduced soil stiffness for the soil above the liquefiable layer. For potentially liquefiable soil layers at deeper depths, a PSDC shall be developed.

- Case 2 – Scourable Layer Only

If the soil is susceptible to scour (degradation + contraction) only, lateral analysis shall be performed assuming there are no soil springs for the scour layer or layers above it.

- Case 3 – Combination of Potentially Liquefiable and Scourable Layers

If the liquefiable layer is below the scourable layer, lateral analysis shall use liquefied soil stiffness (springs) for the liquefiable layer and no soil springs for the scour layer. If the liquefiable layer is above the scourable layer, lateral analysis shall be performed assuming no soil springs for both layers.

C6.2.4.3

For the scourable layer, the occurrence condition implies a 100 % scour (i.e., Degradation and Contraction scour). For a potentially liquefiable layer, the occurrence condition implies that the soil liquefies. Both scenarios are needed to design for deformation and flexural demands.

The non-occurrence condition which implies that neither scour (i.e., Degradation, Contraction, and Local scour) nor liquefaction occurs, is needed to design for shear demand. For additional information on Degradation, Contraction, and Local scour, refer to Section 2 of AASHTO-CA BDS.

6.2.5 Shafts

6.2.5.1 General

All shafts shall be constructed with diameters equal to or greater than the maximum dimension of the supported column.

Type I shafts shall be designed such that the cross section of the confined core is the same for both the column and the shaft, but the concrete cover and area of transverse and longitudinal reinforcement may change between the column and the shaft, as shown in Figure 6.2.5.1-1.

Type-II shafts shall be at least 24 inches larger than the maximum dimension of the supported column, as shown in Figure 6.2.5.1-1.

Shafts supporting columns by means of a pin between the column base and the top of the shaft shall be designed as capacity protected members based on the overstrength shear and moment (if any) demands at the base of the column. If a reduced diameter reinforcement cage is used to form the pinned connection, the pin rebar cage shall be developed in both the column and the shaft. If a pipe or solid steel section is used to form the pinned connection, the lower portion of the pipe/solid section shall be developed in the shaft.

Reinforcing bar clearance requirements for drilled shafts as specified in Section 10 of AASHTO-CA BDS, shall be maintained.

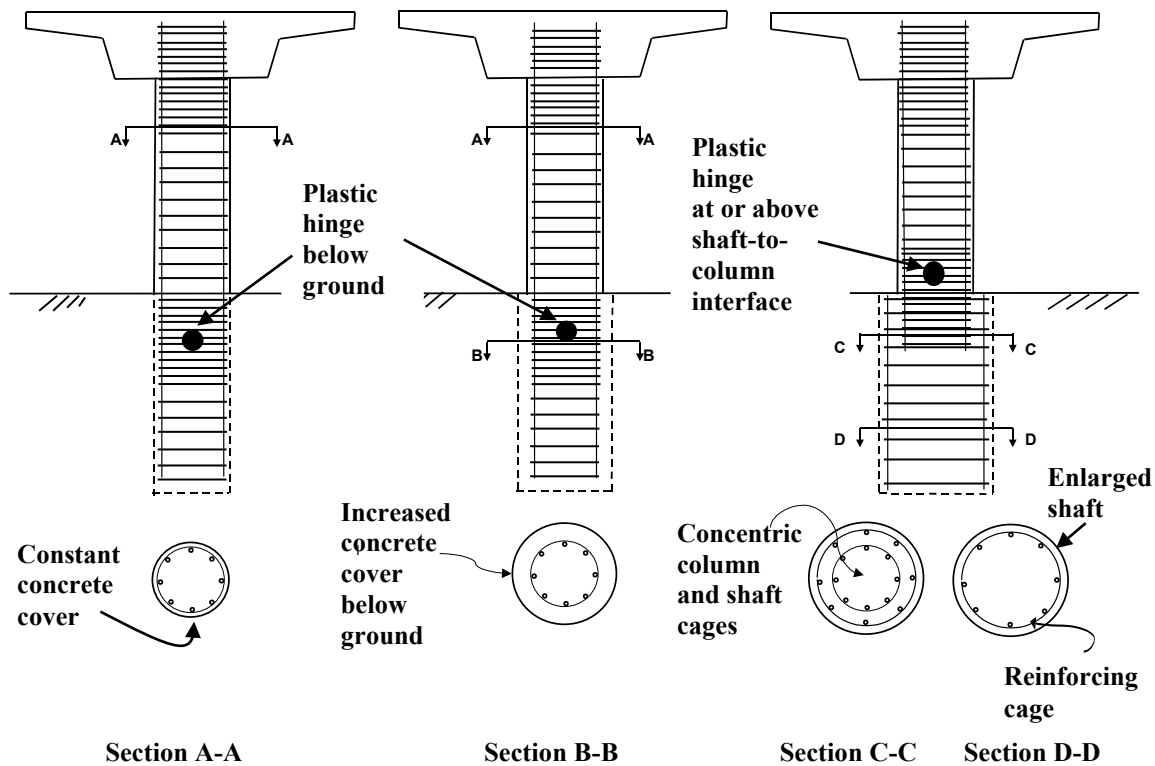
C6.2.5.1

Type I shafts are designed so the plastic hinge will form below ground in the shaft. However, the plastic hinge may form above the ground if steel casing is used below ground.

Type II shafts are designed larger than the column so that the plastic hinge will form at or above the shaft-to-column interface, thereby containing the majority of inelastic action in the ductile column member.

Where a shaft supports an oblong and/or a relatively short column, pinning the base of the column to the shaft will result in a decrease in the size (i.e., diameter and/or length) of the shaft and a consequent decrease in the foundation cost. For shafts connected to columns by a pin, the overstrength moment demand at the base of the column is taken as zero if a solid or pipe section with a cover pipe is used to form the pinned connection. If a reduced diameter reinforcement cage is used to form the pinned connection, the moment demand at the base of the column is taken as the overstrength moment transmitted by the pin reinforcement cage. The overstrength shear at the base of the column may be obtained by dividing the sum of the overstrength moments at the top and bottom of the column (if any) by the column height.

While it is common to design Type II shafts to be 24 inches larger than the column dimension, greater enlargements may be required to satisfy clearance requirements, or to reduce the depth of the shaft. In general, excessive enlargement of the shaft should be avoided for economic reasons. If the required shaft size dictated by a Class S2 soil condition is too large, the designer may consider using a CISS pile or other foundation types such as Type I shaft.



TYPE I SHAFTS

TYPE II SHAFT

NOTE:

Hoop reinforcement only shown. Transverse reinforcement may be hoops or spirals as specified in Sections 8.2.2.3 and 8.2.3.2.

Figure 6.2.5.1-1 Shaft Types

Type II Shafts will increase the foundation costs, compared to Type I Shafts, however there is an advantage of improved post-earthquake inspection and repair.

For structures with unequal column heights, Type I shafts can increase the effective length of the relatively short columns. Type II shafts can decrease the effective length of shaft extensions. The end result will be a structure with a balanced stiffness as discussed in Section 7.1.2.

Refer to Section 10 of AASHTO-CA BDS for spacing, clearance, and cover requirements for drilled shafts. Conventional standard sizes for CIDH piling are provided in Attachment 3 of MTD 3-1.

6.2.5.2 Flexure and Shear Demands on Type I Shafts

Type I shafts shall be designed as seismic critical members, as specified in Section 5.3.

6.2.5.3 Flexure and Shear Demands on Type II Shafts

Type II shafts shall be designed as capacity protected members. The factored moment capacity, ϕM_{ne} at any location along the shaft shall satisfy:

- Type II shafts cast in dry or dewatered hole

$$M_D \leq \phi M_{ne} \quad (6.2.5.3-1)$$

- Type II shafts cast using slurry-displacement method

$$M_D \leq 0.8\phi M_{ne} \quad (6.2.5.3-2)$$

where:

M_D = moment demand at any location generated by the column's overstrength moment and associated shear.

The factored shear capacity, ϕV_n at any location along the shaft shall satisfy:

$$V_D \leq \phi V_n \quad (6.2.5.3-3)$$

where:

V_D = shear demand at any location generated by the column's overstrength moment and associated shear.

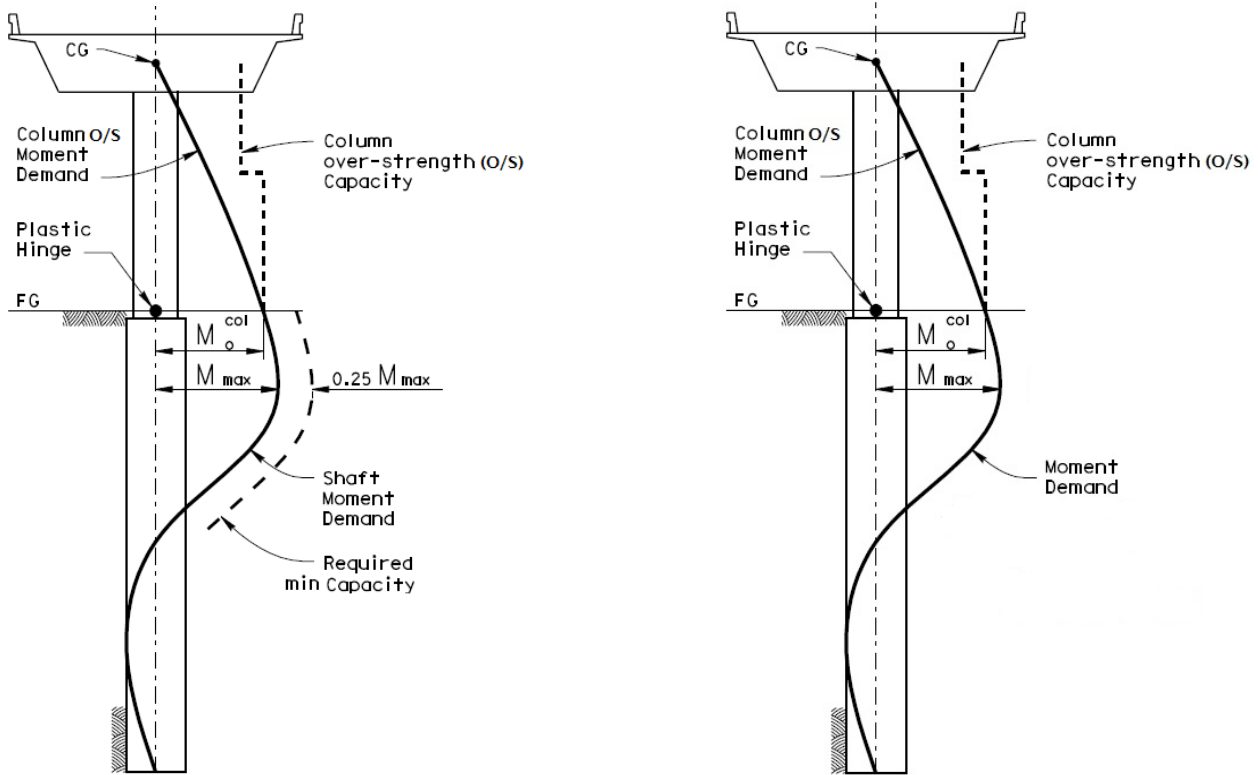
The distribution of moments along a shaft is dependent upon the geotechnical properties of the surrounding soil and the stiffness of the shaft.

Any change in the size of the shaft to meet the column overstrength demand will affect the soil springs and the resulting moment and shear demand in the shaft. Therefore, it may require iteration to achieve a satisfactory design.

The current design criteria for type II shafts will streamline the shaft design and remove excessive conservatism from shafts cast in dry or dewatered holes. In the dry condition, placing the rebar cage and concrete inside the shaft is under a controlled environment with minimal chances of soil caving and anomalies. Due to the additional risk and quality assurance challenges associated with construction under wet conditions, the factored moment capacity is limited to $0.8\phi M_{ne}$, resulting in an additional factor of safety of 1.25 for type II shafts cast using the slurry-displacement method.

Since specified (nominal) material properties, instead of expected material properties are used for shear design, the factor of safety of 1.25 shall not apply to the shear design of Type II shafts.

Typical moment and shear diagrams for a Type II shaft in the transverse direction are illustrated in Figures C6.2.5.3-1 and C6.2.5.3-2. The moment and shear demands in both the transverse and longitudinal directions are considered in the design.

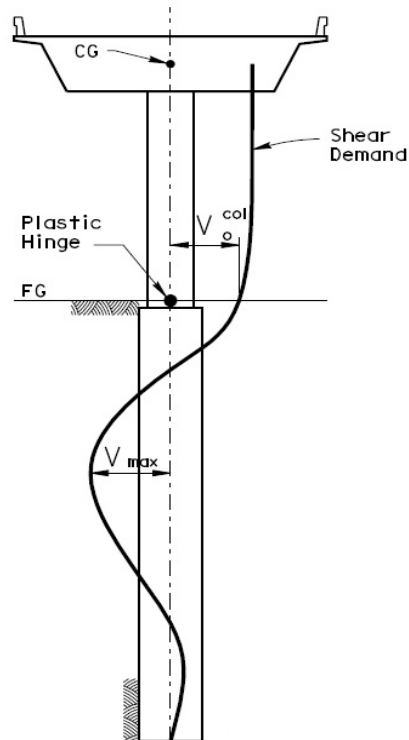


(A) Type II shafts cast using slurry-displacement method

(B) Type II shafts cast in dry holes

$$M_{max} = \text{maximum moment demand in the shaft}$$

Figure C6.2.5.3-1 Typical Moment Diagrams for Type II Shafts



V_{max} = maximum shear demand in the shaft

Figure C6.2.5.3-2 Typical Shear Diagram for Type II Shafts

6.2.5.4 Pile and Shaft Extensions

Piles and shafts extended above the ground shall have a minimum confinement of #4 spiral/hoop reinforcement for the entire length of the pile/shaft. The column section shall meet the ductility requirements of SCMs, as specified in Sections 3.5, 4.4.1 and 5.3. All requirements of Type I and Type II shafts shall also be satisfied.

6.2.6 Lateral Stability of Piles and Shafts

Lateral stability analysis shall be performed for piles and shafts to determine the tip elevation for horizontal loading based on the following assumptions:

- The governing design lateral load and the dead load are applied at the top of the pile/shaft
- The effects of scour and liquefaction are considered, if applicable
- The critical length for lateral stability is the embedded length of pile for which greater lengths do not result in a reduction of 5% or more in the deflection at the pile/shaft cut-off elevation.

The length of pile/shaft for lateral stability shall be greater than or equal to the critical length multiplied by the Embedment Factor shown in Table 6.2.6-1.

C6.2.6

Lateral stability analysis is accomplished by applying the governing design lateral load and the dead load at the top of the column over a range of pile/shaft lengths and recording the resulting top of pile/shaft deflections, as shown in Figure C6.2.6-1.

Pile/shaft group founded in class S1 soil is laterally stable. Therefore, lateral stability analysis is not required for pile/shaft groups in class S1 soil.

Table 6.2.6-1 Embedment Factor for Pile Tip Elevation

Pile/Shaft	Embedment Factor
Pile/shaft groups in Class S2 soil	1.0
Shafts (Types I and II) in multi-column bents	1.0
Shafts without rock sockets in single-column bents	1.2
Shafts with rock sockets in single-column bents	1.0 (for CIDH portion) 1.2 (for rock socket portion)

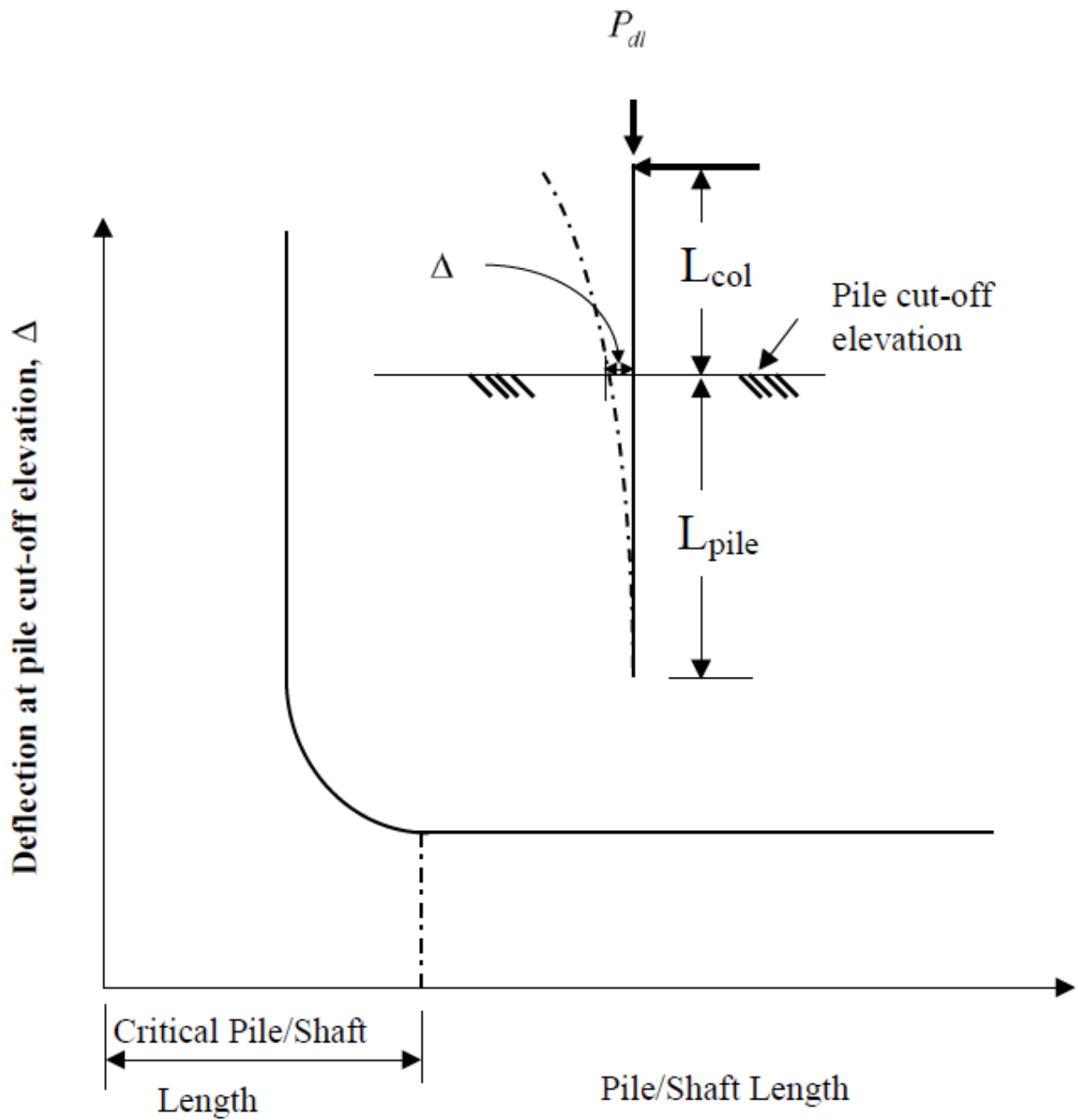


Figure C6.2.6-1 Lateral Stability Analysis of Piles and Shafts

6.3 ABUTMENTS

6.3.1 Longitudinal Stiffness

6.3.1.1 General

For ESA and EDA, effective stiffness obtained from an idealization of the initial stiffness of the nonlinear abutment-backfill force-deformation curve as shown in Figure 6.3.1.2-1 shall be used. For ISA, a bilinear force-deformation curve incorporating effective stiffness, as shown in Figure 6.3.1.2-1, shall be used.

C6.3.1.1

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. While the full nonlinear abutment backbone curve or the bilinear representation of the backbone curve may readily be used with NTHA, the bilinear representation of the backbone curve is most suited for ESA, EDA, and ISA.

For information on the full nonlinear force-deformation relationship, refer to Shamsabadi et al. (2007) for straight abutments or Rollins and Jessee (2013) for skew abutments.

The bilinear model presented herein is based on experimental and calibrated analytical models using engineered structural backfill to a relative compaction of at least 95%.

6.3.1.2 Bilinear Model

Effective abutment stiffness, K_{eff} as shown in Figure 6.3.1.2-1, shall be determined as:

$$K_{eff} = \frac{F_{abut}}{\Delta_{eff}} \quad (6.3.1.2-1)$$

$$\Delta_{eff} = \begin{cases} \Delta_{gap} + \Delta_{abut} & \text{(seat abutment)} \\ \Delta_{abut} & \text{(diaphragm abutment)} \end{cases} \quad (6.3.1.2-2)$$

$$\Delta_{abut} = \frac{F_{abut}}{K_{abut}} \quad (6.3.1.2-3)$$

C6.3.1.2

For seat-type abutments, the expansion hinge gap, Δ_{gap} is taken into consideration in determining the effective abutment wall stiffness as shown in Figure 6.3.1.2-1 and Equation 6.3.1.2-2. The idealized ultimate passive capacity given by Equation 6.3.1.2-4 is based on nonlinear hyperbolic backbone curves fitted by Shamsabadi et al. (2010) to the Force-Displacement data from passive earth pressure tests on large-scale abutment and pile cap tests (Maroney, 1995; Stewart et al., 2007; Rollins and Cole, 2006; Rollins and Jessee, 2013) and subsequent bilinear idealization of the hyperbolic backbone curves.

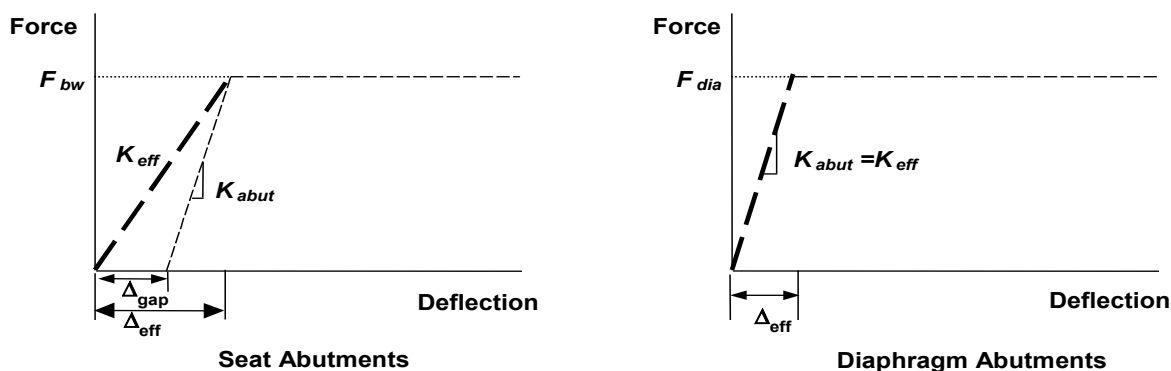


Figure 6.3.1.2-1 Nonlinear Abutment Model

$$F_{abut} = w_{abut} \left(\frac{5.5h_{abut}^{2.5}}{1 + 2.37h_{abut}} \right) R_{sk} \quad (6.3.1.2-4)$$

$$K_{abut} = w_{abut} (5.5h_{abut} + 20) R_{sk} \quad (6.3.1.2-5)$$

$$R_{sk} = e^{-\theta/45} \quad (6.3.1.2-6)$$

$$\theta \leq 66^\circ \quad (6.3.1.2-7)$$

where:

K_{eff} = effective abutment longitudinal stiffness (k/in)

F_{abut} = idealized ultimate passive capacity of the backfill behind abutment backwall or diaphragm (kip). In Figure 6.3.1.2-1, $F_{abut} = F_{bw}$ (for seat abutment), and $F_{abut} = F_{dia}$ (for diaphragm abutment)

Δ_{eff} = effective abutment longitudinal displacement when the passive force reaches F_{abut} (in.)

The ultimate force in the bilinear model was calibrated such that the potential energy (the area below the Force-Displacement curve) stored within the backfill due to translational movement of the superstructure, equals that of the nonlinear hyperbolic force-displacement curve (Shamsabadi and Rollins, 2014).

For skewed abutments, the ultimate capacity and stiffness are reduced by a reduction factor, R_{sk} . The skew reduction factor was calibrated by Shamsabadi and Rollins (2014) against experimental data from full-scale abutment tests on 0, 15, 30, 45 degree skew angles (Stewart et al., 2007; Rollins and Jessee, 2013) as well as from Finite Element simulation models for 60° skewed abutment.

The skew reduction factor accounts for the increased tendency for skew bridges to rotate and translate when subjected to seismic ground motions. The resulting deformation leads to a partial loss of contact between the abutment and the backfill with consequent decrease in abutment stiffness.

Δ_{gap} = width of expansion gap at seat abutment (in.)

Δ_{abut} = abutment displacement at idealized yield (in.)

K_{abut} = abutment longitudinal stiffness as shown in Figure 6.3.1.2-1 (kip/in)

R_{sk} = skew reduction factor

θ = abutment skew angle (degrees)

h_{abut} = height of backwall or diaphragm as shown in Figure 6.3.1.2-2 (ft); $2 \text{ ft} \leq h_{abut} \leq 10 \text{ ft}$. In Figure 6.3.1.2-2, h_{abut} shall be taken as h_{bw} for seat abutments, h_{dia}^{**} for diaphragm abutments designed for full soil pressure, and as h_{dia}^* for diaphragm abutments not designed for full soil pressure.

w_{abut} = abutment width along the skew direction (ft) as shown in Figure 6.3.1.2-3. $w_{abut} = w_{bw}$ for seat abutment and w_{dia} for diaphragm abutment, as shown in Figure 6.3.1.2-2.

For bridges with abutment skews greater than 66° , the longitudinal stiffness shall be taken equal to zero.

The limiting skew angle of 66° is based on bridge geometries observed in California.

Figure C6.3.1.2-1 schematically shows the stiffness reduction due to skew angle, where K represents K_{abut} for non-skew, K_θ represents K_{abut} for abutment with θ (degree) skew, and F_{bw} represents F_{abut} for abutment with θ (degree) skew.

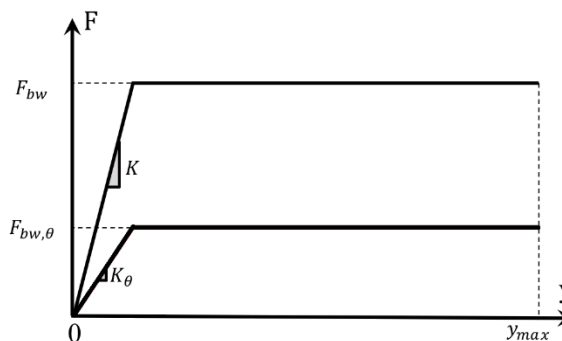


Figure C6.3.1.2-1 Reduction of Force and Stiffness for Skewed Abutments

For seat abutments, the backwall is typically designed to break off in order to protect the foundation from inelastic action.

For diaphragm abutments the entire diaphragm, above and below the soffit, is typically designed to engage the backfill immediately upon longitudinal movement of the bridge. 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 fracture plane.

The limitation on abutment backwall heights (i.e., 2 to 10 feet) is based on the heights of experimentally tested or analytically simulated backwalls.

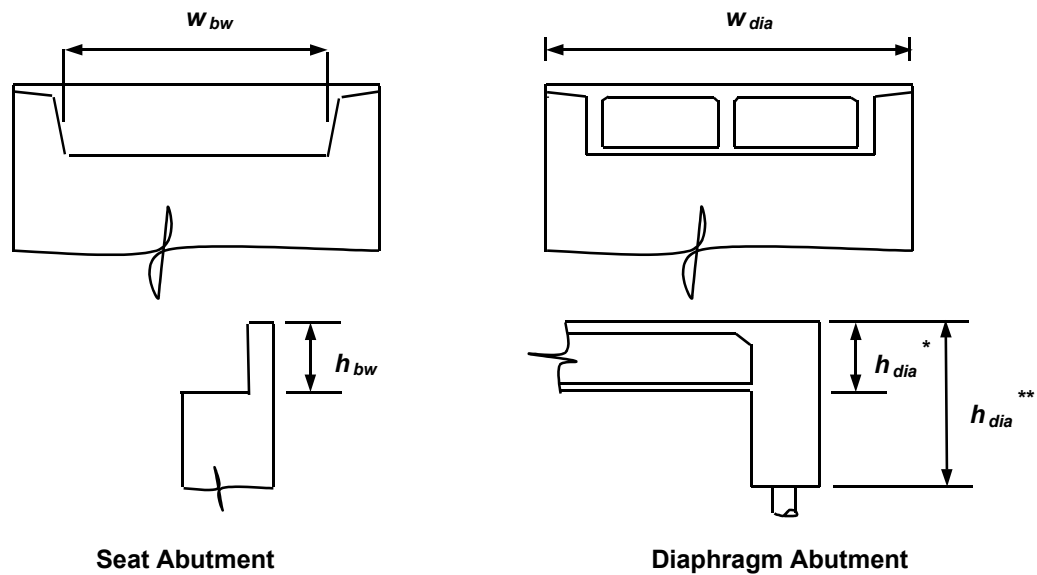


Figure 6.3.1.2-2 Effective Abutment Area

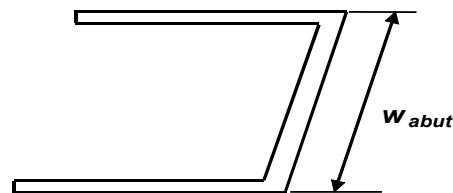


Figure 6.3.1.2-3 Effective Abutment Width for Skewed Bridges

6.3.1.3 Displacement Coefficient

C6.3.1.3

The abutment displacement coefficient, R_A , shall be used in the assessment of the effectiveness of the abutment.

$$R_A = \frac{\Delta_D}{\Delta_{eff}} \quad (6.3.1.3-1)$$

where:

Δ_D = longitudinal displacement demand from elastic analysis—obtained as specified in Section 4.2.1 and 4.2.2.

- For $R_A \leq 2$:

The linear elastic model may be used directly to determine the displacement demand of the bents or frame.

- For $R_A \geq 4$:

The effective abutment stiffness, K_{eff} in the elastic model shall be reduced to a minimum residual stiffness, K_{res} .

K_{res} shall be taken as:

$$K_{res} = 0.1(K_{eff}) \quad (6.3.1.3-2)$$

The elastic analysis shall be repeated to obtain revised bent displacements.

- For $2 < R_A < 4$:

An adjusted effective stiffness, K_{eff}^{adj} shall be used in the elastic model. K_{eff}^{adj} shall be taken as:

$$K_{eff}^{adj} = K_{eff} [1 - 0.45(R_A - 2)] \quad (6.3.1.3-3)$$

where:

K_{eff}^{adj} = effective abutment longitudinal stiffness adjusted for displacement coefficient (k/in)

The elastic analysis shall be repeated to obtain revised bent displacements.

For $R_A \leq 2$: The response is dominated by the abutments.

For $R_A \geq 4$: The abutment participation in the bridge response is assumed to be minimal and hence insignificant to the longitudinal seismic performance. The bents and frames will sustain significant deformation.

The residual spring stiffness, K_{res} 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.

$2 < R_A < 4$: The abutment participation in the bridge response is assumed to be moderate. The abutment stiffness in the elastic model is adjusted by interpolating effective abutment stiffness between K_{eff} and the residual stiffness K_{res} based on the R_A value.

6.3.2 Transverse Stiffness

A nominal transverse spring stiffness, K_{nom} equal to 50 % of the elastic transverse stiffness of the adjacent bent may be used at the abutment in the elastic demand assessment models.

Any additional element such as 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 may conservatively be estimated, ignoring the wingwalls, as 40 kip/in per pile.

6.3.3 Support Length

The support length normal to the centerline of the backwall, N_A shall satisfy:

$$N_A \geq \begin{cases} MR + \Delta_{eq} + L \\ \frac{D_s}{3} \\ 30 \text{ in.} \end{cases} \quad (6.3.3-1)$$

C6.3.2

Seat type abutments are designed to resist transverse service load and moderate levels of ground motion elastically. Linear elastic analysis cannot capture the inelastic response of the shear keys, wingwalls, or piles. The transverse capacity of seat abutments should not be considered effective for the 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, and not the abutment itself. Reasonable transverse displacement of the superstructure relative to the abutment seat can easily be accommodated without catastrophic consequences.

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.

Use of K_{nom} for abutment transverse stiffness is consistent with the stand-alone pushover analysis based design of the adjacent bents and it is conservative since 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.

C6.3.3

Sufficient abutment support length is provided to accommodate the anticipated thermal movement, prestress shortening, creep, shrinkage, and the relative longitudinal earthquake displacement, as shown in Figure 6.3.3-1.

where:

N_A = abutment support length as shown in Figure 6.3.3-1 (in.)

MR = movement range, i.e., total anticipated joint movement from the widest to the narrowest opening (in.)

L = pad dimension along the bridge for elastomeric bearings (in.), or length of masonry plate for PTFE spherical bearings (in.)

D_s = depth of superstructure at the bent cap (in.)

Δ_{eq} = displacement demand for the frame adjacent to the abutment (in.). Displacement of the abutment is assumed to be zero.

In Figure 6.3.3-1, a = thickness of joint filler.

For abutments skewed at an angle θ , the resulting support length measured along the longitudinal axis of the bridge is $(N_A / \cos \theta)$.

The term Movement Range (MR) is equivalent to “Movement Rating,” as previously used in Caltrans practice.

Elastomeric and Polytetrafluoroethylene (PTFE) spherical bearings may be used at bridge abutments but only PTFE spherical bearings are allowed at in-span hinges.

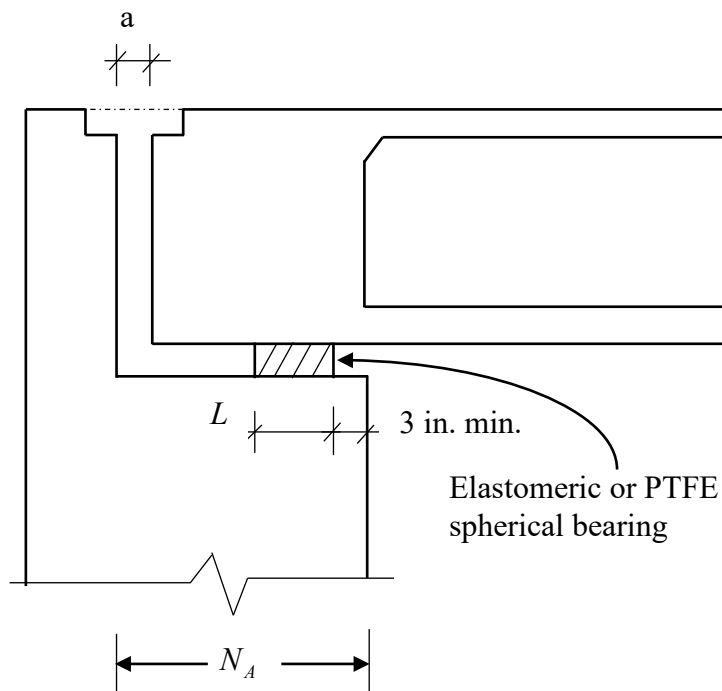


Figure 6.3.3-1 Abutment Support Length Requirements

6.3.4 Shear Key Design

The shear key force demand, F_{sk} for abutments supported on piles and spread footings shall be determined as:

- For abutment on piles

$$F_{sk} = \alpha(0.75V_{piles} + V_{ww}) \quad (6.3.4-1)$$

- For abutment on spread footing

$$F_{sk} = \alpha(0.5P_{dl}) \quad (6.3.4-2)$$

in which:

$$0.5 \leq \alpha \leq 1 \quad (6.3.4-3)$$

where:

F_{sk} = shear key force demand (kip)

V_{piles} = sum of lateral geotechnical capacity of the piles (kip)

V_{ww} = shear capacity of one wingwall (kip)

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

α = adjustment factor for shear key force demand

For cases where the geotechnical capacity of the piles or the nominal sliding resistance of the spread footing is unusually large, the shear key force demand of Equations 6.3.4-1 and 6.3.4-2 shall be taken, respectively, as:

$$F_{sk} = \alpha P_{dl}^{\text{sup}} \leq \alpha(0.75V_{piles} + V_{ww}) \quad (6.3.4-4)$$

$$F_{sk} = \alpha P_{dl}^{\text{sup}} \leq \alpha(0.5P_{dl}) \quad (6.3.4-5)$$

where:

P_{dl}^{sup} = superstructure dead load reaction at the abutment (kip).

C6.3.4

Typically, abutment shear keys are expected to transmit the lateral 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.

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 superstructure rotation or shortening.

For design, the adjustment factor for shear key demand, α equal to 1 should be assumed initially. Subsequently, α may be reduced as needed to produce an acceptable design.

In Equation 6.3.4-1, the factor 0.75 is a safety factor that accounts for the uncertainty in estimating the lateral geotechnical capacity of the piles.

In Equation 6.3.4-2, the factor 0.5 represents the coefficient of friction between the footing and the underlying soil.

For information on lateral capacity of piles, refer to Attachment 1 of MTD 5-1. Lateral capacity of piles may also be obtained from “LPILE” analysis.

The shear capacity of wingwall may be determined in accordance with the shear resistance provisions of AASHTO-CA BDS Section 5.

6.3.5 Shear Key Reinforcement

Abutment shear key reinforcement shall be designed using either the Isolated key method or the Monolithic (i.e., Non-isolated) key method, as shown in Figures 6.3.5-1 and 6.3.5-2, respectively.

The horizontal reinforcement in the stem wall below the shear key shall be designed to carry the shear key force elastically.

For shear keys reinforced with headed bars, the minimum development length of full-size headed bars shall be taken as $14d_b$.

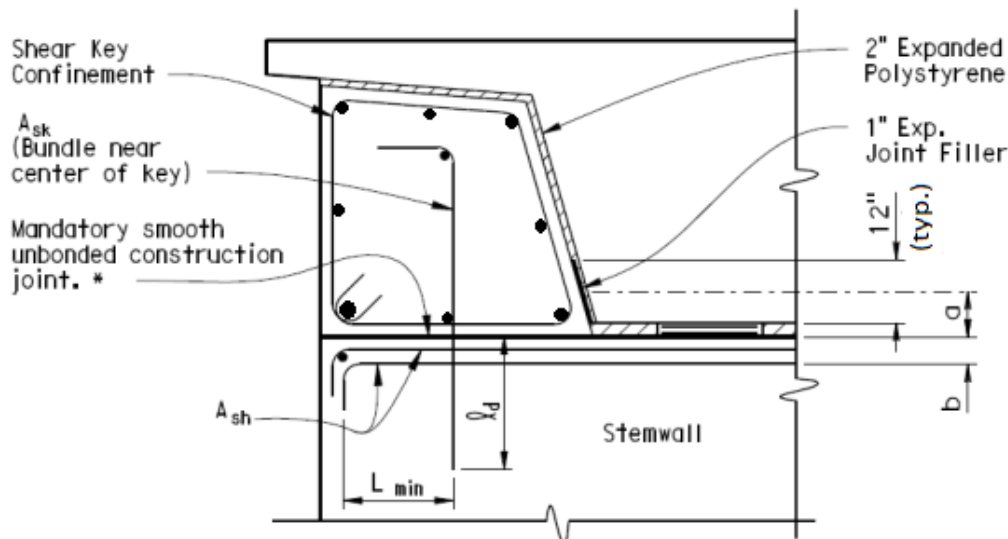
C6.3.5

A shear key is isolated when a smooth construction joint is used at the shear key interfaces with the stemwall and backwall and only specifically designed reinforcement crosses the interface.

Due to development length requirements, it is recommended that vertical shear key reinforcement be no larger than #11 bars.

In situations such as non-standard overhangs, high skews, and retrofit conditions at widenings where the size of the shear key is not adequate to develop straight bars, hooks or headed bars may be used. Headed bars are recommended in place of large radius hooks.

A full-size Headed bar has a net head area equal to nine times the cross sectional area of the bar (i.e., $9A_b$).

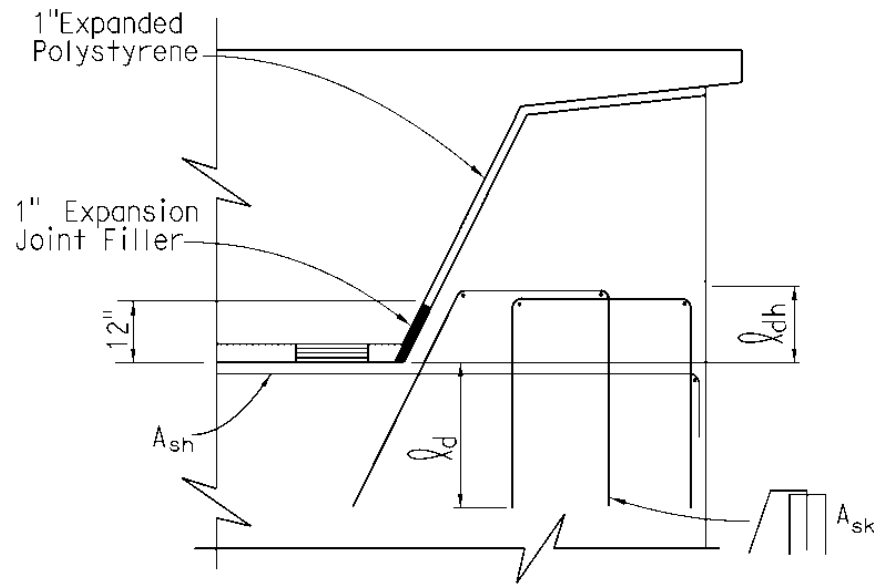


* Smooth construction joint is required at the shear key interfaces with the stemwall and backwall to effectively isolate the shear key. These interfaces shall be trowel-finished smooth before application of a bond breaker such as construction paper. Form oil shall not be used as a bond breaker for this purpose.

NOTES:

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

Figure 6.3.5-1 Isolated Shear Key Reinforcement Details



(B) Monolithic Shear Key

NOTES:

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

Figure 6.3.5-2 Monolithic Shear Key Reinforcement Details**6.3.5.1 Isolated Key**

- *Vertical Reinforcement*

The required area of interface shear reinforcement, A_{sk} crossing the shear plane, shall be taken as:

$$A_{sk}^{Iso} = \frac{F_{sk}}{1.8f_{ye}} \quad (6.3.5.1-1)$$

The shear key vertical reinforcement shall be placed in a single line parallel to the bridge centerline and located within the middle third of the shear key base as shown in Figure 6.3.5-1.

C6.3.5.1

Equations 6.3.5.1-1, 6.3.5.1-2, and the reinforcement details shown in Figure 6.3.5-1 are based on experimental tests on exterior shear keys (Bozorgzadeh et al., 2007). The reinforcing detail was developed to ensure that exterior shear keys fail through a well-defined horizontal plane that is easily repaired after an earthquake.

A minimum horizontal distance, L_{min} as shown in Figure 6.3.5-1 should be provided. L_{min} is determined as:

$$L_{min,hooked} = 0.6(a + b) + l_{dh} \quad (C6.3.5.1-1)$$

- *Shear Key Confinement Reinforcement*

Shear key confinement reinforcement shall be a minimum of #4 bars at 12 inches on center in all directions placed at the periphery of the key block, as illustrated in Figure 6.3.5-1.

- *Horizontal Reinforcement in the Stem Wall (Hanger Bars)*

The area of horizontal reinforcement in the stem wall, A_{sh} shall be taken as:

$$A_{sh} = 2A_{sk}^{Iso} \quad (6.3.5.1-2)$$

where:

A_{sk}^{Iso} = area of interface shear reinforcement provided (in.²)

$$L_{\min, \text{headed}} = 0.6(a + b) + 3 \quad (C6.3.5.1-2)$$

where:

L_{\min} = minimum horizontal distance from the vertical shear key bars to the hooked or headed end of the lowest layer of hanger bars (in.)

a = vertical distance from the location of the applied force on the shear key to the top surface of the stem wall, taken as one half the vertical length of the expansion joint filler plus the pad thickness (in.), as shown in Figure 6.3.5-1

b = vertical distance from the top surface of the stem wall to the centroid of the lowest layer of shear key horizontal reinforcement (in.), as shown in Figure 6.3.5-1

l_{dh} = development length in tension of standard hooked bars as specified in AASHTO-CA BDS (in.)

The size of the horizontal reinforcement in the stem wall may be varied as needed to ensure the shear key vertical reinforcement is located within the middle third of the shear key base.

6.3.5.2 Monolithic Key

- *Vertical Reinforcement*

The area of interface shear reinforcement crossing the shear plane, A_{sk} shall be taken as:

$$A_{sk}^{mono} = \frac{1}{1.4f_{ye}} (F_{sk} - 0.4A_{cv}) \quad (6.3.5.2-1)$$

in which:

$$0.4A_{cv} < F_{sk} \leq \min \left(\begin{array}{l} 0.25f_{ce}'A_{cv} \\ 1.5A_{cv} \end{array} \right) \quad (6.3.5.2-2)$$

$$A_{sk}^{mono} \geq \frac{0.05A_{cv}}{f_{ye}} \quad (6.3.5.2-3)$$

C6.3.5.2

Shear key construction using normal weight concrete placed monolithically is assumed.

Equations 6.3.5.2-1 to 6.3.5.2-4 are based on the interface shear transfer-shear friction provisions of AASHTO-CA BDS.

where:

A_{cv} = area of concrete engaged in interface shear transfer (in.²)

f_{ye} = expected yield stress for A706 reinforcement (ksi)

f'_{ce} = expected compressive strength of unconfined concrete (ksi)

- *Horizontal Reinforcement in the Stem Wall (Hanger Bars)*

The area of horizontal reinforcement in the stem wall, A_{sh} shall be determined as:

$$A_{sh} = \max \left\{ \begin{array}{l} 2A_{sk}^{mono} \\ \frac{F_{sk}}{f_{ye}} \end{array} \right. \quad (6.3.5.2-4)$$

where:

A_{sk}^{mono} = area of interface shear reinforcement provided for monolithic shear key.

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BRIDGE COMPONENTS

7.1 FRAMES

7.1.1 Frame and Bent Proportioning

Bridge frames shall be proportioned to achieve a balance in the stiffness and geometry.

7.1.2 Balanced Stiffness

The effective stiffness-to-mass ratio between any two bents within a frame or between any two columns within a bent shall satisfy the requirements specified in Table 7.1.2-1.

The effective stiffness-to-mass ratio between adjacent bents within a frame or between adjacent columns within a bent shall satisfy the requirements specified in Table 7.1.2-1.

The effective stiffness of a bent or column shall be based on effective section properties as specified in Section 3.4.

In Table 7.1.2-1:

k_i^e = effective stiffness of bent or column i
(kip/in.)

k_j^e = effective stiffness of bent or column j
(kip/in.)

m_i = tributary mass of bent or column i (kip-sec²/in.)

m_j = tributary mass of bent or column j (kip-sec²/in.)

C7.1.1

The best way to increase a structure's likelihood of responding to seismic attack in its fundamental mode of vibration is to balance its stiffness and mass distribution. Irregularities in geometry increase the likelihood of complex nonlinear response that cannot be accurately predicted by elastic modeling or plane frame inelastic static modeling.

C7.1.2

An increase in superstructure mass along the length of the frame should be accompanied by a reasonable increase in column stiffness, as shown in Figure C7.1.2-1.

The following considerations are taken into account when calculating effective stiffness: framing effects, end conditions, column height, percentages of longitudinal and transverse column steel, column diameter, and foundation flexibility.

Some of the consequences of unbalanced stiffness 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

The following techniques may be used to satisfy the requirements of Table 7.1.2-1.

- Use oversized shafts
- Adjust effective column lengths (i.e. lower footings using isolation casing)

Table 7.1.2-1 Column/Bent Stiffness-to-Mass Ratios for Bents/Frames

Column/Bent	Stiffness-to-Mass Ratio
For any two bents in a frame or any two columns in a Bent	$0.5 \leq \frac{\left(\frac{k_i^e}{m_i} \right)}{\left(\frac{k_j^e}{m_j} \right)} \leq 2.0 \quad (7.1.2-1)$
For adjacent bents in a frame or adjacent columns in a Bent	$0.75 \leq \frac{\left(\frac{k_i^e}{m_i} \right)}{\left(\frac{k_j^e}{m_j} \right)} \leq 1.33 \quad (7.1.2-2)$

- Modify column end conditions
- Reduce/redistribute superstructure mass
- Vary the column cross section and longitudinal reinforcement ratios
- Add or relocate columns/bents
- Modify the hinge/expansion joint layout

Section 7.6.1 provides additional information on optimizing the performance of bridge frames.

A careful evaluation of the local ductility demands and capacities is recommended if project constraints make it impractical to satisfy the stiffness requirements as specified in Table 7.1.2-1.

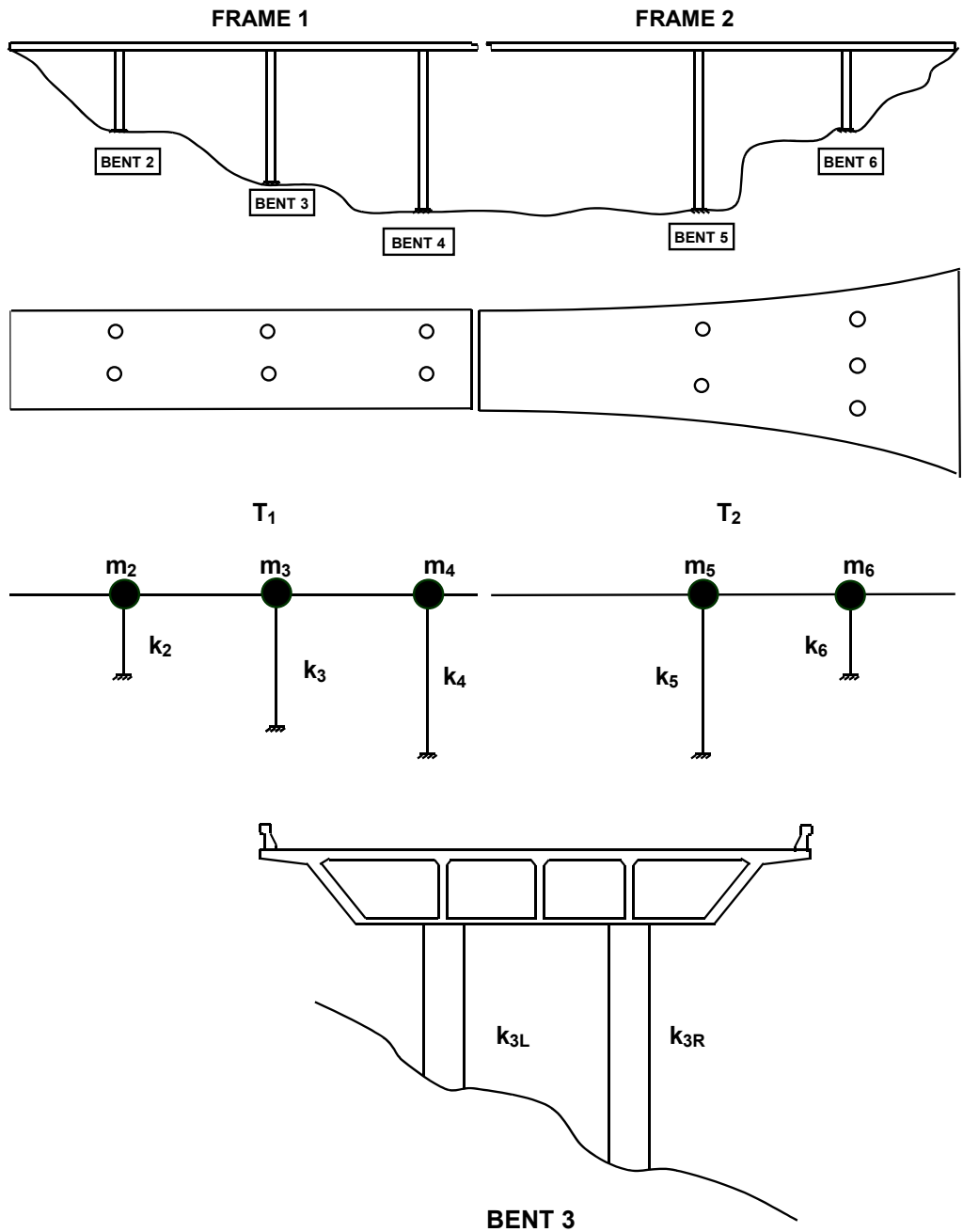


Figure C7.1.2-1 Balanced Stiffness Parameters

7.1.3 Balanced Frame Geometry

The ratio of fundamental periods of vibration for adjacent frames in the longitudinal and transverse directions shall satisfy:

$$0.7 \leq \frac{T_i}{T_j} \leq 1.43 \quad (7.1.3-1)$$

where:

T_i = natural period of frame i (sec)

T_j = natural period of frame j (sec)

C7.1.3

The consequences of unbalanced frame geometry include a greater likelihood of out-of-phase response between adjacent frames leading to large relative displacements that increase the probability of longitudinal unseating and pounding between frames at the expansion joints. The pounding of adjacent frames will transfer the seismic demand from one frame to another, which in combination with relative translation of the frames, may exceed the stand-alone capacity of the frame receiving the additional seismic demand.

The techniques recommended in C7.1.2 may also be used to satisfy Equation 7.1.3-1.

7.2 SUPERSTRUCTURES

7.2.1 Girders

7.2.1.1 Effective Superstructure Width

The effective width of superstructure resisting longitudinal seismic moments, B_{eff} shall be taken as:

$$B_{eff} = \begin{cases} D_c + 2D_s & \text{– Box girders and slab} \\ & \text{superstructures} \\ D_c + D_s & \text{– Open soffit superstructures} \end{cases} \quad (7.2.1.1-1)$$

where:

D_c = column cross sectional dimension in the transverse direction. For flared columns, D_c shall be based on the prismatic column section, ignoring the flares

D_s = depth of superstructure at the bent cap

The effective superstructure width may be increased for cross-sections away from the bent cap by using a 45° spread from the cap face until the

C7.2.1.1

The effective width for open soffit structures (e.g. T-Beams and I- Girders) is reduced because they offer less resistance to the torsional rotation of the bent cap.

If the effective width cannot accommodate enough steel to satisfy the overstrength requirements of Sections 4.4.2 and 4.4.3, the following actions may be taken:

- Thicken the soffit and/or deck slabs
- Increase the resisting section by widening the column
- Haunch the superstructure
- Add additional columns
- Increase the structure depth

full section becomes effective, as shown in Figure 7.2.1.1-1 (A) and (B).

On skewed bridges, the effective width shall be projected normal to the girders with one end of the width intersecting the bent face such that one half of the width lies on either side of the column centerline.

The method for determining the effective width for skewed bridges is illustrated in Figure C7.2.1.1-1.

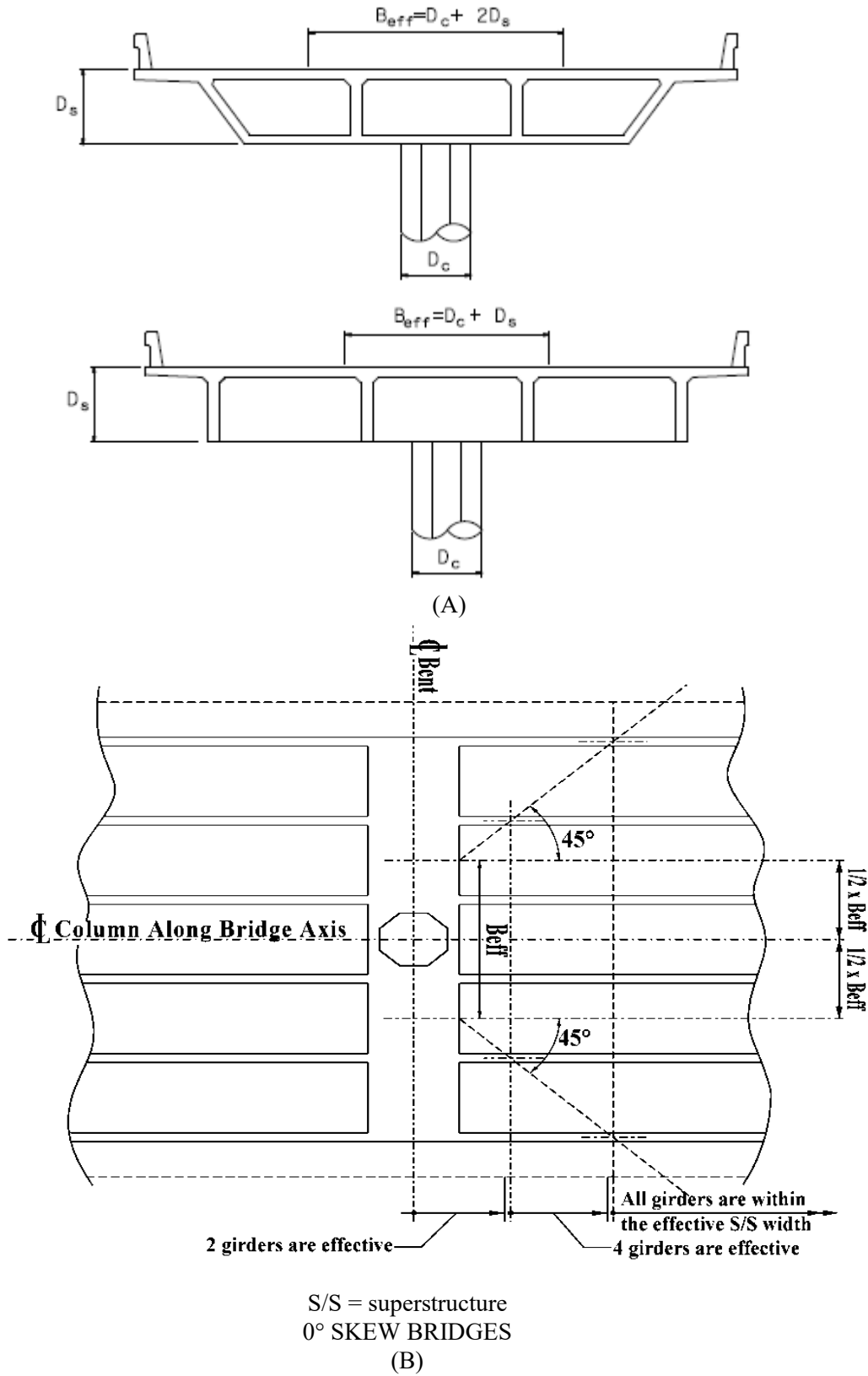


Figure 7.2.1.1-1 Effective Superstructure Width

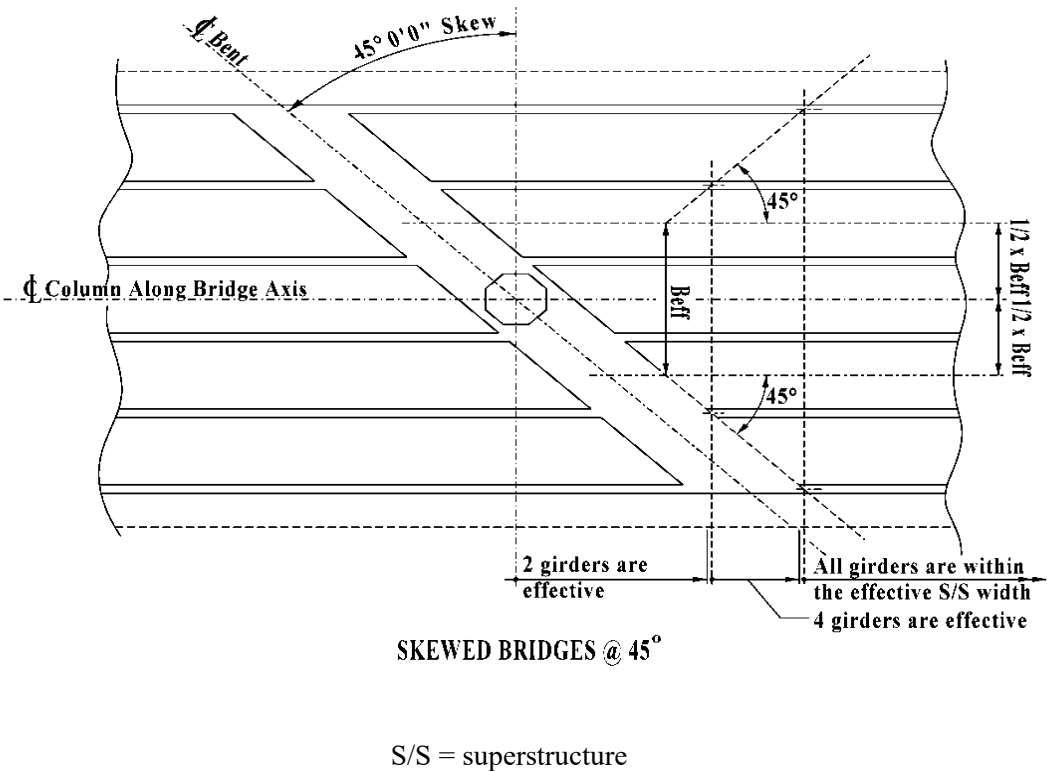
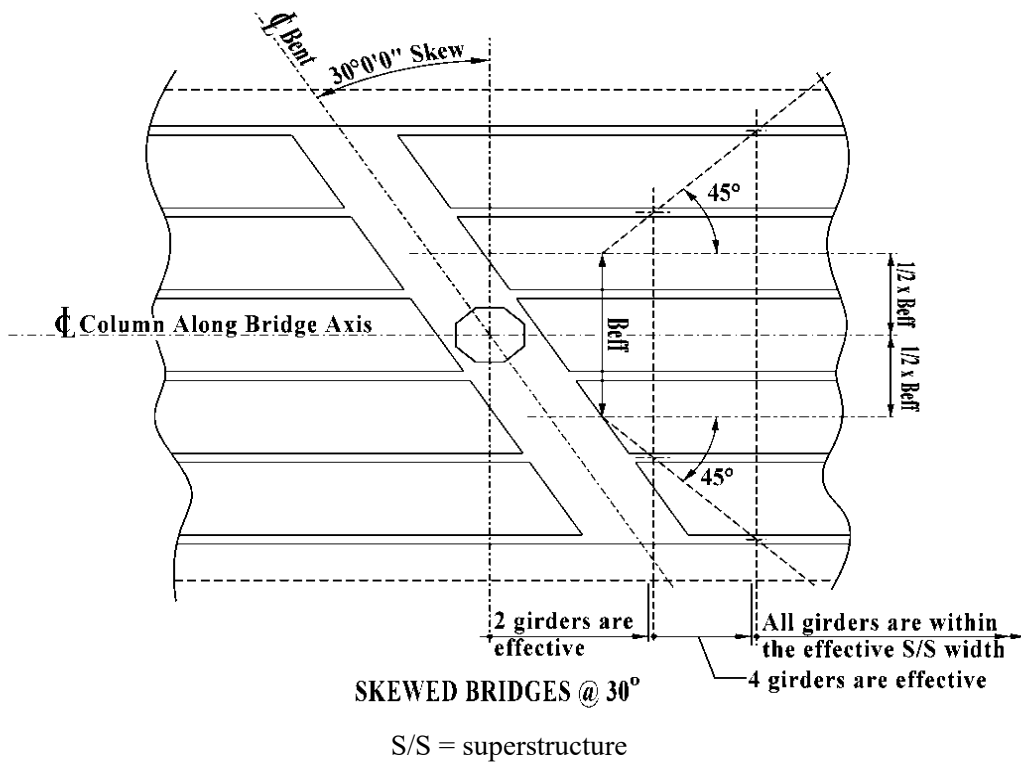
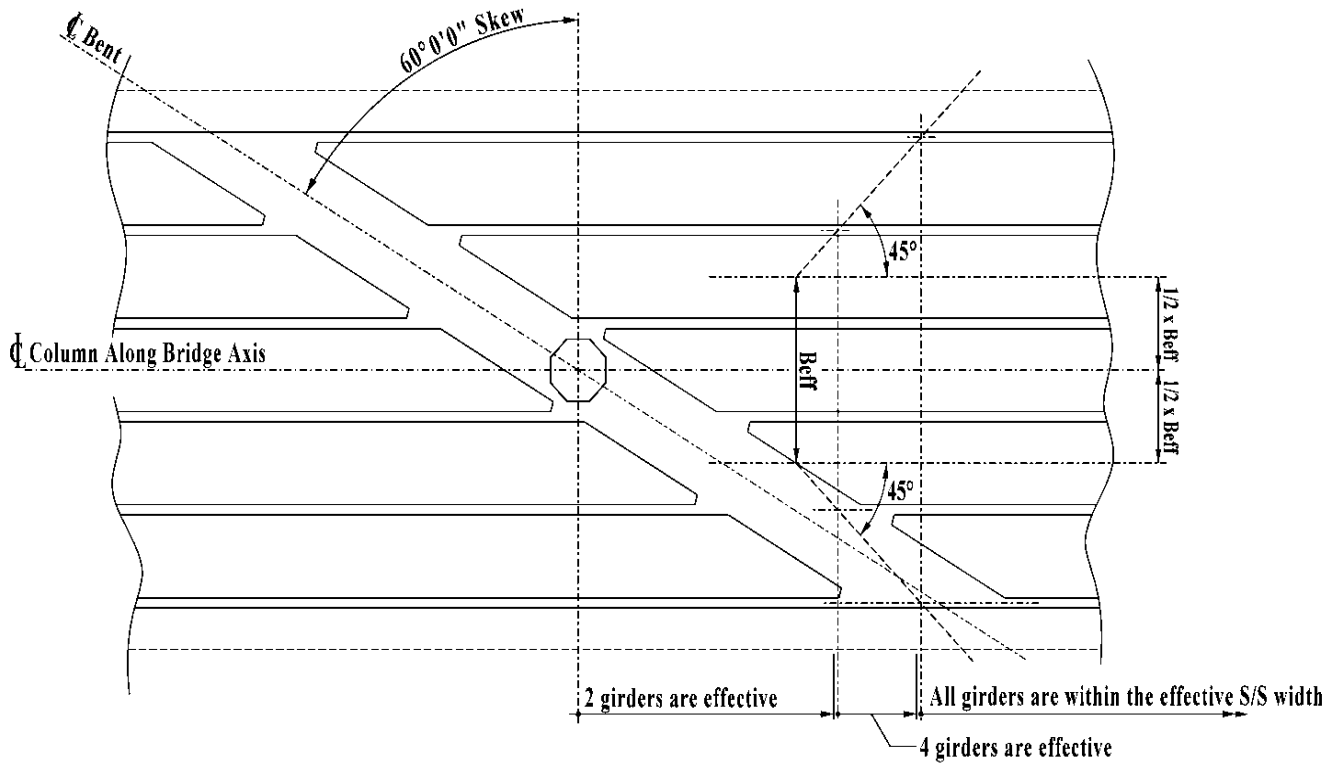


Figure C7.2.1.1-1 Effective Superstructure Width for Skewed Bridge



SKEWED BRIDGES @ 60°

S/S = superstructure

Figure C7.2.1.1-1 Effective Superstructure Width for Skewed Bridge (continued)

7.2.1.2 Precast Girders

Multi-span precast (PC) girder bridges located at sites with peak ground acceleration (PGA) equal to or greater than 0.6g shall be provided with superstructure top and bottom steel at the supports to ensure continuity of reinforcement in the longitudinal direction and capacity protection of the superstructure, as shown in Figure 7.2.1.2-1 (Type A and Type B). The bottom reinforcement shall be in the form of prestressing strands, or mild steel, or an equivalent combination of prestressing strands and mild steel. A minimum of 4 - 0.6 inch diameter prestressing strands, or 4 - #8 mild steel bars, or equivalent mild steel and/or prestressing strands, shall be provided in the girder bottom over the supports. This bottom continuity reinforcement may also be used to satisfy the requirements of vertical acceleration, as specified in Section 7.2.2.

For sites with peak ground acceleration less than 0.6g, a pinned substructure-to-superstructure connection with superstructure negative moment continuity, as shown in Figure 7.2.1.2-1 (Type C) may also be used. A true pin that does not transmit moment across the substructure-to-superstructure joint shall be used for Type C connection.

If the substructure is pinned to the superstructure as shown in Figure 7.2.1.2-1 (Types B and C), the base of the SCM shall be fixed.

Substructure-to-superstructure connections with only negative moment continuity and without a true pin connection, as illustrated in Figure C7.2.1.2-1 (Type D), shall not be allowed.

Allowable precast girder seismic connections are summarized in Table 7.2.1.2-1.

Table 7.2.1.2-1 Allowable Connection Types for Precast Girders

PGA	Allowable Connection Type
< 0.6g	A, B, C
≥ 0.6g	A, B

C7.2.1.2

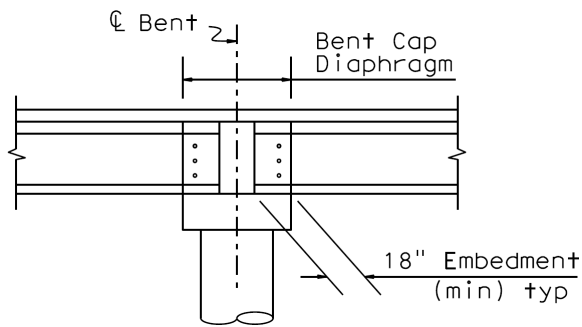
Precast girders made continuous for positive and negative moment continuity across supports are used in more seismically active areas (PGA ≥ 0.6g) in order to reduce the risk of unseating and to improve energy dissipation. As used herein, “continuity of reinforcement” includes reinforcement details across superstructure-to-cap joints that emulate a continuous cast-in-place (CIP) joint, as illustrated in Figure 7.2.1.2-1 (Type A and Type B). Research (Werff et al., 2015) has confirmed that precast girders with integral column-to-superstructure details effectively resist longitudinal seismic demands. Type A connection is considered the most structurally robust.

For Type C connections, the assumption of girder continuity is only valid for sites with PGA less than 0.6g.

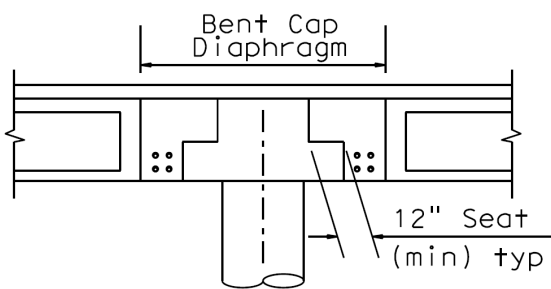
Designers should show an appropriate deck pour sequence to avoid excessive negative moment at the joints which may occur if bent cap joints are made integral prior to pouring the deck. Alternatively, the joint may be designed to accommodate large negative moments from construction staging.

The required reinforcement in the girder bottom to resist positive moment is placed during casting of the precast girders. The required top reinforcement is positioned in the top slab and made integral at the bridge site. Strands extended from PC girders cannot be used to meet negative seismic moment demands.

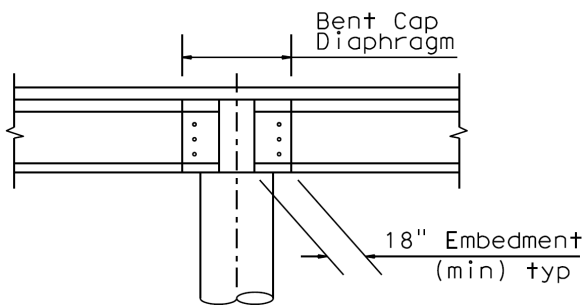
Historically, precast girders have often lacked a direct positive moment connection at the bents. This connection configuration is illustrated as Type C (Figure 7.2.1.2-1) and Type D (Figure C7.2.1.2-1). However, Type C is allowed in relatively low seismicity areas provided no moments are transferred through the pin connection. Type D connection is not allowed in order to prevent possible superstructure damage in seismic events.



(A1)



(A2)



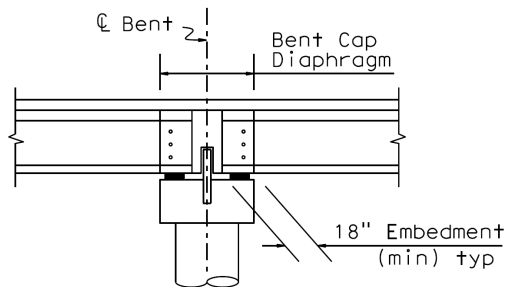
(A3)

- Substructure integral with superstructure
- Superstructure positive and negative moment continuity

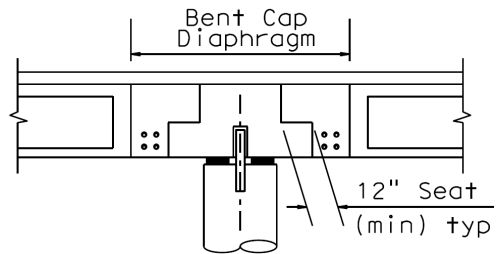
Type A

NOTES

1. All reinforcement and connection details not shown.
2. Drop cap width may be different from bent cap diaphragm width.



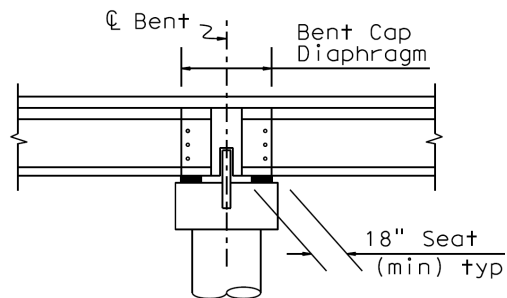
(B1)



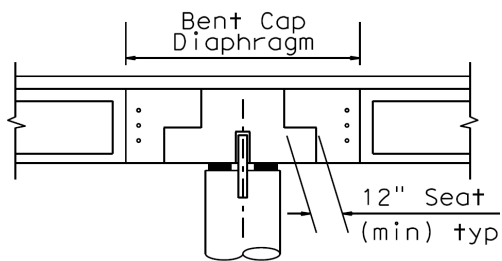
(B2)

- Superstructure pinned to top of column or drop cap (Base of SCM fixed)
- Superstructure positive and negative moment continuity
- Top of column fixed to drop cap (Figure B1)

Type B



(C1)

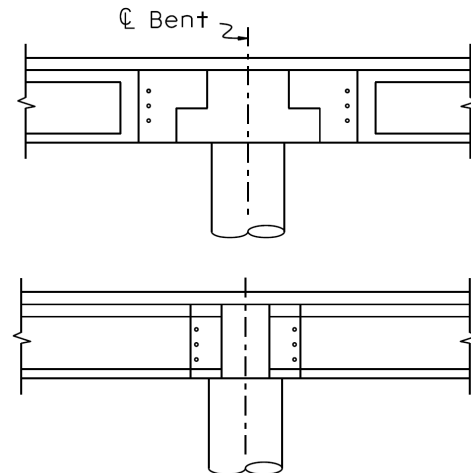


(C2)

- Superstructure negative moment continuity with a true pin connection
- Top of column fixed to drop cap (Figure C1)

Type C (for PGA < 0.6g only)

Figure 7.2.1.2-1 Allowable Precast Girder-to-Substructure Connections



- Superstructure negative moment continuity only
- Type D**

Figure C7.2.1.2-1 Typical Historical Precast Girder Connection

7.2.2 Reinforcement Requirements for Vertical Acceleration

Multi-span girder bridges located at sites with peak ground accelerations equal to or greater than 0.6g shall be designed for the effects of vertical ground excitation as specified herein. Moment demands induced by a uniformly applied vertical force equal to 25% of the dead load applied upward and downward as shown in Figure 7.2.2-1 shall not exceed the nominal flexural capacity of the superstructure based on the reinforcement types specified in Table 7.2.2-1. The superstructure at seat type abutments shall be modeled as pinned in the vertical direction.

The reinforcement used for vertical acceleration shall be evenly distributed and may be used to satisfy the requirements for other load cases. Any splice used shall be placed away from the critical zones (mid-spans and near supports) and shown on the plans.

The longitudinal side reinforcement in the girder webs across the bent cap/diaphragm shall consist of mild steel capable of resisting 125% of the dead load shear at the face of bent cap/diaphragm by means of shear friction. For CIP girders, the longitudinal side

C7.2.2

The intent of this provision is to ensure that all superstructure types have a nominal amount of reinforcement available to resist the demands from any vertical excitation. Mild steel is the preferred reinforcement for vertical acceleration because of its superior ductility over prestressing strands and should be used wherever there is enough space to place the bars, such as the locations shown in Table 7.2.2-1. Due to space constraints in most precast girders, prestressing strands are allowed to be used for vertical acceleration reinforcement as specified in Table 7.2.2-1.

The uniformly applied vertical force of 25% of the dead load should not be combined with the superstructure dead load.

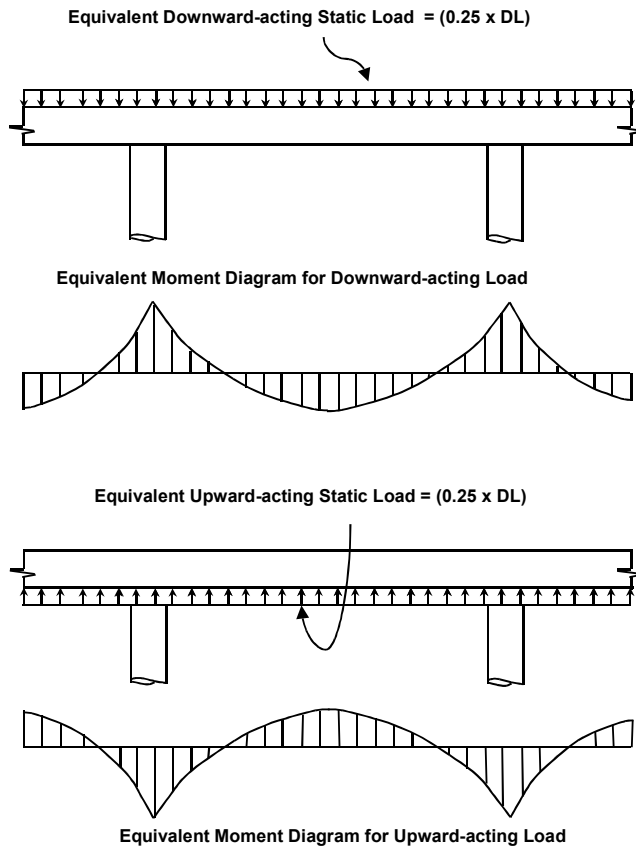


Figure 7.2.2-1 Equivalent Static Vertical Loads and Moments on a Segment of Superstructure for Vertical Acceleration

Table 7.2.2-1 Reinforcement Types for Vertical Acceleration

Loading	Location	Girder Type	
		Cast-in-Place	Precast
Downward-acting	Midspan	Mild steel and/or Prestressing strand	
	Bent	Mild steel	
Upward-acting	Midspan	Mild steel	
	Bent	Mild steel	Use provision for PC girder bottom reinforcement at supports in Section 7.2.1.2

reinforcement shall pass through the bent cap diaphragm and extend into the girder for a minimum of $1.5 D_s$ beyond the face of the bent cap diaphragm.

For precast prestressed girders, the longitudinal side reinforcement shall be embedded in the girder a minimum distance of $(D_s + l_{db})$ from the face of the diaphragm and developed into the diaphragm, where l_{db} is the basic tension development length, as specified in Section 5 of AASHTO-CA BDS.

7.2.3 Hinges

7.2.3.1 General

Polytetrafluoroethylene (PTFE) spherical bearings shall be used at all in-span bridge hinges. For seismic design, PTFE spherical bearings shall be positioned as shown in Figure 7.2.3.2-1. The bottom (i.e., supporting) corbel shall be designed for maximum service loads. Any changes in moment demand arising from opening of the hinge shall be considered in the design of the top (i.e., supported) corbel.

7.2.3.2 Support Length Requirements in the Longitudinal Direction

The support length normal to the centerline of bearing, N_H shall satisfy:

$$N_H \geq \begin{cases} MR + \Delta_{eq} + L_{mp} \\ \frac{D_s}{3} \\ 30 \text{ in.} \end{cases} \quad (7.2.3.2-1)$$

C7.2.3.1

In general, service loads will be greater than the seismic demands on in-span hinges. Therefore, in-span hinges are typically designed for non-seismic loads and checked for the effects of seismic loading. The hinge allows each frame to displace independently during an earthquake. Large relative displacements can develop if displacements of the bridge frames are out-of-phase. All frames including balanced frames or frames with small differences in mass and/or stiffness will exhibit some out-of-phase response. The objective of meeting the fundamental period requirements between adjacent frames is to prevent unbalanced loading of one frame by the adjacent frame.

Locating expansion joints between closely spaced adjacent bents such that the superstructures on either side of the joint are supported by cantilever beam action only, requires approval through a PSDC.

C7.2.3.2

Sufficient support length should be available to accommodate the anticipated thermal movement, prestress shortening, creep, and shrinkage (i.e., Movement Range, MR), as well as the relative longitudinal earthquake displacement demand between adjacent frames. The total anticipated joint movement, MR is obtained from the Joint Movement Calculation form.

The intent of the support length specified in

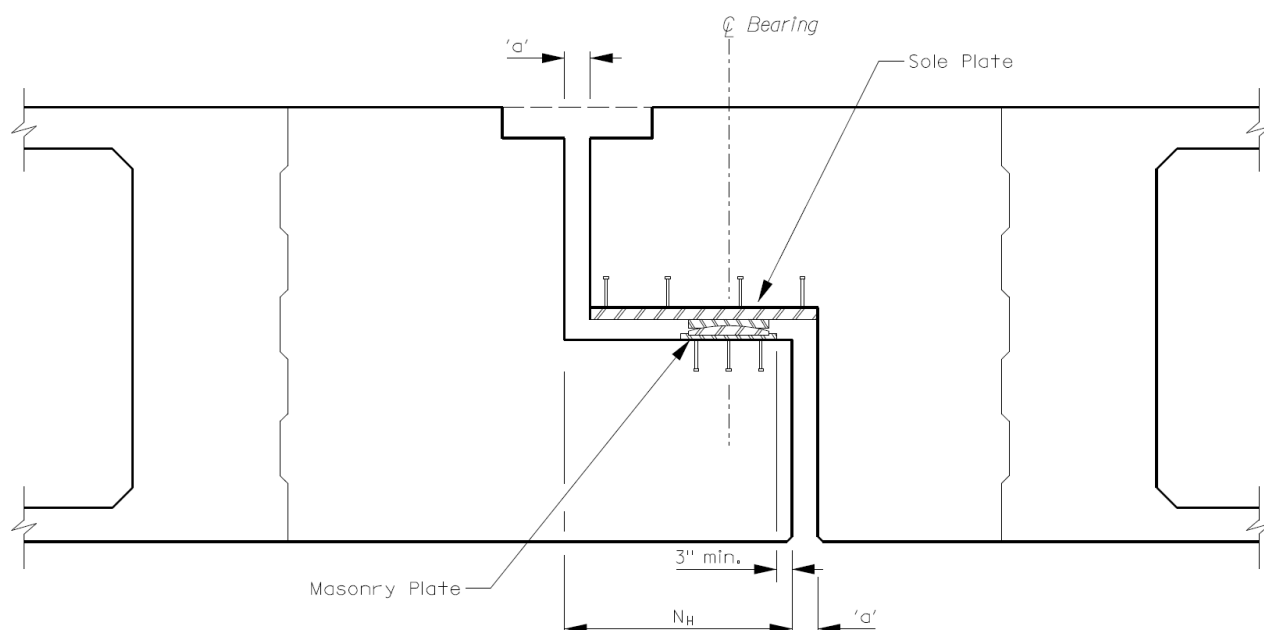


Figure 7.2.3.2-1 Support Length Requirements at In-span Hinges

where:

N_H = support length normal to the centerline of bearing (in.), as shown in Figure 7.2.3.2-1

L_{mp} = length of masonry plate for the bearing (in.)

Δ_{eq} = relative longitudinal earthquake displacement demand based on Safety Evaluation Earthquake (in.)

Δ_{eq} shall be obtained using either of the following methods:

Method 1:

Nonlinear Time History Analysis (NTHA) with elements that model gaps and impact on gap closure, and

Method 2: Analysis methods other than NTHA

$$\Delta_{eq} = \sqrt{(\Delta_{D1})^2 + (\Delta_{D2})^2 - 0.4\Delta_{D1}\Delta_{D2}} \quad (7.2.3.2-2)$$

where:

Equation 7.2.3.2-1 is to ensure that the sliding surface of the sole plate is in full contact with the concave plate of the bearing when the joint opens during the design seismic event.

The length of the bearing masonry plate is dependent on the load carried by each bearing. Therefore, the designer may adjust the support length by using an appropriate number of bearings across the hinge.

For bridges skewed at an angle, θ , the resulting support length measured along the longitudinal axis of the bridge is $(N_H / \cos \theta)$.

Elastic Analysis, in general, cannot be used to determine the displacement or force demands at the intermediate expansion joints in multi-frame structures. A more sophisticated analysis such as NTHA is required to capture relative frame displacements (Δ_{eq}). Hence Method 1 is preferred over Method 2.

For Method 2, the equation for Δ_{eq} is based on the results of a Caltrans' sponsored research (Desroches and Fenves, 1997) for the case of in-span

Δ_{D1} and Δ_{D2} = earthquake displacement demands on either side of the hinge (in.)

hinges without restrainers. Equation 7.2.3.2-2 differs from that used in SDC Version 1.7 by the inclusion of a cross correlation term. The coefficient of 0.4 used in the cross correlation term was obtained by assuming average values of target displacement demand ductilities and effective period ratios, (T_i/T_j) over the allowable period ratios specified in Section 7.1.3.

7.2.3.3 Shear Key Requirements in the Transverse Direction

C7.2.3.3

Shear keys shall be designed to transfer the shear between adjacent frames if the shear transfer mechanism is included in the demand assessment.

Typically, hinges are expected to transmit the lateral shear forces generated by small earthquakes and service loads. Determining the earthquake force demand on shear keys is difficult since the magnitude is dependent on how much relative displacement occurs between the frames.

Forces generated by any elastic model shall not be used to design shear keys. The transverse shear demand at the hinge shall be taken as the smaller of the sum of the overstrength shear demands of the columns in the bents adjacent to either side of the hinge.

EDA and ESA may predict force demands on the shear keys that differ significantly from the actual forces.

The minimum transverse shear key shall be:

The half-inch to three-quarter inch gap provided around the shear keys is intended to eliminate binding of the hinge under service operation and to ensure that lateral rotation will occur thereby minimizing moment transfer across the expansion joint.

- for multi-cell box girder vehicular bridges - two 8 x 8 x 5/8 in. hollow structural steel sections (i.e., 2 - HSS 8x8x5/8)
- for slab bridges, single-cell box girder vehicular bridges, and pedestrian bridges - two 5 x 5 x 1/2 in. hollow structural steel sections (i.e., 2 - HSS 5x5x1/2)

A half-inch to three-quarter inch gap shall be provided around the shear keys.

Although large relative displacements are not anticipated for frames with similar periods exposed to synchronous ground motion, certain structural configurations may be susceptible to lateral instability if the transverse shear keys completely fail. Particularly susceptible are: skewed bridges, bridges with three or less girders and narrow bridges with significant superelevation.

Internal concrete keys shall not be used.

Hollow structural steel section used as transverse shear keys will substantially increase the shear transfer capacity across expansion joints if significant out-of-phase displacements are anticipated.

The shear capacity of the hollow structural steel section may be determined in accordance with the provisions of Section 6 of *Caltrans Seismic Design Specifications for Steel Bridges* (Caltrans, 2016). Refer to Bridge Standard Details XS7-085 for hinge

transverse shear key details for multi-cell box girder vehicular bridges. For non-box girder bridges, a special transverse shear key detail is required.

7.2.3.4 Restrainers

Restrainers shall not be used to reduce the required support length at hinge seats in new bridges. Adequate support length as specified in Section 7.2.3.2, shall be provided to prevent unseating as a primary requirement.

C7.2.3.4

Hinge restrainers are not mandatory but may be useful in reducing bridge damage and/or excessive movement during small-to-moderate earthquakes. Restrainers are recommended in widenings where the existing bridge has already been retrofitted with restrainers.

Design of restrainers should not be based on the force demands predicted by Elastic Dynamic Analysis (EDA). BDA 14-1 provides an approximate method for determining the size and number of restrainers at expansion joints.

If the designer elects to use restrainers, the following guidelines shall be followed:

- A minimum of two restrainer units are required at each hinge and shall be symmetrically located at the exterior bays. Where possible, restrainer units shall be placed in alternating cells.
- Restrainers shall be detailed to allow for easy inspection and replacement
- Restrainer layout shall be symmetrical about the centerline of the superstructure
- Restrainer systems shall incorporate an adequate gap for expansion, which shall be shown on the Structure plans
- Permanent access openings for the restrainers shall be provided for inspection and repair
- Restrainers used in post-tensioned superstructures shall not be anchored prior to completion of the pre-stressing operation.

Adequate lengths of cables or bars should be used in order to assure sufficient elongation capacity. Hinge restrainer design should ensure that joint movements are kept within acceptable limits and restrainers work within the elastic range. Insufficient elongation ability can cause premature failure of the

restrainers while restrainers that are too long may not prevent unseating.

When placing restrainers on new structures with post-tensioned superstructure spans, the engineer should consider the shortening of the spans.

Yield indicators are required on all cable restrainers, as shown in Standard Detail Sheet XS 7-090. Refer to Bridge Standard Detail Sheets for drawings of different types of cable restrainer units.

For material properties pertaining to restrainer cables, refer to Federal Specification RR-W-410 - Wire Rope and Strand.

7.2.3.5 Pipe Seat Extenders

Pipes shall not be used as seat extenders in new bridges.

C7.2.3.5

Pipe seat extenders may be used for seismic retrofit of bridges.

7.2.4 Superstructure Depth

The superstructure depth, D_s , shall be measured at the face of the bent cap, and shall be taken as specified in Table 7.2.4-1.

Table 7.2.4-1 Depth of Superstructure

Superstructure Type	Analysis Direction	Superstructure Depth, D_s
Superstructure without drop cap	Transverse	Distance between top of deck and superstructure soffit
	Longitudinal	
Superstructure with drop cap	Transverse	Distance between top of deck and bottom of drop cap
	Longitudinal	Distance between top of deck and superstructure soffit

7.3 BENT CAPS

7.3.1 Integral Bent Caps

The effective width of integral bent caps for resisting flexural demands from plastic hinging in SCMs shall be as specified in Section 4 of AASHTO-CA BDS.

Bent cap widths shall satisfy Equation 7.4.3-1.

7.3.2 Non-Integral Bent Caps

7.3.2.1 General

Non-integral caps shall satisfy all requirements for frames in the transverse direction.

7.3.2.2 Minimum Bent Cap Width

Non-integral caps supporting superstructures with expansion joints at the cap shall have sufficient width to prevent unseating of the superstructure. The support length for non-integral bent caps shall satisfy Equation 7.2.3.2-1.

7.3.3 Bent Cap Depth

Bent cap depths shall be proportioned to meet the requirements of Sections 7.6.2 and 8.3.1.

C7.3.1

Bent caps are considered integral if they terminate at the outside of the exterior girder and respond monolithically with the girder system during dynamic excitation.

C7.3.2.1

Non-integral bent caps may provide support for simply supported superstructure spans or continuous spans with a separation detail such as an elastomeric pad between the bent cap and the superstructure.

C7.3.2.2

Continuity devices such as rigid restrainers or web plates may be used to ensure that unseating does not occur but can not be used in lieu of adequate bent cap width.

C7.3.3

With the exception of slab bridges, every effort should be made to provide enough cap depth to develop the longitudinal reinforcement of SCMs without using hooks or headed bars. Slab bridge requirements are specified in Section 9.

7.4 SUPERSTRUCTURE JOINTS

7.4.1 Joint Performance

Moment resisting connections between the superstructure and the column shall be designed to resist the column overstrength demands, M_o^{col} and V_o^{col} while remaining essentially elastic.

7.4.2 Joint Proportioning

All superstructure-to-column moment resisting joints shall be proportioned to satisfy:

$$\text{Principal tension: } p_t \leq 12\sqrt{f'_c} \quad (7.4.2-1)$$

(In Equation 7.4.2-1, f'_c and p_t have units of psi)

$$\text{Principal compression: } p_c \leq 0.25f'_c \quad (7.4.2-2)$$

in which:

$$p_t = \frac{(f_h + f_v)}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2} \quad (7.4.2-3)$$

$$p_c = \frac{(f_h + f_v)}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2} \quad (7.4.2-4)$$

$$f_h = \frac{P_b}{B_{cap} D_s} \quad (7.4.2-5)$$

$$f_v = \frac{P_c}{A_{jh}} \quad (7.4.2-6)$$

$$v_{jv} = \frac{T_c}{A_{jv}} \quad (7.4.2-7)$$

$$A_{jh} = (D_c + D_s) B_{cap} \quad (7.4.2-8)$$

$$A_{jv} = l_{ac,provided} (B_{cap}) \quad (7.4.2-9)$$

where:

A_{jh} = effective horizontal joint area

A_{jv} = effective vertical joint area

B_{cap} = bent cap width

C7.4.2

The principal stresses and forces acting on a superstructure-to-column joint are illustrated in Figure C7.4.2-1.

The limit of $p_c \leq 0.25f'_c$ is based on the need to prevent diagonal crushing of the joint core concrete. As a result of diagonal cracks induced by cyclically acting earthquake forces and the tensile strains in the joint shear reinforcement, concrete compressive strength is reduced (Priestley et al., 1996).

A negative result from Equation 7.4.2-3 signifies that the joint has nominal principal tensile stresses.

Unless the prestressing is specifically designed to provide horizontal joint compression, f_h can typically be ignored without significantly affecting the principal stress calculation.

Equation 7.4.2-9 defines the effective joint area in terms of the bent cap width regardless of the direction of bending. This simplified definition of A_{jv} may conservatively underestimate the effective joint area for SCMs with large cross section aspect ratios, in longitudinal bending.

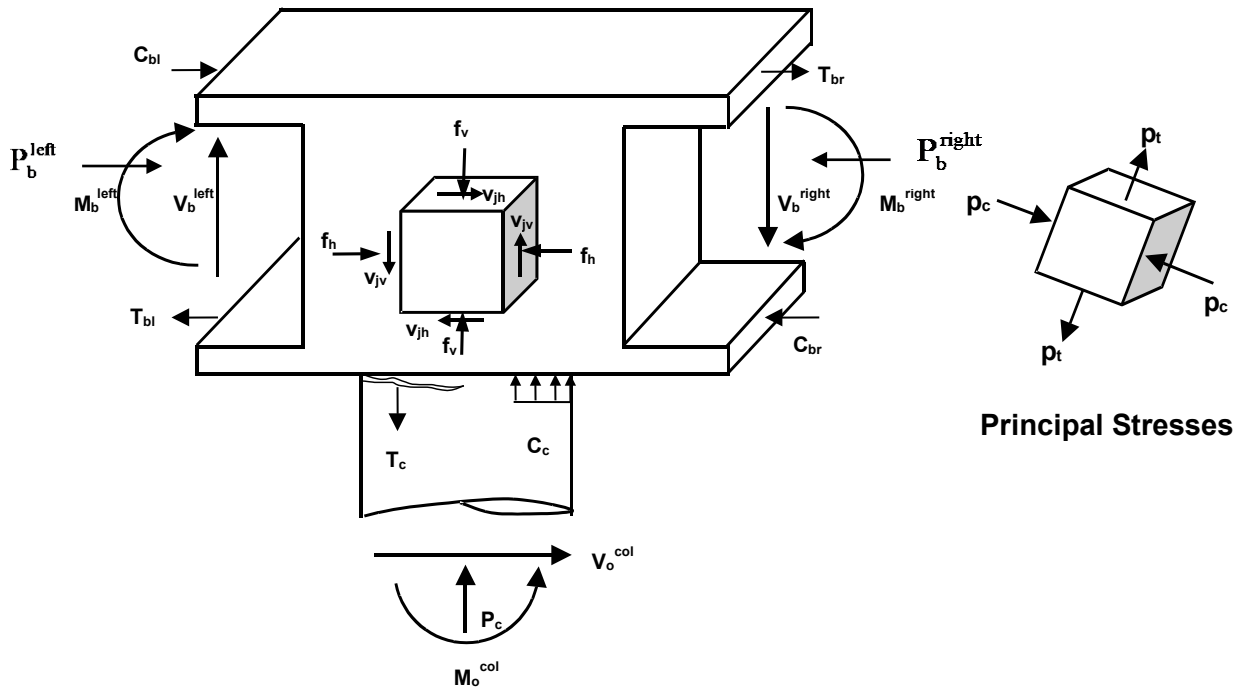


Figure C7.4.2-1 Joint Shear Stresses in T Joints

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

P_b = beam axial force (including prestressing) at the center of the joint

T_c = column tensile force defined as M_o^{col} / h , where h is the distance from c.g. of tensile force to c.g. of compressive force on the section. Alternatively, T_c may be obtained from the moment-curvature analysis of the cross section.

For slab bridges, A_{jh} and A_{jv} shall be determined in accordance with Section 9.5.

7.4.3 Minimum Bent Cap Width

The minimum bent cap width required for adequate joint shear transfer shall be taken as:

$$B_{cap} = D_c + 24 \quad (7.4.3-1)$$

where:

B_{cap} = bent cap width (in.)

D_c = diameter or cross section dimension of column in the direction of the cap width (in.)

7.4.4 Types of Joint

7.4.4.1 "T" Joints

The following types of joints shall be classified as "T" joints for joint shear analysis:

- 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 cap beam longitudinal reinforcement is fully developed beyond the exterior face of the column.

7.4.4.2 Knee Joints

An exterior column-to-superstructure joint shall be classified as a Knee joint if it satisfies:

$$S < D_c \quad (7.4.4.2-1)$$

where:

S = cap beam short stub length. S is equal to the distance from the exterior girder edge at soffit to the face of column measured along the bent centerline as shown in Figure 7.4.4.2-1.

For joint shear reinforcement design, knee joints shall be identified as:

C7.4.3

Larger cap widths may be required to accommodate joint shear reinforcement.

C7.4.4.2

Knee joints differ from T joints because the joint response varies with the direction of the moment (opening or closing) applied to the joint, as shown in Figure C7.4.4.2-1. Therefore, knee joints are evaluated for both opening and closing failure modes.

In the opening moment case, 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

Case 1: $S < \frac{D_c}{2}$ (7.4.4.2-2)

Case 2: $\frac{D_c}{2} \leq S < D_c$ (7.4.4.2-3)

competence of the anchorage of the outer column rebars and beam top rebars (Priestley et al., 1996).

In the closing moment case, 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 (Priestley et al., 1996).

It may be desirable to pin the top of the column to avoid moment transfer through a knee joint. However, the benefits of a pinned joint should be weighed against increased foundation demands and the effect on the frame's overall performance.

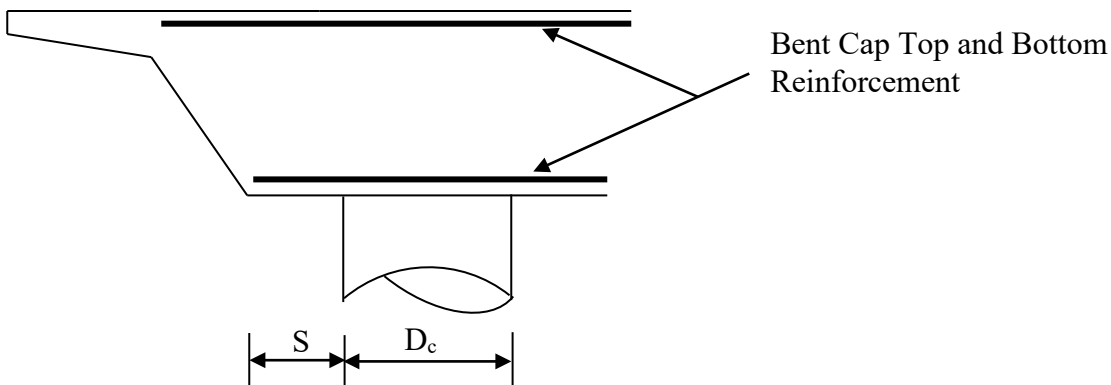
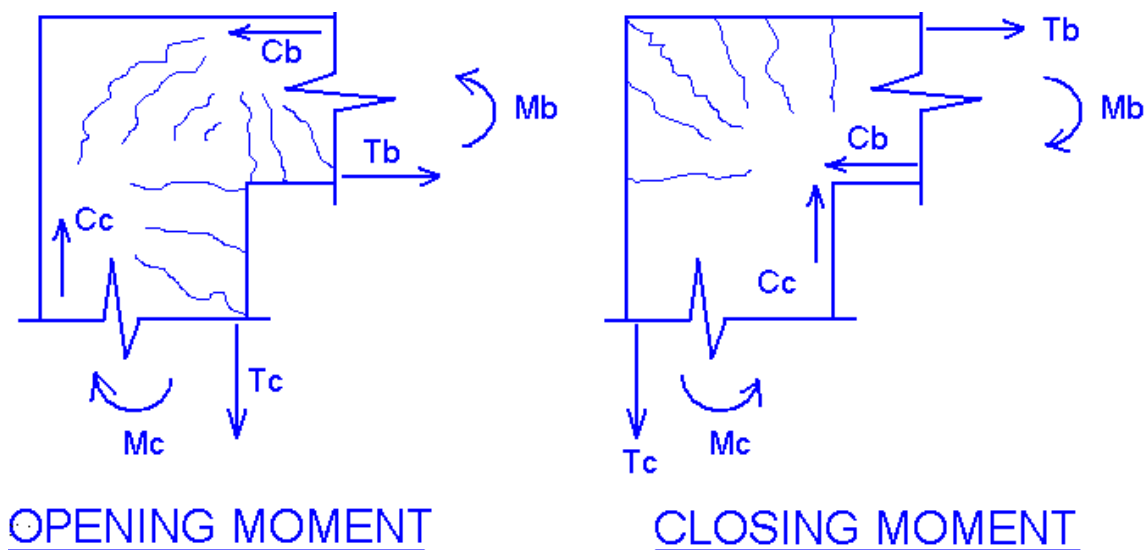


Figure 7.4.4.2-1 Knee Joint Parameters



Cb = beam compressive force, Cc = column compressive force, Tb = beam tensile force,
Tc = column tensile force, Mb = beam moment, Mc = column moment

Figure C7.4.4.2-1 Knee Joint Failure Modes

7.4.5 Joint Shear Reinforcement

7.4.5.1 Minimum Joint Shear Reinforcement

The minimum volumetric ratio of transverse column reinforcement, $\rho_{s,\min}$ continued into the bent cap shall be taken as:

$$\rho_{s,\min} = \frac{3.5\sqrt{f'_c}}{f_{yh}} \quad (7.4.5.1-1)$$

where:

f'_c = specified compressive strength of unconfined concrete (psi)

C7.4.5.1

Minimum joint shear reinforcement may be provided in the form of column transverse steel continued into the bent cap.

f_{yh} = specified minimum yield strength of transverse reinforcement (psi)

Where the principal tension stress, p_t is greater than $3.5\sqrt{f'_c}$, the joint shear reinforcement specified in Sections 7.4.5.2 (for T joints), 7.4.5.3 (for Knee joints), or 9.5 (for slab bridges) shall be provided.

Where $p_t \leq 3.5\sqrt{f'_c}$, only the minimum joint shear reinforcement shall be provided.

7.4.5.2 T Joint Shear Reinforcement

C7.4.5.2

A) Vertical Stirrups:

Vertical joint shear reinforcement, A_s^{jv} shall satisfy:

Figures 7.4.5.2-2, 7.4.5.2-3 and 7.4.5.2-4 illustrate the location of joint shear reinforcement.

A_s^{jv} may be used to meet other requirements documented elsewhere including the shear in the bent cap.

$$A_s^{jv} \geq 0.2A_{st} \quad (7.4.5.2-1)$$

where:

A_s^{jv} = area of vertical joint shear reinforcement

A_{st} = total area of column longitudinal reinforcement anchored in the joint (in.²)

Vertical stirrups shall be placed transversely within a distance D_c extending from either side of the column centerline. The vertical stirrups, A_s^{jv} shall be placed on each side of the column as shown in Figures 7.4.5.2-1, 7.4.5.2-2 and 7.4.5.2-4.

The stirrups provided in the overlapping areas in Figure 7.4.5.2-1 shall count towards meeting the requirements of both areas creating the overlap.

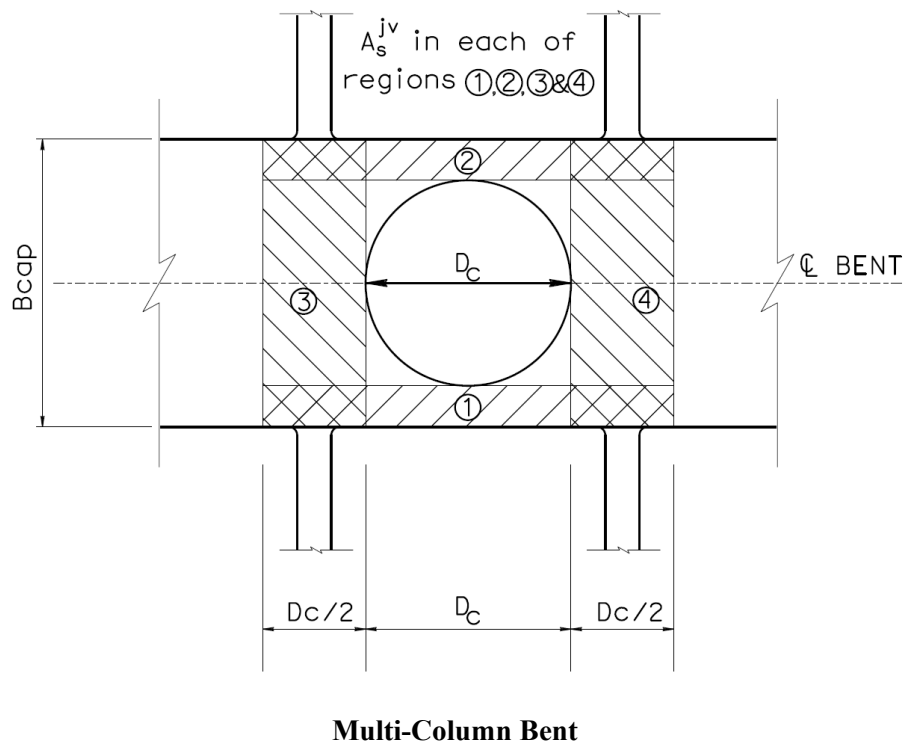
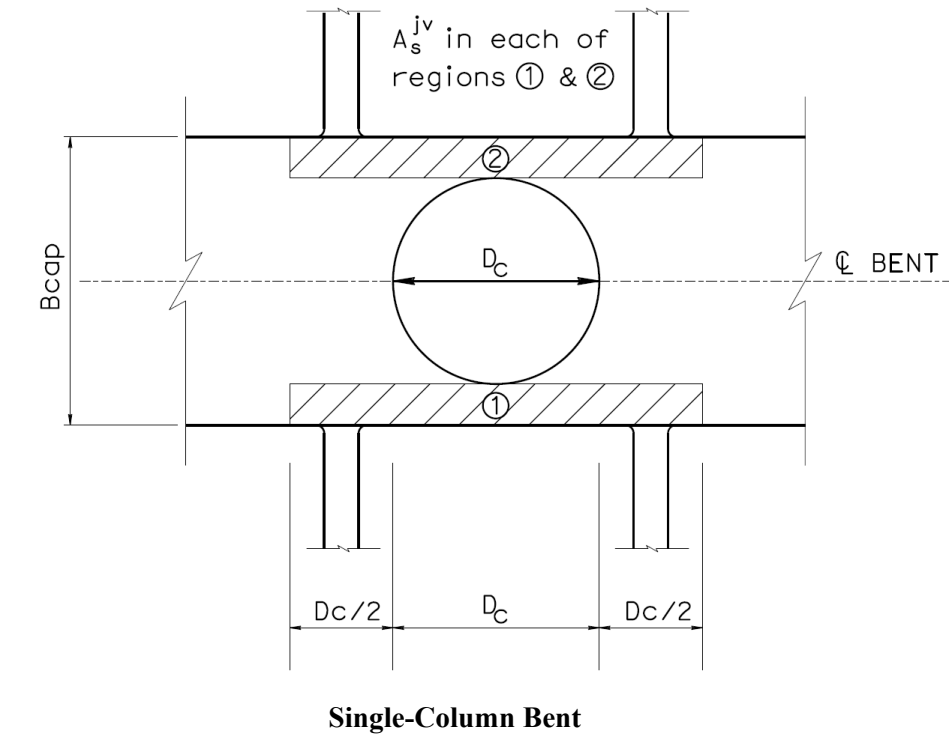


Figure 7.4.5.2-1 Location of Vertical Joint Reinforcement (Plan View of Bridge)

Bent Cap Details, Section at Column for Bridges with 0 to 20° Skew
 (Detail applies to sections within 2 x Diameter of SCM, centered about centerline of SCM).
 (Detail applies to T-Beam and Box Girder Bridges where deck reinforcement is placed parallel to Cap)

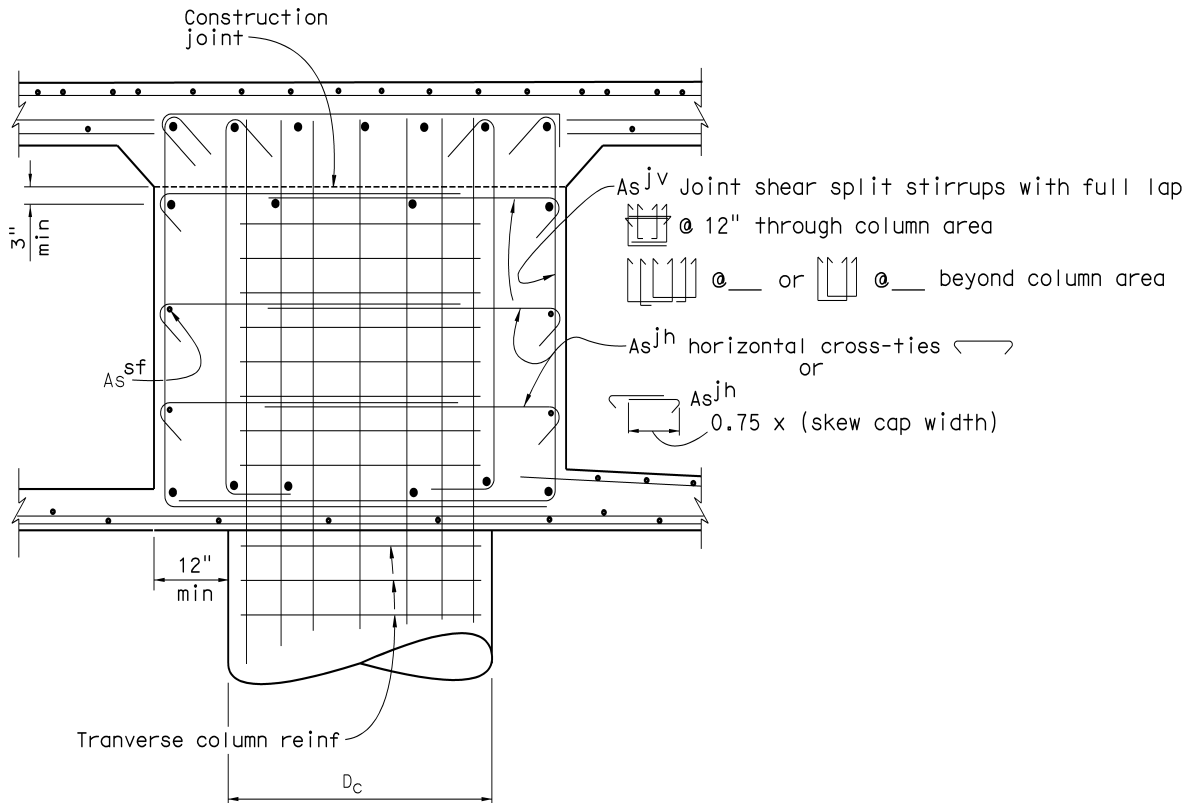


Figure 7.4.5.2-2 Joint Shear Reinforcement Details

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

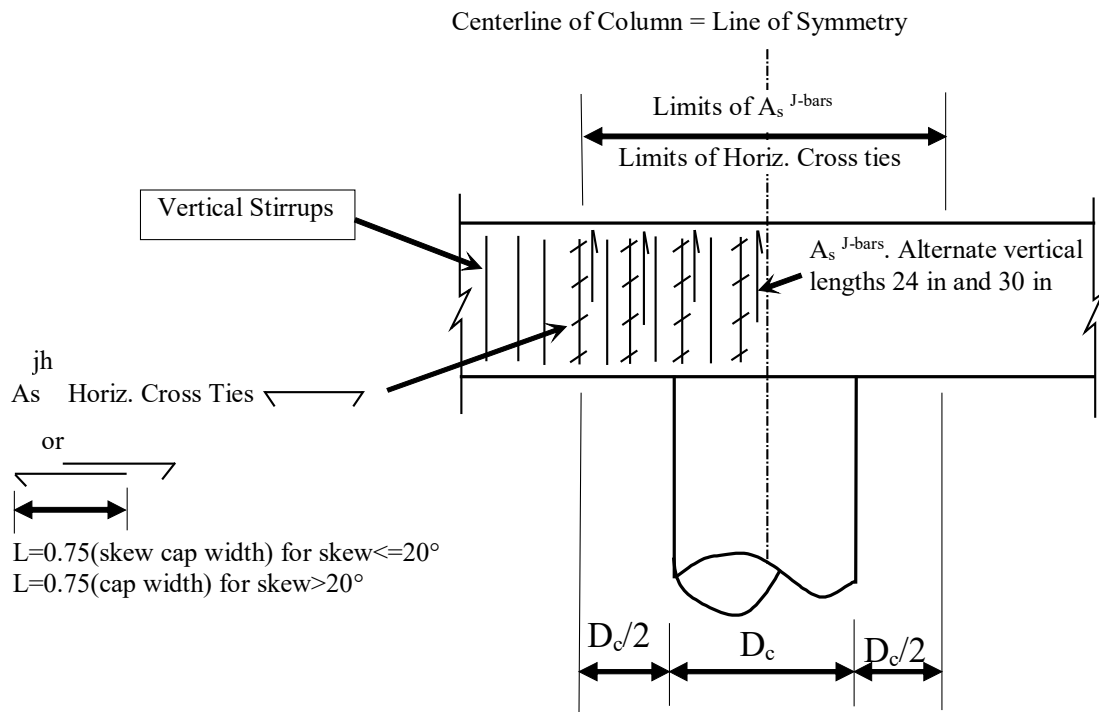


Figure 7.4.5.2-3 Location of Horizontal Joint Shear Reinforcement

Bent Cap Details, Section at Column for Bridges with Skew Larger than 20°
 (Detail applies to sections within 2 x Diameter of SCM, centered about CL of SCM).
 (Detail applies to T-Beam and Box Girder Bridges where deck reinforcement is placed normal or radial to CL Bridge)

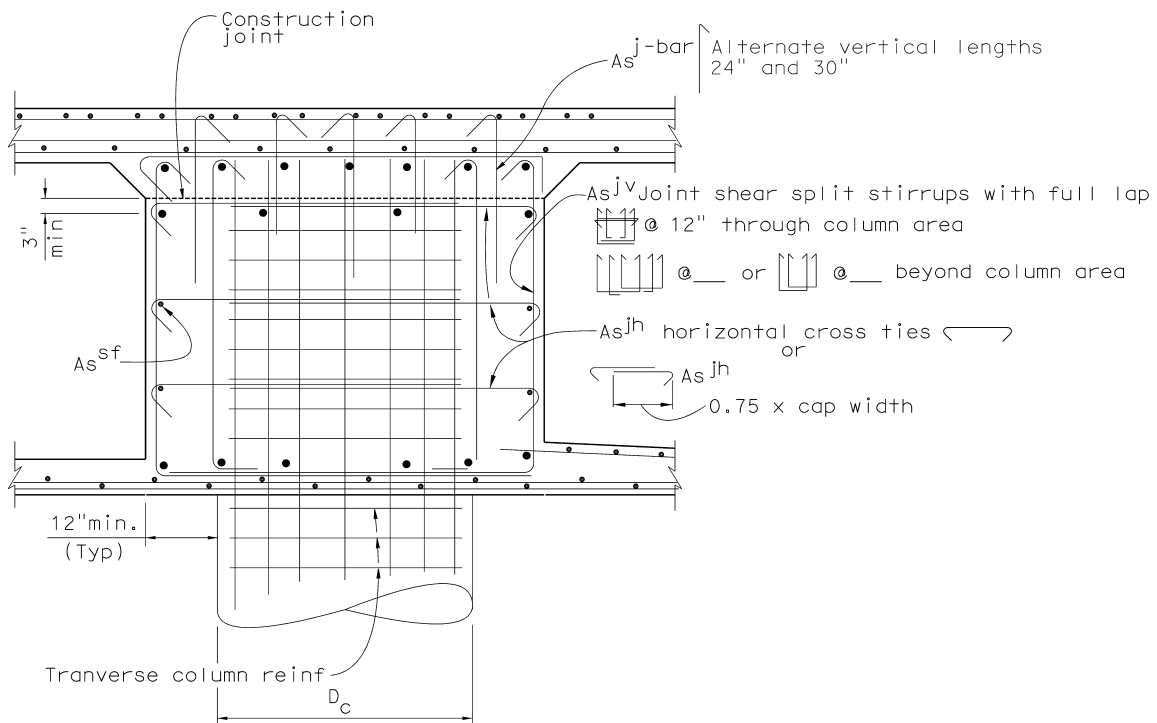


Figure 7.4.5.2-4 Joint Shear Reinforcement Details for Skewed Bridges

B) Horizontal Stirrups:

Horizontal stirrups or ties, A_s^{jh} shall satisfy:

$$A_s^{jh} \geq 0.1A_{st} \quad (7.4.5.2-2)$$

where:

A_s^{jh} = area of horizontal stirrups or ties

The 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. A_s^{jh} shall be placed within the distance D_c extending from either side of the column centerline, as shown in Figure 7.4.5.2-3.

C) Horizontal Side Reinforcement:

The total longitudinal side face reinforcement in the bent cap, A_s^{sf} shall satisfy:

$$A_s^{sf} \geq \max \begin{cases} 0.1A_{cap}^{top} \\ 0.1A_{cap}^{bot} \end{cases} \quad (7.4.5.2-3)$$

Fifty percent of A_s^{sf} is placed on each of the two side faces.

where:

A_{cap}^{top} = area of top flexural steel (in.²)

A_{cap}^{bot} = area of bottom flexural steel (in.²)

A_s^{sf} shall be placed near the side faces of the bent cap with a maximum spacing of 12 inches, as shown in Figures 7.4.5.2-2 and 7.4.5.2-4. Any side reinforcement placed to meet other requirements shall count towards meeting the requirement for A_s^{sf} .

D) J-Dowels

For bents skewed more than 20°, J-dowels shall be provided. The J-dowel reinforcement, A_s^{j-bar} shall satisfy:

$$A_s^{j-bar} \geq 0.08A_{st} \quad (7.4.5.2-4)$$

The J-dowels shall be hooked (with a 135° or greater hook) around the longitudinal top deck steel and extend alternately 24 inches and 30 inches into the bent cap.

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 SCM, as shown in Figures 7.4.5.2-3 and 7.4.5.2-4.

E) Transverse Reinforcement

Transverse reinforcement for the SCM extended into the bent cap shall consist of hoops. The volumetric ratio of the hoops, ρ_s^T shall satisfy:

$$\rho_s^T = \frac{4A_b}{D' S} \geq 0.4 \left(\frac{A_{st}}{l_{ac, provided}^2} \right) \quad (7.4.5.2-5)$$

For interlocking cores, ρ_s^T shall be based on the area of reinforcement, A_{st} of each core.

F) Anchorage for SCM Main Reinforcement

The longitudinal reinforcement of SCMs shall extend into the cap beam as specified in Section 8.3.1.1 to fully develop the compression strut mechanism in the joint.

7.4.5.3 Knee Joint Shear Reinforcement

The main bent cap top and bottom bars shall be fully developed from the inside face of the SCM and shall extend as closely as possible to the outside face of the bent cap, as shown in Figure 7.4.4.2-1.

The transverse reinforcement for the SCM extended into the bent cap may be used to satisfy this reinforcement requirement.

C7.4.5.3

Knee joint shear reinforcement details for straight (0 - 20° skew) and skewed (> 20° skew) bridge configurations are similar to those shown in Figures 7.4.5.2-2 and 7.4.5.2-4, respectively.

A) Bent Cap Top and Bottom Flexural Reinforcement (For Cases 1 and 2 Knee Joint)

Top and bottom reinforcement in the form of continuous “U” bars, A_s^{u-bar} shall be provided within the bent cap width, as illustrated in Figures 7.4.5.3-2 to 7.4.5.3-4. The area of A_s^{u-bar} shall satisfy:

$$A_s^{u-bar} \geq 0.33A_{st} \quad (7.4.5.3-1)$$

where:

A_{st} = area of column longitudinal reinforcement anchored in the joint (in.²)

Splices in the U bars shall not be located within a distance, l_d from the interior face of the column.

B) Vertical Stirrups (For Cases 1 and 2 Knee Joint)

Vertical stirrups or ties, A_s^{jv} shall be placed transversely within each of regions 1, 2, and 3 of Figure 7.4.5.3-1. The required area of A_s^{jv} shall satisfy:

$$A_s^{jv} \geq 0.2A_{st} \quad (7.4.5.3-2)$$

The stirrups provided in the overlapping areas of Figure 7.4.5.3-1 shall count towards meeting the requirements of both areas creating the overlap. A_s^{jv} may be used to meet other requirements documented elsewhere including shear in the bent cap.

The U bars may be combined with bent cap main top and bottom reinforcement using mechanical couplers.

See also Figure 7.4.5.3-5 for illustration of rebar placement.

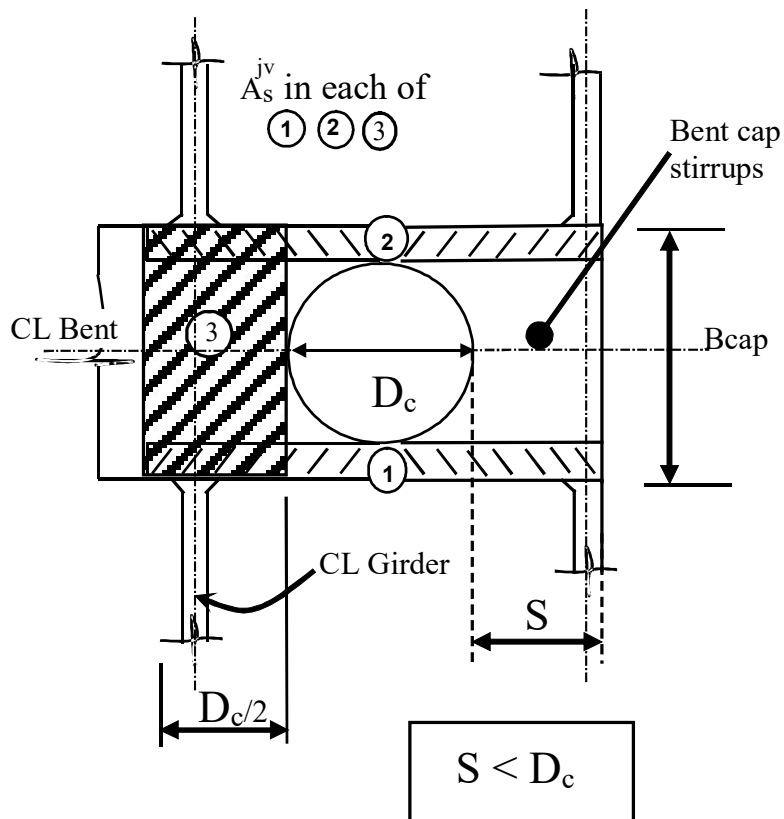


Figure 7.4.5.3-1 Location of Knee Joint Vertical Shear Reinforcement (Plan View)

C) Horizontal Stirrups (For Cases 1 and 2 Knee Joint)

Horizontal stirrups or ties, A_s^{jh} shall be placed transversely around the vertical stirrups or ties in two or more intermediate layers spaced vertically at not more than 18 inches, as illustrated in Figure 7.4.5.3-5. The required area of A_s^{jh} shall satisfy:

$$A_s^{jh} \geq 0.1A_{st} \quad (7.4.5.3-3)$$

The horizontal stirrups or ties shall be placed within the limits shown in Figures 7.4.5.3-2 and 7.4.5.3-3.

(D) Horizontal Side Reinforcement (For Cases 1 and 2 Knee Joint)

Longitudinal side face reinforcement, A_s^{sf} shall be placed near the side faces of the bent cap with a maximum spacing of 12 inches. The required area of A_s^{sf} shall satisfy:

$$A_s^{sf} \geq \max \begin{cases} 0.1(A_{cap}^{top}) \\ 0.1(A_{cap}^{bot}) \end{cases} \quad (7.4.5.3-4)$$

where:

A_{cap}^{top} = area of top flexural steel (in.²)

A_{cap}^{bot} = area of bottom flexural steel (in.²)

The 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.

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

Horizontal ties, A_s^{jhc} shall be placed at the end of the bent cap, as shown in Figures 7.4.5.3-2, 7.4.5.3-3, and 7.4.5.3-5. The required area of A_s^{jhc} shall satisfy:

$$A_s^{jhc} \geq 0.33A_s^{u-bar} \quad (7.4.5.3-5)$$

Horizontal ties 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.

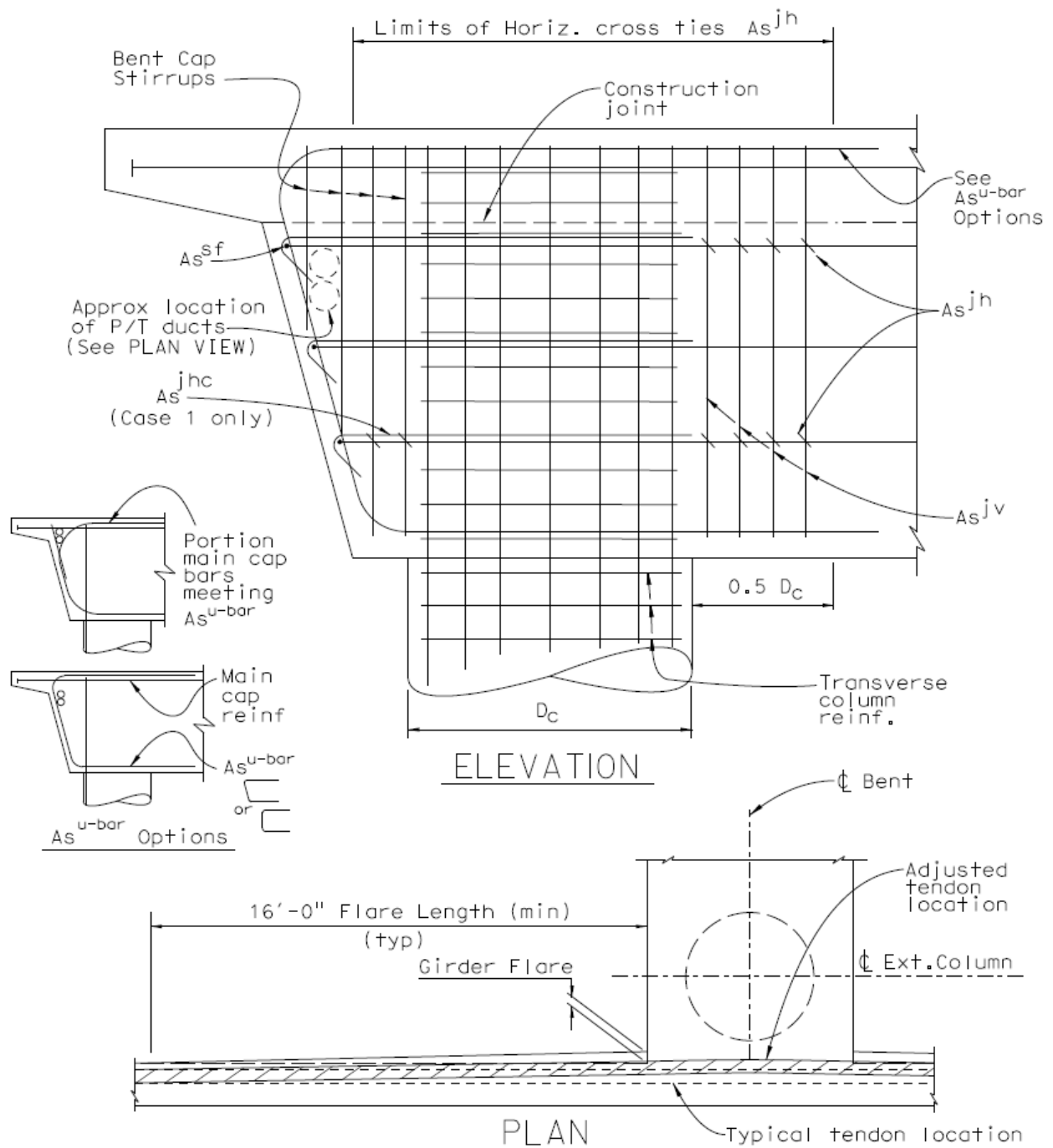
F) J-Dowels (For Cases 1 and 2 Knee Joint)

For bents skewed more than 20° , “J” bars (dowels) shall be provided, as shown in Figures 7.4.5.3-3, and 7.4.5.3-4. The area of A_s^{j-bar} shall satisfy:

$$A_s^{j-bar} \geq 0.08A_{st} \quad (7.4.5.3-6)$$

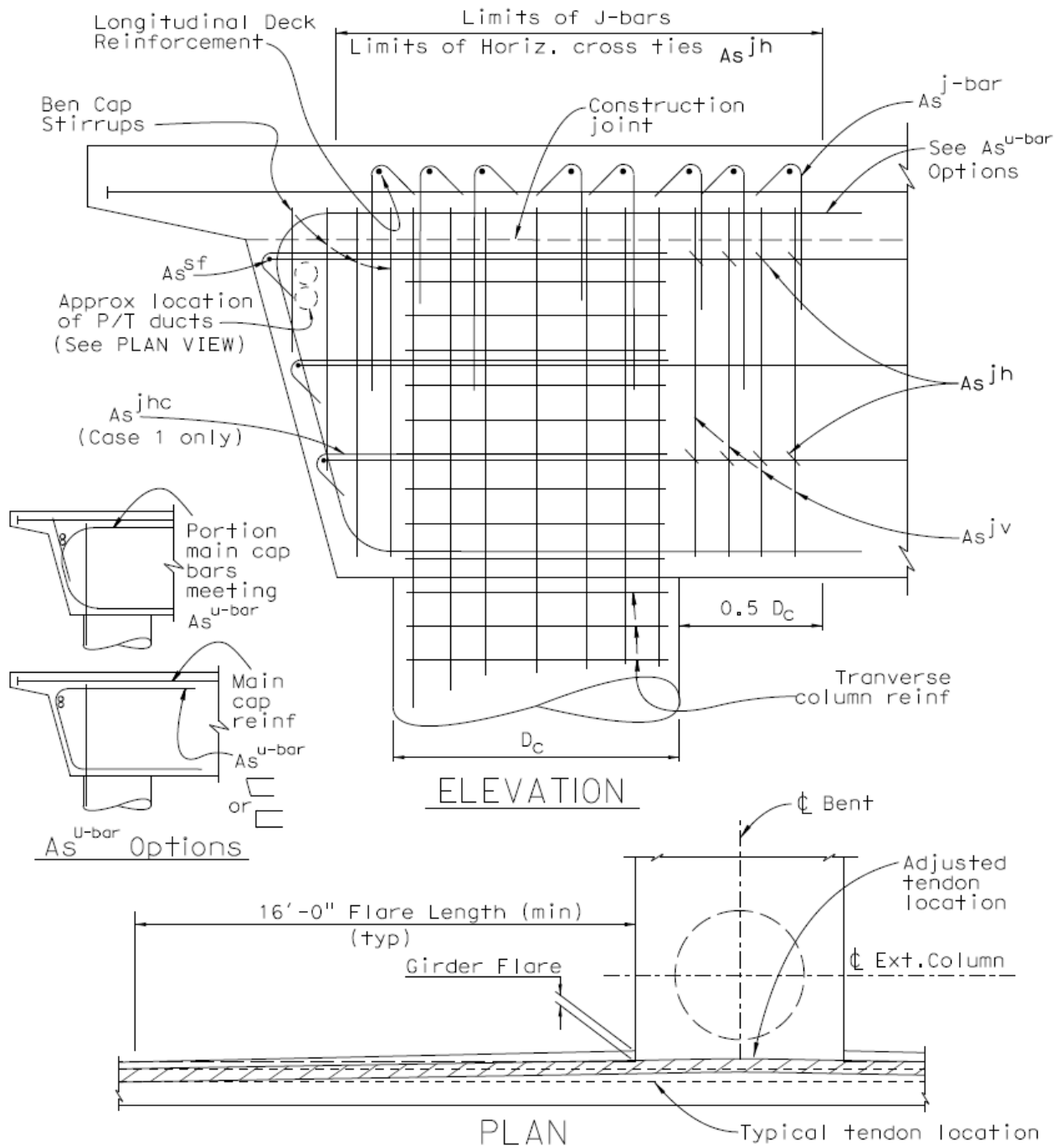
The “J” bars shall be hooked around the longitudinal top deck steel with at least a 135° hook and extend alternately 24 inches and 30 inches into the bent cap.

The J dowels shall be placed within a rectangular region defined by the bent cap width and the limits shown in Figure 7.4.5.3-3.

**NOTE:**

Flaring the inside face of exterior girders may be required for cast-in-place post-tensioned box girder construction in order to meet clearance requirements for ducts and U-bar reinforcement. To accommodate other girder and bent cap situations, it may be necessary to adjust rebar positions while ensuring required concrete covers are met.

Figure 7.4.5.3-2 Knee Joint Shear Reinforcement - Skew $\leq 20^\circ$



NOTE:

Flaring the inside face of exterior girders may be required for cast-in-place post-tensioned box girder construction in order to meet clearance requirements for ducts and U-bar reinforcement. To accommodate other girder and bent cap situations, it may be necessary to adjust rebar positions while ensuring required concrete covers are met.

Figure 7.4.5.3-3 Knee Joint Shear Reinforcement - Skew > 20°

G) Transverse Reinforcement

Transverse reinforcement in the joint region shall consist of hoops. The volumetric ratio of the transverse reinforcement, ρ_s^{knee} shall satisfy:

- For Case 1 Knee joint

$$\rho_s^{knee} = \frac{0.76A_{st}}{D_c l_{ac,provided}} \quad (7.4.5.3-7)$$

- For Case 2 Knee joint, Integral bent cap

$$\rho_s^{knee} = 0.4 \times \frac{A_{st}}{l_{ac,provided}^2} \quad (7.4.5.3-8)$$

- For Case 2 Knee joint, Non-integral bent cap

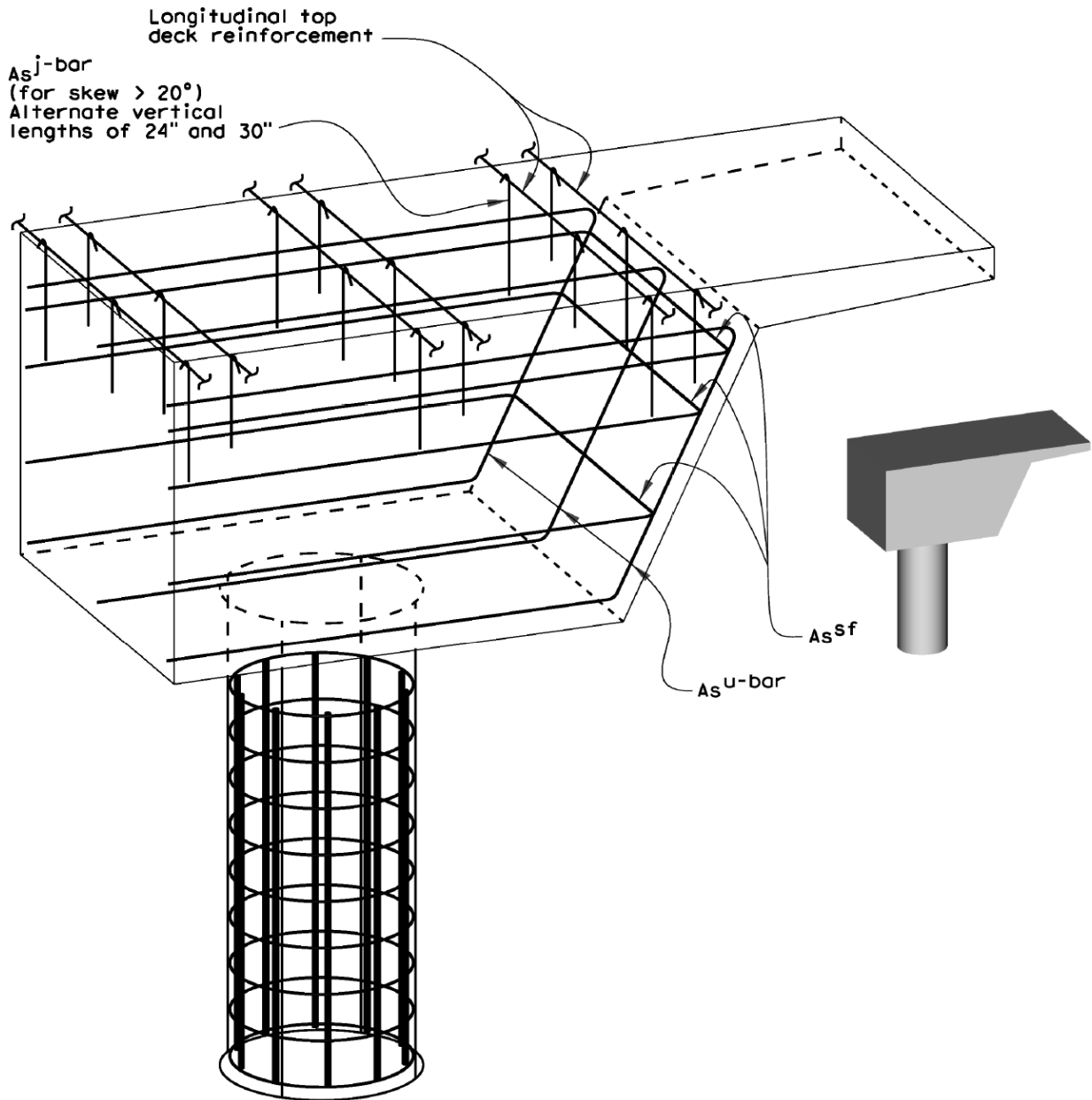
$$\rho_s^{knee} = 0.6 \times \frac{A_{st}}{l_{ac,provided}^2} \quad (7.4.5.3-9)$$

For interlocking cores, ρ_s^{knee} shall be calculated on the basis of A_{st} and D_c 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 closely as possible to the top bent cap reinforcement.

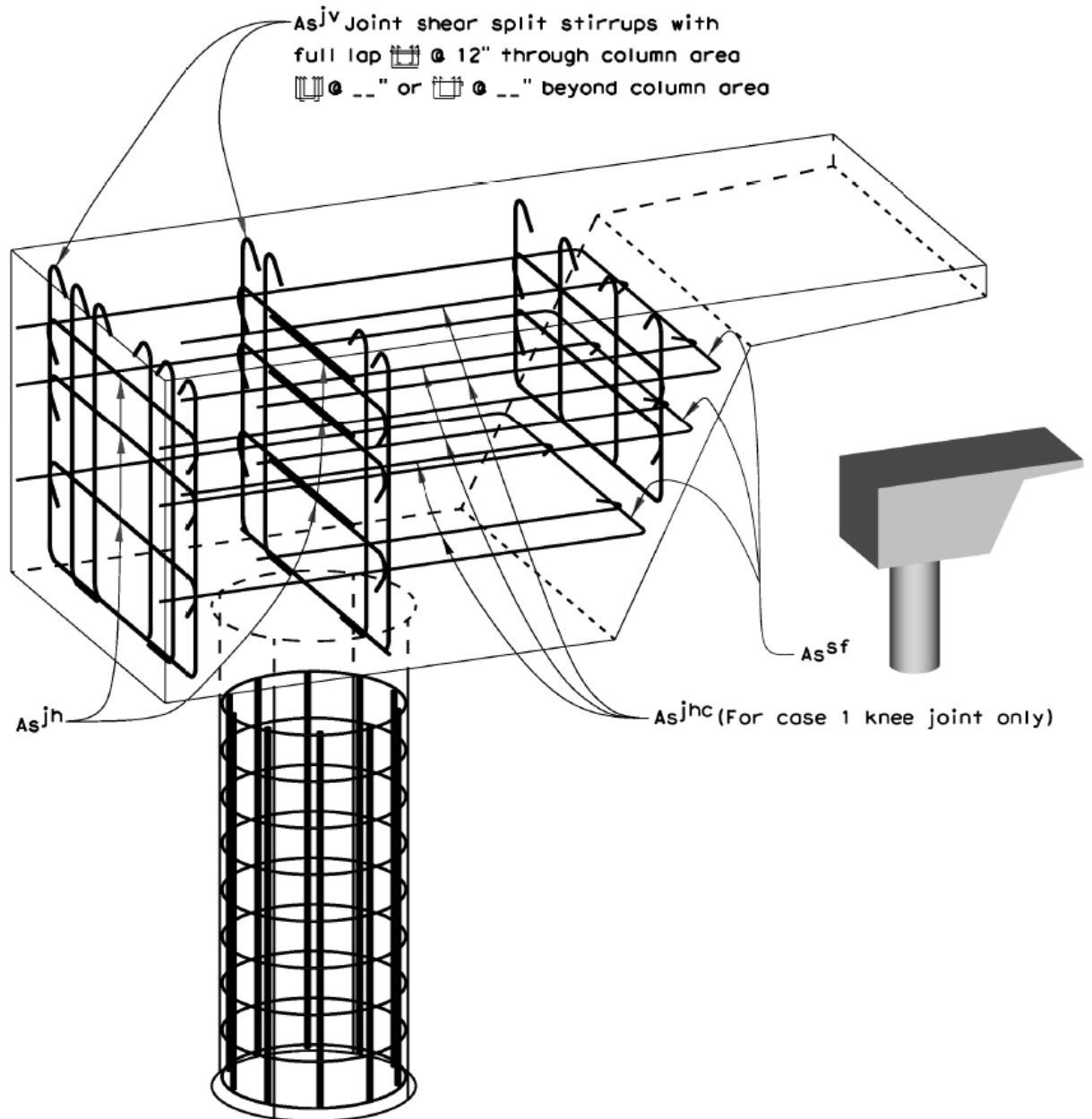
The transverse reinforcement for the SCM extended into the bent cap may be used to satisfy this requirement.

In Equations 7.4.5.3-7 to 7.4.5.3-9, the volumetric ratio of transverse reinforcement, ρ_s^{knee} is as defined in Section C5.3.8.2.

**Notes:**

- 1) See Figure 7.4.5.3-5 for 3-D representation of other knee joint shear bars not shown
- 2) Not all bars shown for each bar type
- 3) Column transverse and longitudinal reinforcement extended into bent cap not shown for clarity

Figure 7.4.5.3-4: 3-D Representation of Knee Joint Shear Reinforcement (A_s^{u-bar} , A_s^{j-bar} , A_s^{sf})

**Notes:**

- 1) See Figure 7.4.5.3-4 for 3-D representation of other knee joint shear bars not shown
- 2) Not all bars shown for each bar type
- 3) Column transverse and longitudinal reinforcement extended into bent cap not shown for clarity

Figure 7.4.5.3-5: 3-D Representation of Knee Joint Shear Reinforcement (A_s^{jv} , A_s^{jh} , A_s^{jhc} , A_s^{sf})

7.5 BEARINGS AND EXPANSION JOINTS

7.5.1 Bearings

Polytetrafluoroethylene (PTFE) spherical bearings shall be used at all in-span bridge hinges.

Bearings shall be checked to ensure that their displacement capacity and mode of failure are consistent with the assumptions made in the seismic analysis.

C7.5.1

Typically, elastomeric bearings are considered sacrificial elements while PTFE spherical bearings are capacity protected elements. Bearings should be detailed so that they can be easily inspected for damage and replaced or repaired after an earthquake.

It is preferable to use as many bearings as possible in order to ensure a balanced distribution of load across the hinge and reduce the load transmitted to each bearing.

Elastomeric and PTFE spherical bearings are permitted by the SDC without recourse to the PSDC procedure. Any of these bearing types can be used at bridge abutments because of the ease in replacing bearings at abutments. However, only PTFE spherical bearings are to be used at in-span hinges because of the difficulty in replacing bearings at in-span hinges. PTFE spherical bearings have relatively large displacement capacities and last longer than elastomeric bearings. PTFE spherical bearings are not susceptible to edge stress issues because rotation occurs along the spherical surface.

Elastomeric Bearings

The lateral shear capacity of elastomeric bearing pads is controlled by either the dynamic friction capacity between the pad and the bearing seat or the shear strain capacity of the pad. The dynamic coefficient of friction between concrete and neoprene may be taken as 0.40 while that between neoprene and steel may be taken as 0.35. The maximum shear strain resisted by elastomeric pads prior to failure is estimated at $\pm 150\%$.

Sliding Bearings

PTFE spherical bearings utilize low friction PTFE sheet resin. Typical friction coefficients for these bearings vary from 0.04 to 0.08. The friction coefficient is dependent on contact pressure, temperature, sliding speed, and the

number of sliding cycles. Friction values may be as much as 0.12 at sliding speeds anticipated under seismic loads.

A common mode of failure for sliding bearings under moderate earthquakes occurs when the PTFE surface slides beyond the limits of the sole plate often damaging the PTFE surface. The sole plate should be extended to eliminate this mode of failure whenever possible.

Seismic Isolation Bearings

Seismic isolation bearings are nonstandard bridge features and their use is subject to the PSDC procedure. Information on seismic isolation bearings is available in the draft MTD 20-22: Seismic Design of Ordinary Bridges with Isolation Bearings.

7.5.2 Seismic Expansion Joints

Recovery bridges shall be designed with seismic expansion joints. Seismic joints shall accommodate the required horizontal movements and rotations while maintaining their full functionality with little or no damage under the Functional Evaluation Earthquake (FEE).

C7.5.2

In contrast to seismic joints, conventional bridge joints are not designed for seismic loads and can only accommodate longitudinal and very limited transverse movement of the bridge deck, typically less than one inch. Conventional joints are expected to be damaged and replaced after a seismic event. Examples of seismic joints are the plate joint seal assemblies shown in XS sheets XS8-070 through XS8-100. Plate joint seal assemblies are capable of accommodating relatively large joint openings, thereby preventing traffic disruption even after a major seismic event.

Seismic joints may also be used for ordinary bridges with the following characteristics:

- Skew greater than or equal to 35 degrees
- Curved bridges when the joint transverse movement under service conditions is greater than 1 inch and no shear keys prevent such movement
- Bridges that cross fault lines or located in areas with tectonic movements
- Where bridge joints are required to maintain their functionality with little or no damage after a seismic event

7.6 COLUMNS

7.6.1 General

The longitudinal reinforcement for all columns with circular and square cross-sections shall be arranged in a circular array.

The longitudinal reinforcement for rectangular and oblong columns shall be arranged in an interlocking circular array.

Columns in single column bents should be analyzed, designed, and detailed as fixed at both column ends.

In general, columns in a multi-column bent that is monolithic with the superstructure should be designed as fixed at the top and pinned at the pile cap or footing. However, multi-column bents pinned to the superstructure shall be fixed at the pile caps or footings.

C7.6.1

In structures with drop bent caps, pinning the base of the columns is not recommended unless the designer can ensure that there is adequate framing action between the bent cap and the superstructure to ensure longitudinal stability of the bridge.

Columns supporting outrigger bents are an exception and are designed as fixed at the base and pinned at the top.

Typically, columns supported on Type I and II shafts are considered as fixed at the base.

Pinning the base of columns leads to a reduction in the foundation size and foundation costs. In comparison to a fixed base column, a pinned base column results in a softer structure leading to larger drifts (lateral displacement) particularly under seismic demands. Also, pinning the base may increase the moments at the top of columns under strength and service load combinations compared to those in a fixed-fixed column. Consequently, pinned columns may be subjected to higher moment magnification factors. The combined effects of increased moments at the fixed end and the moment magnification may lead to an increase in a column's longitudinal reinforcement.

Column design for seismic, strength, and service limit states may result in a dense reinforcement arrangement with consequent construction-related difficulties. If the column reinforcement exceeds acceptable limits, one or a combination of the following alternatives may be used to achieve adequate column design and bridge performance:

- Examine reinforcement configuration
- Increase the number of columns per bent
- Use oblong columns
- Use larger columns in multi-column bents
- Shorten span lengths and add bents
- Use higher strength concrete for columns
- Use pile shafts in lieu of footings

- Incorporate foundation flexibility in modeling and analysis
- Increase the elastic length of shorter columns
- Use lightweight concrete superstructure
- Add hinges in the superstructure of long prestressed structures
- Use 3-D analysis instead of Cantilever analysis to achieve more realistic load distribution and reduce $P - \Delta$ effects on single column bents

7.6.2 Column Dimensions

The diameter of circular columns, D_c shall be proportioned relative to the superstructure depth at the bent cap, D_s and the footing depth, D_{fig} to satisfy:

$$\left\{ \begin{array}{l} 0.7 \leq \frac{D_c}{D_s} \leq 1.0 \text{ (Sup without drop cap)} \\ \frac{D_c}{D_s} \leq 1.0 \quad \text{(Sup with drop cap)} \end{array} \right. \quad (7.6.2-1)$$

$$0.7 \leq \frac{D_{fig}}{D_c} \quad (7.6.2-2)$$

If a reduced column section is used at the column-to-superstructure or column-to-footing connection, D_c shall be taken as the diameter of the reduced column section.

7.6.3 Column Flares

7.6.3.1 Horizontally Isolated Column Flares

The top of flared column sections shall be horizontally isolated from the soffit of the bent cap.

The horizontal gap isolating the flare from the bent cap shall extend over the entire cross section of the flare excluding a core region equivalent to

C7.6.2

The relationship between column cross section dimension, D_c and bent cap depth, D_s and between D_c and footing depth, D_{fig} are based on engineering design experience.

Every effort should be made to limit the column cross sectional dimensions to the depth of the superstructure. If the column dimensions exceed the depth of the bent cap it may be difficult to meet the joint shear requirements as specified in Section 7.4.2, and the superstructure capacity requirements as specified in Section 5.4.4. Pinning the top of columns or using a reduced column section may be a viable alternative in these cases.

C7.6.3.1

Isolating the flare from the superstructure/cap beam soffit, as shown in Figure C7.6.3.1-1, allows the flexural hinge to form at the top of the flare, minimizing the seismic shear demand on the column. The added mass and stiffness of the

the prismatic column cross section.

The flare gap shall be large enough so that it will not close during a seismic event. The minimum gap thickness shall be 4 inches.

The gap thickness, G shall be based on the estimated ductility demand and corresponding plastic hinge rotation capacity.

isolated flare typically can be ignored in the dynamic analysis.

The horizontal flare isolation detail is easier to construct than a combined horizontal and vertical isolation detail and is preferred wherever possible.

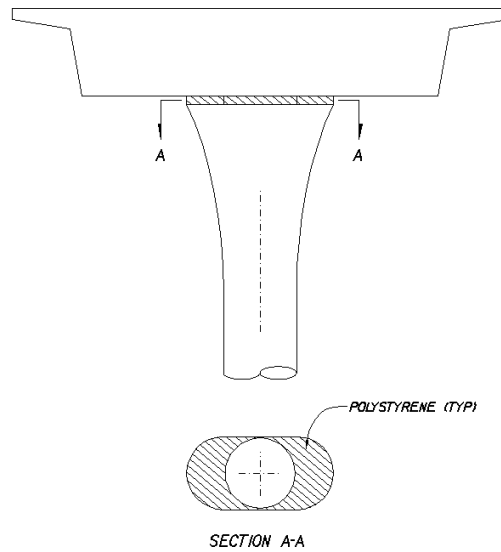


Figure C7.6.3.1-1 Horizontal Flare Isolation

7.6.3.2 Vertical Flare Isolation

Vertical flare isolation may be used in combination with horizontal isolation if the plastic hinge rotation (based on the equivalent plastic hinge length for horizontally-isolated flared columns as specified in Section 5.3.4) provides insufficient column displacement capacity.

C7.6.3.2

Vertical flare isolation is illustrated in Figure C7.6.3.2-1

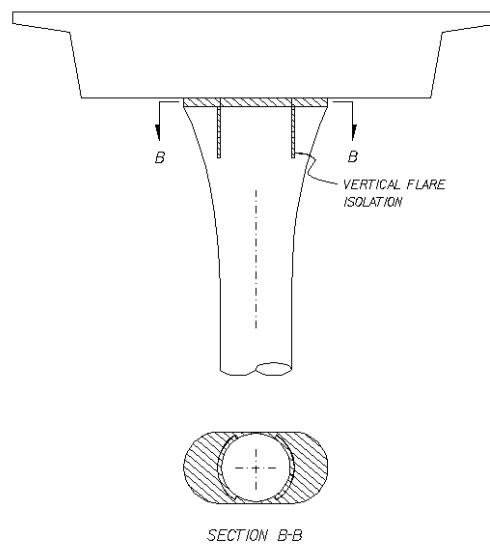


Figure C7.6.3.2-1 Vertical Flare Isolation

7.6.3.3 Lightly Reinforced Integral Flares

Lightly reinforced integral flares shall only be used when required for service load design or aesthetic considerations and the peak ground acceleration is less than 0.5g. The flare geometry shall be kept as slender as possible.

The column section at the base of the flare shall have adequate capacity to ensure that the plastic hinge will form at the top of the flare. Higher plastic hinging forces resulting from an integrally connected, lightly reinforced flare shall be considered in the design of the column, superstructure, and footing.

7.6.3.4 Flare Reinforcement

Column flares shall be nominally reinforced outside the confined column core to prevent the flare concrete from completely separating from the column at high ductility demands.

Splicing of transverse flare reinforcement shall satisfy the requirements of “service splice.” The splice location shall be shown on the Plans.

7.6.4 Column Key Design

Shear keys in pinned column connections shall be designed for the axial and shear forces associated with the column’s overstrength moment, M_o^{col} including the effects of overturning.

The area of interface shear key reinforcement, A_{sk} shall be taken as:

- If P is compressive:

$$A_{sk} = \frac{1.2(V_o^{col} - 0.25P)}{f_y} \quad (7.6.4-1)$$

- If P is tensile:

$$A_{sk} = \frac{1.2(V_o^{col} + P)}{f_y} \quad (7.6.4-2)$$

C7.6.3.3

Column flares that are integrally connected to the bent cap soffit should be avoided. Test results have shown that slender lightly reinforced flares perform adequately after cracking has developed in the flare concrete, essentially separating the flare from the confined column core. However, integral flares require higher shear forces and moments to form the plastic hinge at the top of the column compared to isolated flares.

C7.6.3.4

Refer to Attachment 1 of MTD 6-1 for flare reinforcement details.

Refer to Section C8.2.1 for additional information on rebar splicing.

C7.6.4

The key reinforcement shall be located as close to the center of the column as possible to minimize developing a force couple within the key reinforcement.

Equations 7.6.4-1 and 7.6.4-2 assume the use of normal weight concrete placed against a clean and intentionally roughened concrete surface.

where:

P = absolute value of the net axial force normal to the shear plane (kip). P shall be equal to the lowest axial load if compressive or greatest axial load if tensile, considering the effects of overturning.

The area of dowel reinforcement provided in the pinned connection to satisfy the column key design shall be greater than or equal to 4 in.²

Any moment generated by the key steel shall be considered in the design.

The designer shall indicate on the plans that the receiving concrete surface for the pin shall be intentionally roughened to an amplitude of 0.25 in.

The pin connection shall be proportioned such that the area of concrete engaged in interface shear transfer, A_{cv} , satisfies:

$$A_{cv} \geq \begin{cases} \frac{4V_o^{col}}{f_c'} \\ 0.67V_o^{col} \end{cases} \quad (7.6.4-3)$$

The area of concrete section used in the pin shall satisfy the axial resistance requirements as specified in Section 5 of AASHTO-CA BDS based on the column with the greatest axial load and a resistance factor ϕ of 1.0.

Adequate thickness of the expansion joint filler shall be provided around the column shear key to accommodate the maximum column rotation during a seismic event without crushing the edge of the column concrete against the capacity protected member.

In reality, most devices used to provide a pinned connection between the column end and the footing/cap beam will generate some moment which, if not accounted for in the design, may lead to unintended plastic hinging in the adjacent capacity protected member.

Concrete-filled steel pipe or solid bar sections may be used in lieu of reinforcing steel to relieve congestion and reduce the moment generated within the key. However, for columns in net tension, additional means should be employed to address uplift.

Corrosion of the pin device should be considered.



SEISMIC DETAILING

8.1 GENERAL

This Section provides the seismic requirements for splices, transverse reinforcement, and development of main flexural/longitudinal reinforcing steel. These requirements shall apply to both cast-in-place and precast members.

8.2 SPLICES IN REINFORCING STEEL

8.2.1 General

Reinforcing bar splices shall be either: "Ultimate Butt", "Service", or "Lap" as specified in Table 8.2.1-1. Locations of splices shall be clearly shown on the plans.

If a project includes Ultimate Butt splice and/or Service splice, the Design Engineer shall convey this information to the Specifications Engineer through a "Memorandum to Specifications Engineer and Estimator."

8.2.2 Reinforcement Splices in Seismic Critical Members

8.2.2.1 No-Splice Zones

The "No-Splice Zones" for SCMs shall correspond to the plastic hinge regions specified in Section 5.3.2. No-Splice Zones shall be clearly identified on the plans.

Except as specified herein, splicing of main flexural reinforcement shall not be permitted in No-Splice Zones.

C8.1

This section is intended to provide specifications on aspects of detailing that are most relevant to the efficient seismic performance of bridges. Other aspects of structural detailing are covered in other Caltrans documents.

C8.2.1

Specifications for splicing bar reinforcement are provided in the Standard Specifications (Caltrans, 2015).

Additional information on lap splices is available in BDD 13-16.

C8.2.2.1

The No-Splice Zone in seismic critical members may be shown on the plans either as a fixed dimension or as a fraction of the height or length of the member.

Table 8.2.1-1 Splice Requirements for Main Longitudinal Reinforcement

Member	Location	Splice Type
All SCMs	Inside plastic hinge region	Not allowed, except as specified in Section 8.2.2.1
	Outside plastic hinge region	Ultimate butt
Type II Shaft	Top 20 ft	Not allowed
	Elsewhere	Service
Piles within pile groups in class S1 soil	Everywhere	Lap
Bent Caps, Footings, and Pile Caps	Inside Critical Zone ^{*1}	Not allowed
	Outside Critical Zone ^{*1}	Service
Superstructures	Inside Critical Zone ^{*1}	Not allowed
	Outside Critical Zone ^{*1}	Service or Lap ^{*2}

^{*1} Critical zones for capacity protected members shall be taken as locations where the moment demand is greater than 75% of the maximum moment demand in the member.

^{*2} Service splice shall be used for reinforcement provided for longitudinal pushover analysis. Lap splice shall not be used to splice this reinforcement.

Splicing of main flexural reinforcement in No-splice zones may be permitted for relatively long SCMs where the length of commercially available reinforcing steel is inadequate to span the plastic hinge region. The length of commercially available reinforcing steel shall not be taken less than 60 ft.

The transverse reinforcement shall have the same area and spacing throughout the required length of No-Splice Zone.

8.2.2.2 Longitudinal Reinforcement

Splicing of main flexural reinforcement outside the No-Splice Zone shall meet the ultimate butt splice performance requirements as specified in Table 8.2.1-1.

For relatively long SCMs, the longest commercially available reinforcing steel length should be used to ensure that splicing within the No-splice zone is avoided or minimized. The allowable splice zone is the portion of the SCM outside the plastic hinge region.

Splicing of main flexural reinforcement shall not be allowed if the flexural reinforcement in the SCM can be placed with a single length of commercially available reinforcing bar.

8.2.2.3 Hoop Splices and Spiral Terminations

Transverse reinforcement in SCMs shall be either ultimate butt-spliced hoops or continuous spiral as specified in Table 8.2.2.3-1. Transverse reinforcement shall be used over the entire length of the seismic critical member.

When spiral reinforcement is used, the plastic hinge region shall be designated as the "No Splice Zone" for spiral reinforcement.

Spiral reinforcement at SCM-to-soffit and SCM-to-footing interfaces shall terminate in and begin with a "Special Hook" as shown in Figure 8.2.2.3-1 (Spiral Termination Type 1). The pitch of the spiral reinforcement at these locations shall be less than or equal to half the pitch of the spiral at a typical section. Special spiral hooks shall be shown on the plans.

For SCMs requiring spiral reinforcement longer than is available in a continuous roll, the spiral shall terminate and restart outside the plastic hinge region with spiral Termination Type 1.

Spiral reinforcement at the ends of SCM rebar cages shall terminate with either Termination Type 1 or 2 as shown in Figure 8.2.2.3-1. The termination detail shall be shown on the plans.

C8.2.2.3

Butt-splicing is achieved by the use of either welding or mechanical couplers.

The "No Splice Zone" for spiral reinforcement means that spiral reinforcement termination is not allowed at any other location in the plastic hinge region other than at the SCM-to-soffit and SCM-to-footing interfaces.

Spiral reinforcement is generally discontinued at the SCM-to-soffit interface to facilitate placement of main bars in the cap beam, and is restarted just above the soffit bars. Similarly, the spiral reinforcement may be discontinued at the SCM-to-footing interface. Since the spiral reinforcement is subjected to high strains at these locations of a SCM, a special hook is required.

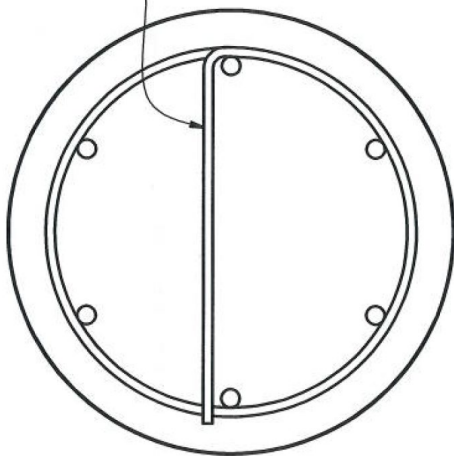
The spiral reinforcement at the ends of SCM rebar cages is subjected to relatively lower levels of strain.

A Special Hook, also referred to as Spiral Termination Type 1, as shown in Figure 8.2.2.3-1, is obtained by making an extra turn in the spiral coil in addition to a tail equal in length to the diameter of the rebar cage and passing through the core of the SCM.

Table 8.2.2.3-1 Applicability of Transverse Reinforcement in Seismic Critical Members

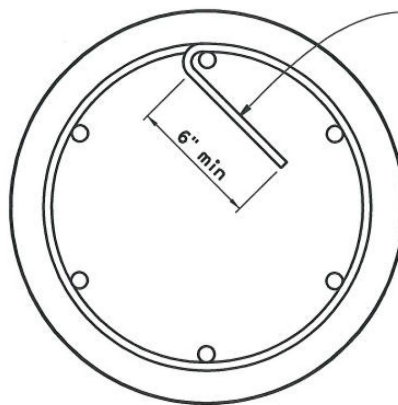
SCM	Transverse Reinforcement	
	Hoops	Spiral
Column	Required (Ultimate butt splice)	Not allowed
Type I shafts, Pile/Shaft group in class S2 soil, and Pile extensions	Allowed when rebar cage diameter \geq 14 in. (Ultimate butt splice)	Allowed when rebar cage diameter < 30 in.

Spiral terminations to have an extra turn with $\frac{1}{2}$ the pitch in addition to a 90° hook extended through column core, as shown. See Note 1



Spiral Anchor 90° Hook Detail

(A) Spiral Termination Type 1



Spiral terminations shall have 80 bar diameter lap in addition to a 6" long 135° hook. See Note 1

Spiral Anchor 135° Hook Detail

(B) Spiral Termination Type 2

Notes:

1. Hooks for spiral termination detail shall line up so as not to interfere with construction operation.
2. Inspection tubes are not shown

Figure 8.2.2.3-1 Spiral Termination Details

8.2.3 Reinforcement Splices in Capacity Protected Members

8.2.3.1 General

Reinforcing steel splices in capacity protected members shall meet the requirements specified in Table 8.2.1-1. Splices in main flexural reinforcement of CPMs shall be located away from the critical zones where the moment demands are greater than 75% of the maximum moment demands in the member.

8.2.3.2 Hoop Splices and Spiral Terminations

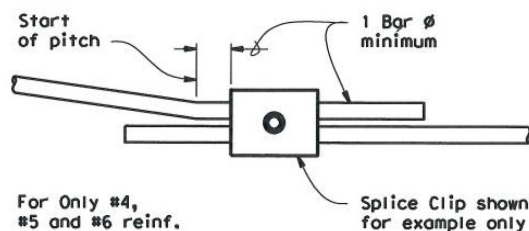
The splice and termination types for transverse reinforcement in capacity protected members shall be as specified in Table 8.2.3.2-1. The splice and termination detail shall be shown on the plans.

Combination of spiral reinforcement with hoops shall not be used except in the superstructure, where hoops may replace spirals in order to reduce rebar congestion.

The spiral reinforcement used in piles within a pile group in class S1 soil shall be continuous spiral with a termination Type 2 at the ends of the piles, as shown in Figure 8.2.2.3-1. Where a full roll of spiral reinforcement is inadequate for a pile, the spiral may be terminated and restarted with termination Type 2 or spliced with a mechanical splice.

C8.2.3.2

Where a full roll of spiral reinforcement is inadequate for a pile founded in class S1 soil, the mechanical lap splice detail shown in Figure C8.2.3.2-1 may be used to splice the spiral reinforcement.



Mechanical Lap Splice

Figure C3.2.3.2-1 Spiral Splice

Table 8.2.3.2-1 Applicability of Transverse Reinforcement in Capacity Protected Members

Capacity Protected Member	Transverse Reinforcement	
	Hoops	Spiral
Type II Shafts	Required with Ultimate butt splice	Not allowed
Piles within a Pile/Shaft group in class S1 soil	Allowed when rebar cage diameter \geq 14 in. (with Service splice)	Allowed when rebar cage diameter < 30 in.

8.3 DEVELOPMENT OF LONGITUDINAL REINFORCEMENT IN SEISMIC CRITICAL MEMBERS

8.3.1 Minimum Development Length of Longitudinal Reinforcement into Bent Caps and Footings

8.3.1.1 General

Longitudinal reinforcement in SCMs shall be extended into cap beams and footings as close as practically possible to the top surface of the cap beam and bottom surface of the footing.

Except for slab bridge superstructures, the minimum length of straight and hooked longitudinal bars extended into cap beams and footings shall be taken as:

$$l_{ac} = 24d_{bl} \quad (8.3.1.1-1)$$

where:

l_{ac} = minimum length of longitudinal reinforcement in SCM extended into cap beams or footings (in.)

d_{bl} = nominal bar diameter of longitudinal reinforcement (in.)

The minimum length of longitudinal bars with full-size headed bar terminations extended into cap beams and footings shall be taken as:

$$l_{ac} = 18d_{bl} \quad (8.3.1.1-2)$$

8.3.1.2 Epoxy-Coated Longitudinal Bars

For epoxy-coated longitudinal bars, the minimum development lengths shall be taken as the values determined by Equation 8.3.1.1-1 or 8.2.1.1-2 multiplied by a factor of 1.2.

C8.3.1.1

The minimum length of straight, hooked, and headed longitudinal bars extended into slab bridge superstructures are specified in Section 9.2.

While it is expected that the use of hooked bar termination will reduce the anchorage requirement specified in Equation 8.3.1.1-1, no such reduction is permitted until definitive test data on hooked bar termination become available.

In a study of development length provisions for column longitudinal bars extended into cap beams (Unanwa and Mahan, 2012), it was shown that, for the least expected concrete compressive strength of 5 ksi required by Section 3.3.6 (Equation 3.3.6-4), the minimum development length, $l_{ac} = 24d_{bl}$ is similar to those specified by AASHTO (2012) and Priestley (1996) but is conservative relative to AASHTO (2012) and Priestley et al. (1996) for higher concrete strengths.

8.3.1.3 Bundled Bars

The minimum development length of individual bundled bars in SCMs extended into a bent cap or footing shall be increased by twenty percent for a two-bar bundle and fifty percent for a three-bar bundle over the minimum development length of an individual unbundled bar. Four-bar bundles shall not be permitted in SCMs.

8.3.1.4 Confinement Along Development Length

The longitudinal reinforcement in SCMs shall be confined along the development length, l_{ac} by transverse hoops or spirals with the same volumetric ratio as that required at the top or bottom, as applicable, of the SCMs.

8.3.2 Embedment Length and Clearances for Column Reinforcement Extended into Type II Shafts

Longitudinal reinforcement in columns shall be embedded into Type II shafts, a minimum distance as specified in Table 8.3.2-1.

In Table 8.3.2-1:

$D_{c,max}$ = largest cross sectional dimension of the column (in.)

d_{bl} = nominal diameter of the column longitudinal bar (in.)

The portion of the column cage embedded in the shaft shall maintain the following reinforcing bar clearances:

- Shafts with Construction Joint
A 3.5 in. x 3.5 in. minimum clear window between the horizontal and vertical column reinforcing steel
- Shafts without Construction Joint
A 5 in. x 5 in. minimum clear window between the horizontal and vertical column reinforcing steel

The embedment lengths specified in Table 8.3.2-1 eliminate the staggered arrangement (i.e., alternate lengths) of the column longitudinal bars but result in development lengths comparable to those calculated with the previous SDC Version 1.7. The first term of the embedment length, $D_{c,max}$ is intended to mitigate potential penetration of splitting cracks into the shaft while the second term accounts for development of the column longitudinal bars. The specified development lengths meet or exceed the basic tension development lengths specified in AASHTO (2012).

The specified embedment length is conservative relative to that obtained through experimental and analytical research (Murcia-Delso et al., 2013), but was adopted to ensure an adequate factor of safety against potential splitting cracks and anchorage of the column longitudinal bars beyond the plastic hinge penetration in the shaft.

The designer should consider constructability issues if embedment lengths exceed 20 feet.

Table 8.3.2-1 Embedment Length of Column Bars into Type II Shafts

Bar size #	Embedment Length (in.)	
	Epoxy-coated bars	Non epoxy-coated bars
11 and smaller	$D_{c,max} + 50 d_{bl}$	$D_{c,max} + 42 d_{bl}$
14 and 18	$D_{c,max} + 58 d_{bl}$	$D_{c,max} + 48 d_{bl}$

8.3.3 Reinforcing Bar Hooks

Reinforcing bar hooks in seismic critical members shall be seismic hooks, defined as hooks with at least a 135° bend and a straight extension of at least $6d_b$ or 3 in., whichever is greater. Seismic hooks shall be detailed and shown on the plans.

8.4 SPACING OF REINFORCEMENT IN SEISMIC CRITICAL MEMBERS

8.4.1 Maximum spacing of Transverse Reinforcement

8.4.1.1 Inside the Plastic Hinge Region

The maximum spacing of transverse reinforcement in the plastic hinge regions shall be taken as the smallest of the following:

- Six times the nominal diameter of the longitudinal reinforcement
- Eight inches

8.4.1.2 Outside the Plastic Hinge Region

The maximum spacing of transverse reinforcement outside the plastic hinge regions where the shear demand in the SCM is greater than the shear capacity provided by the concrete, shall be taken as the smallest of the following:

- One half of the least dimension or diameter of the SCM
- Nine times the nominal diameter of the longitudinal reinforcement
- Ten inches

The maximum spacing of transverse reinforcement outside the plastic hinge regions where the shear demand in the SCM is less than the shear capacity provided by the concrete, shall be as specified in Section 5 of AASHTO-CA BDS.

C8.4.1.1

The maximum spacing requirements should prevent the buckling of longitudinal reinforcement between adjacent layers of transverse reinforcement.

8.4.2 Maximum and Minimum Lateral Spacing of Longitudinal Reinforcement C8.4.2

The maximum center to center lateral spacing of longitudinal reinforcement shall be 10 inches for SCMs with diameters less than or equal to 5 ft and 12 inches for SCMs with diameters larger than 5 ft.

Longitudinal reinforcing bars in the interlocking portion of a SCM shall have a maximum lateral spacing of 12 inches.

The minimum lateral spacing of longitudinal reinforcement in columns and piles shall be as specified in Section 5 of AASHTO-CA BDS.

The minimum lateral spacing of longitudinal bars in Types I and II shafts shall be as specified in Sections 5 and 10 of AASHTO-CA BDS.

Research has shown that the lateral spacing of longitudinal bars in circular reinforced concrete members has little effect on the effectiveness of the confinement and ductility of the member (Mander et al., 1988b; Papadopoulos and Shing, 2014) and that a circumferential spacing larger than the traditionally recommended maximum spacing of 8 inches can be used without a detrimental effect on flexural ductility and structural performance (Papadopoulos and Shing, 2014).

The maximum lateral spacing of longitudinal reinforcement specified herein is also applicable to Type II shafts.



SLAB BRIDGES

9.1 GENERAL

Slab bridges shall meet all the strength and ductility requirements of the SDC. If any provision of this section conflicts with other sections of the SDC, this section shall govern for the seismic design of slab bridges. Slab bridges designed on the basis of the design charts in BDA 4-10 shall satisfy all applicable seismic provisions of the SDC.

Slab bridges shall not be required to comply with the provisions of Sections 7.2.2 (Vertical Acceleration) and 7.2.3.3 (Hinge Shear Key transverse requirements).

9.2 SLAB DEPTH

The depth of slab superstructures shall be adequate to develop the longitudinal reinforcing bars from the supporting SCMs.

The minimum length of longitudinal reinforcing bars in SCMs extended into the cap beam shall be taken as:

- Straight bars: $24d_{bl}$
- Hooked bars: $19d_{bl}$
- Full size Headed bars: $11d_{bl}$

C9.1

The slab bridge provisions are for cast-in-place slab bridges and precast/voided slab bridges that emulate continuous cast-in-place slab bridges.

C9.2

Slab bridge superstructures usually have relatively shallow depths which also makes it difficult to satisfy seismic requirements. To ensure that slab depths are adequate to develop the longitudinal bars in SCMs and resist the overstrength demands generated at the top of SCMs while remaining essentially elastic, the following measures are recommended:

- Use a drop cap/haunched slab
- Increase the slab depth
- Pin the top of SCMs
- Use a reduced SCM section between the slab and end of the SCM

The minimum development length of $11d_{bl}$ for headed bars is only valid for #9 and smaller bar sizes. Research (Papadopoulos et al., 2015) has shown that embedment lengths as low as $8.7d_{bl}$ can develop the tensile strength of headed bars if punching damage can be controlled.

For the definition of Full size Headed bars, refer to Section C6.3.5.

9.3 EFFECTIVE WIDTH FOR JOINT SHEAR

The effective width of slab, B_{eff}^{slab} at the SCM-to-superstructure joint, resisting the overstrength moment of SCMs shall be taken as:

$$B_{eff}^{slab} = D_c + 2D_s \quad (9.3-1)$$

where:

D_s = depth of slab at the SCM support

The area of slab with the dimension, B_{eff}^{slab} in the transverse and longitudinal directions, as shown in Figure 9.3-1, shall define the regions for placement of joint shear reinforcement as follows:

- Core Region – region of slab-to-SCM joint directly above the SCM
- Joint Region - region of slab-to-SCM joint with plan dimensions of $(D_c + D_s)$ in both the longitudinal and transverse directions.
- Joint Perimeter - region of slab-to-SCM joint outside the “Joint” region defined above but within the effective slab area, $(B_{eff}^{slab} \times B_{eff}^{slab})$

For slab bridges with drop caps, the joint region for placement of joint shear reinforcement shall be defined by $(w_{cap} \times w_{cap})$, where w_{cap} is the width of the drop cap.

C9.3

If the effective width of slab, B_{eff}^{slab} for one column overlaps with B_{eff}^{slab} for an adjacent column, the entire width between the columns is effective.

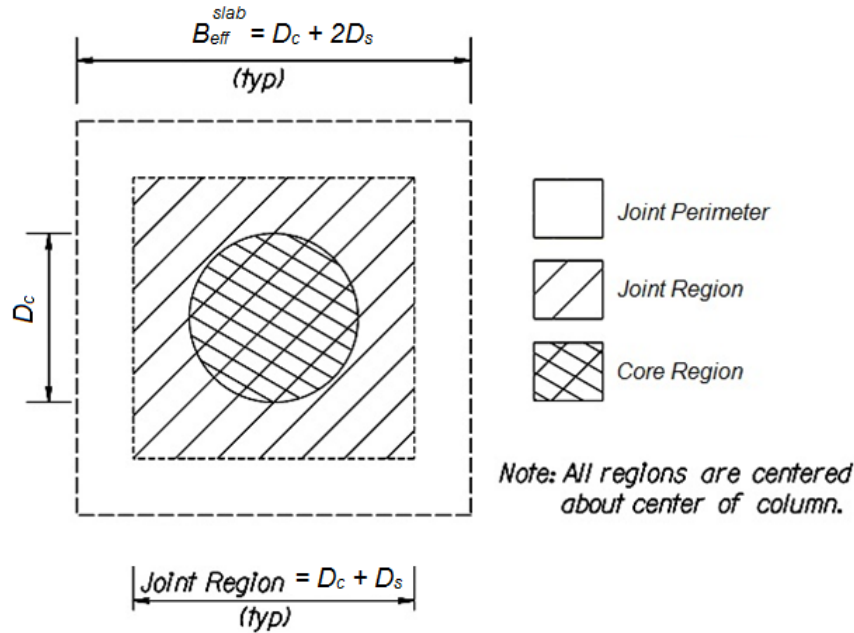


Figure 9.3-1 Slab Bridge Joint Shear Reinforcement Regions

9.4 DROP CAPS

Drop caps shall be designed to resist the overstrength demands of the supporting SCMs.

The width of drop caps shall not be greater than the effective width specified in Equation 9.3-1.

C9.4

Slab bridges with depths less than 16 inches should have drop caps in order to ensure adequate development of the longitudinal bars in SCMs.

9.5 JOINT SHEAR REINFORCEMENT

Moment-resisting slab superstructure-to-column joints shall satisfy the provisions of Sections 7.4.1 (Joint Performance), 7.4.2 (Joint Proportioning), 7.4.4 (Types of Joint), and 7.4.5.1 (Minimum Joint Shear Reinforcement).

The additional reinforcement required for joint shear in monolithic superstructure-to-SCM joints shall be as specified in Table 9.5-1 and illustrated in Figure 9.5-1.

Vertical stirrups, either as A_s^{j-bar} bars in the Core Region or A_s^{jv} in the Joint Region and Joint Perimeter, shall be provided at each intersection point of the joint shear bars. The stirrups shall have hooks

C9.5

For a slab bridge, the effective horizontal joint area, A_{jh} and effective vertical joint area, A_{jv} for use in Equations 7.4.2-8 and 7.4.2-9 are:

$$A_{jh} = \begin{cases} (D_c + 2D_s)^2 & \text{without drop cap} \\ (D_c + 2D_s)w_{cap} & \text{with drop cap} \end{cases} \tag{C9.5-1}$$

$$A_{jv} = \begin{cases} (D_c + 2D_s)l_{ac,provided} & \text{without drop cap} \\ w_{cap}l_{ac,provided} & \text{with drop cap} \end{cases} \tag{C9.5-2}$$

($\geq 135^\circ$) around the top longitudinal flexural reinforcement. A_s^{jv} bars shall also have 90° hooks around the bottom longitudinal flexural reinforcement.

The symbols and parameters in Table 9.5-1 shall be defined as:

A_{cap}^{top} = area of top flexural reinforcement in each direction within the effective width (in.²)

A_{cap}^{bot} = area of bottom flexural reinforcement in each direction within the effective width (in.²)

Flexural reinf. required = the flexural reinforcement required to force plastic hinge into the SCM (in.²), i.e., $M_{ne,slab} \geq 1.2M_p^{col}$, in which $M_{ne,slab}$ is the slab nominal moment capacity based on expected material properties.

A_{st} = area of column longitudinal reinforcement anchored in the joint (in.²).

Add. Reinf. = additional joint shear reinforcement (in.²). The limits of the additional reinforcement shall be shown on the plans.

A_s^{u-bar} = area of additional bent cap reinforcement (u-shaped) for knee joints in the transverse direction (in.²).

A_s^{j-bar} = area of “J” shaped bars inside the Core Region above the SCM (in.²). A_s^{j-bar} bars shall extend down to the top of the column.

A_s^{jv} = total area of vertical stirrups in each of the Joint Region and Joint Perimeter (in.²). The stirrups shall be uniformly distributed.

A_s^{jh} = total area of horizontal ties in the effective width along the longitudinal direction

Two sets of at least two vertically distributed ties shall be placed within the Core Region and one set of at least two vertically distributed ties shall be placed at each edge of the Joint Region.

Testing and modeling of pile/shaft-to-slab T-joint and Knee-joint connections (Ayoub and Sanders, 2010; Papadopoulos et al., 2015) have demonstrated that appropriately reinforced joint regions will limit damage and force plastic hinging into the SCM. However, the use of knee joints is discouraged for slab bridges.

In Table 9.5-1, 100% of the additional reinforcement, Add. Reinf., is placed in the top for A_{cap}^{top} and 100% is placed in the bottom for A_{cap}^{bot} .

The required flexural reinforcement to force the plastic hinge into the SCM may be detailed by using appropriate bar sizes larger than those required by BDA 4-10.

When a reduced column section between the slab and end of the SCM is used, A_{st} is the area of longitudinal steel connecting the top of the SCM to the slab.

Additional joint shear reinforcement is illustrated in Figure C9.5-1.

A_s^{sf} = total area of side face reinforcement

One set of at least two vertically distributed bars shall be placed at each side of the joint effective width along the transverse direction.

A_s^{jhc} = total area of horizontal end ties along the transverse direction for knee joints

$A_{v,joint}$ = SCM transverse reinforcement in the joint region

The number of hoops for the transverse steel in the joint region shall be determined as:

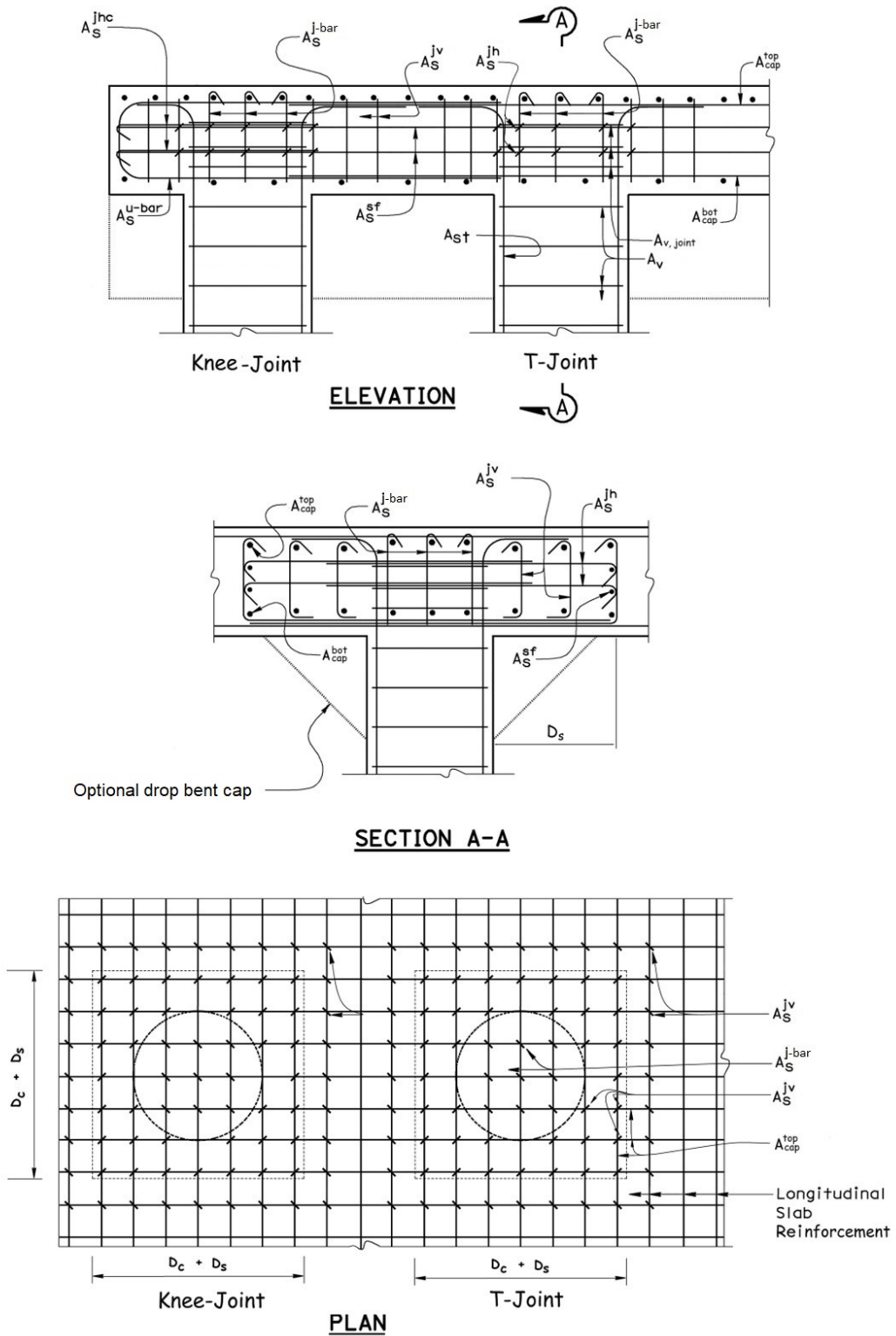
$$\text{Number of hoops} = \frac{0.18A_{st}}{\frac{\pi}{2}(A_b)} \quad (9.5-1)$$

where:

A_b = cross-sectional area of an individual transverse bar.

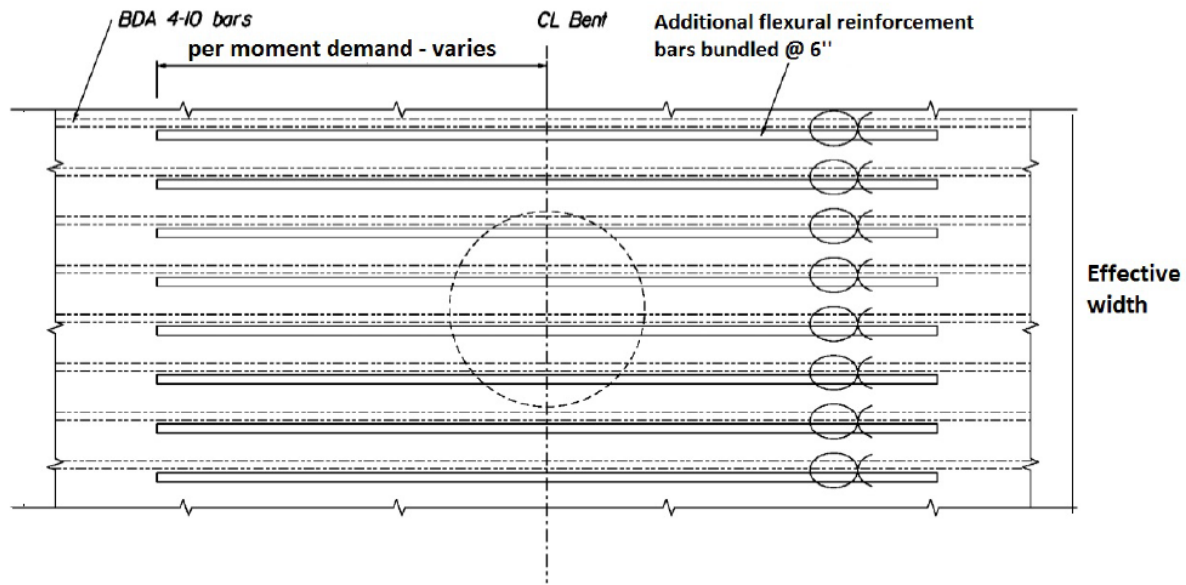
Table 9.5-1 Reinforcement for SCM-to-Slab Superstructure Joints

Reinforcement Name/Location	Slab T-Joints (see Figure 9.5-1)	Slab Knee-Joints (see Figure 9.5-1)
Area of flexural reinforcement in the longitudinal and transverse direction within the effective width	$A_{cap}^{top}, A_{cap}^{bot} = [\text{Flexural reinf. required} + \text{Add. reinf.}]$ Add. reinf. = $0.25A_{st}$ (drop cap) Add. reinf. = $0.35A_{st}$ (flat slab)	$A_{cap}^{top}, A_{cap}^{bot} = [\text{Flexural reinf. required} + \text{Add. reinf.}]$ Add. reinf. = $0.35A_{st}$ (long. dir.) $A_s^{U-bar} = 0.35A_{st}$ (trans. dir.)
J-bars in the core zone of joint region	$A_s^{j-bar} = 0.35A_{st}$	$A_s^{j-bar} = 0.35A_{st}$
Total area of vertical stirrups in each of Joint region and Joint Perimeter	$A_s^{jv} = 1.15A_{st}$	$A_s^{jv} = 0.85A_{st}$
Horizontal ties	$A_s^{jh} = 0.1A_{st}$	$A_s^{jh} = 0.1A_{st}$
Horizontal side reinforcement	$A_s^{sf} \geq \begin{cases} 0.1A_{cap}^{top} \\ or \\ 0.1A_{cap}^{bot} \end{cases}$	$A_s^{sf} \geq \begin{cases} 0.1A_{cap}^{top} \\ or \\ 0.1A_{cap}^{bot} \end{cases}$
Horizontal cap end ties	-	$A_s^{jhc} = 0.33A_s^{u-bar} \text{ min}$
Transverse column reinf. extended into slab	$A_{v,joint} \geq 0.18A_{st}$	$A_{v,joint} \geq 0.18A_{st}$



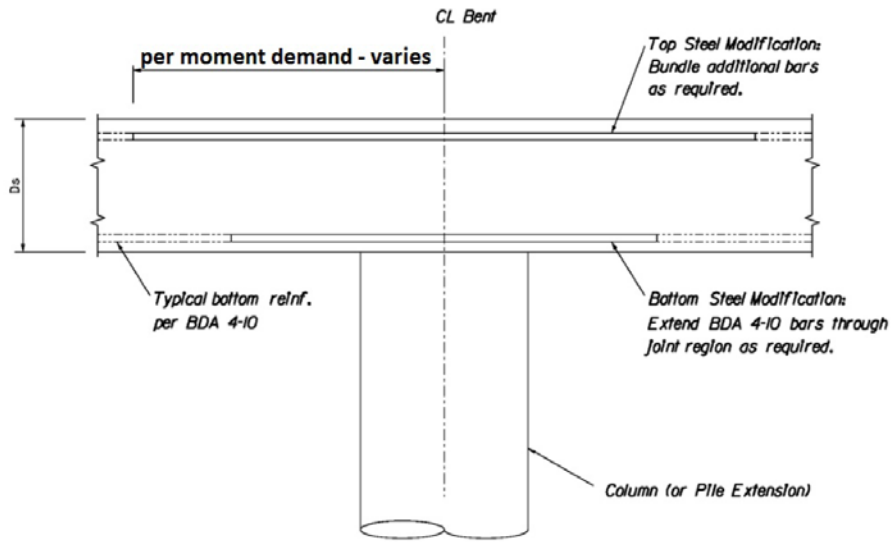
Note: Not all reinforcement shown

Figure 9.5-1 Slab Bridge Joint Shear Reinforcement



PLAN - TOP SLAB SEISMIC REINFORCEMENT

No Scale



ELEVATION

No Scale

Figure C9.5-1 Additional Slab Bridge Flexural Reinforcement

9.6 SLAB BRIDGE SUPPORTS

Pile/shaft extensions supporting slab bridges shall have a minimum diameter of 18 inches (precast pile) or 24 inches (cast-in-place pile) and shall meet the requirements of Section 6.2.5.4 – Pile and Shaft Extensions.

The minimum abutment support length for slab bridges shall be 24 inches. External or internal shear keys shall be provided for seat-type abutments.

In lieu of using an abutment seat, a pinned connection or a reduced abutment section may be used between the superstructure and the abutment.

9.7 SLAB BRIDGE HINGES

The minimum support length for in-span slab bridge hinges shall be 18 inches.

9.8 SPLICING OF REINFORCEMENT

Longitudinal slab reinforcement may be spliced with a service splice or lap splice. If a service coupler is used, the splice may be located anywhere in the span. If a lap splice is used, the splice shall be located not less than $0.2L$ from the centerline of the bent, where L is the span length. The lap splices shall be staggered.

C9.6

Precast piles are cast in forms requiring less cover and have no inspection tube requirements. Cast-in-place (CIP) shafts require more cover and a larger minimum bar spacing than precast piles to accommodate inspection tubes. The minimum sizes of both precast and CIP piles also account for the minimum hoop diameters required for reinforcing bars sizes per MTD 20-9.

Examples of acceptable pile extension and abutment support details for slab bridges may be found in Appendix A of MTD 20-7.

C9.7

Different configurations of slab bridge hinges are shown in Figure C9.7-1.

Larger longitudinal displacement capacity and greater stiffness may be achieved with Figure C9.7-1(C) where independent frames eliminate the risk of unseating altogether.

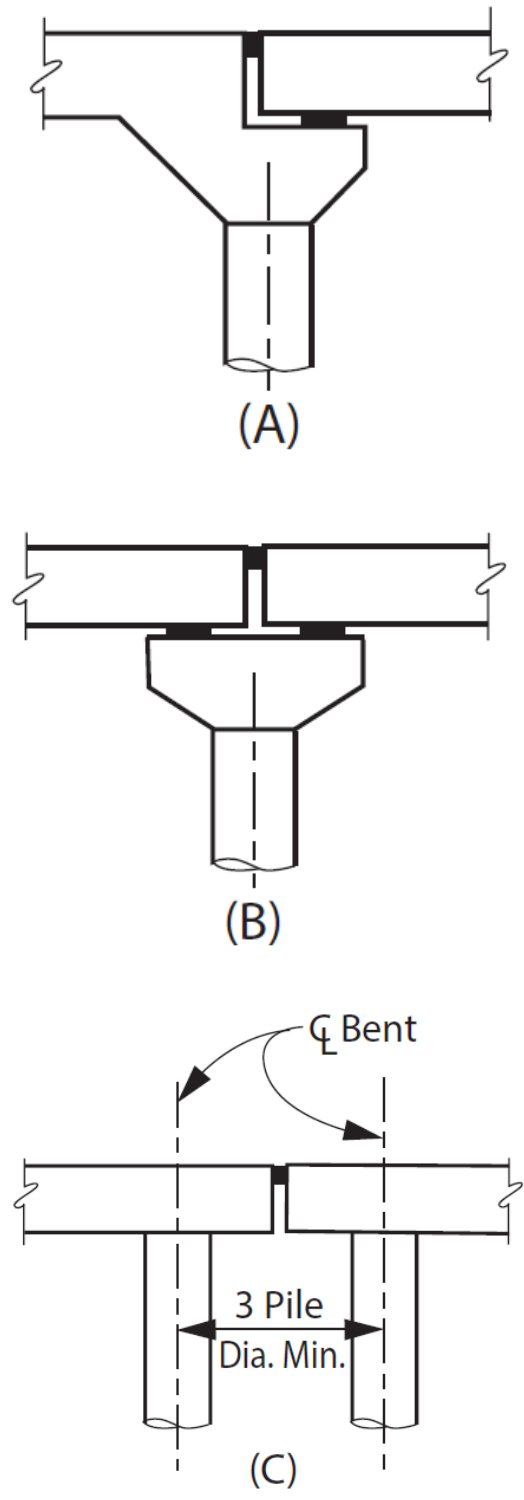


Figure C9.7-1 Slab Bridge Hinge Options



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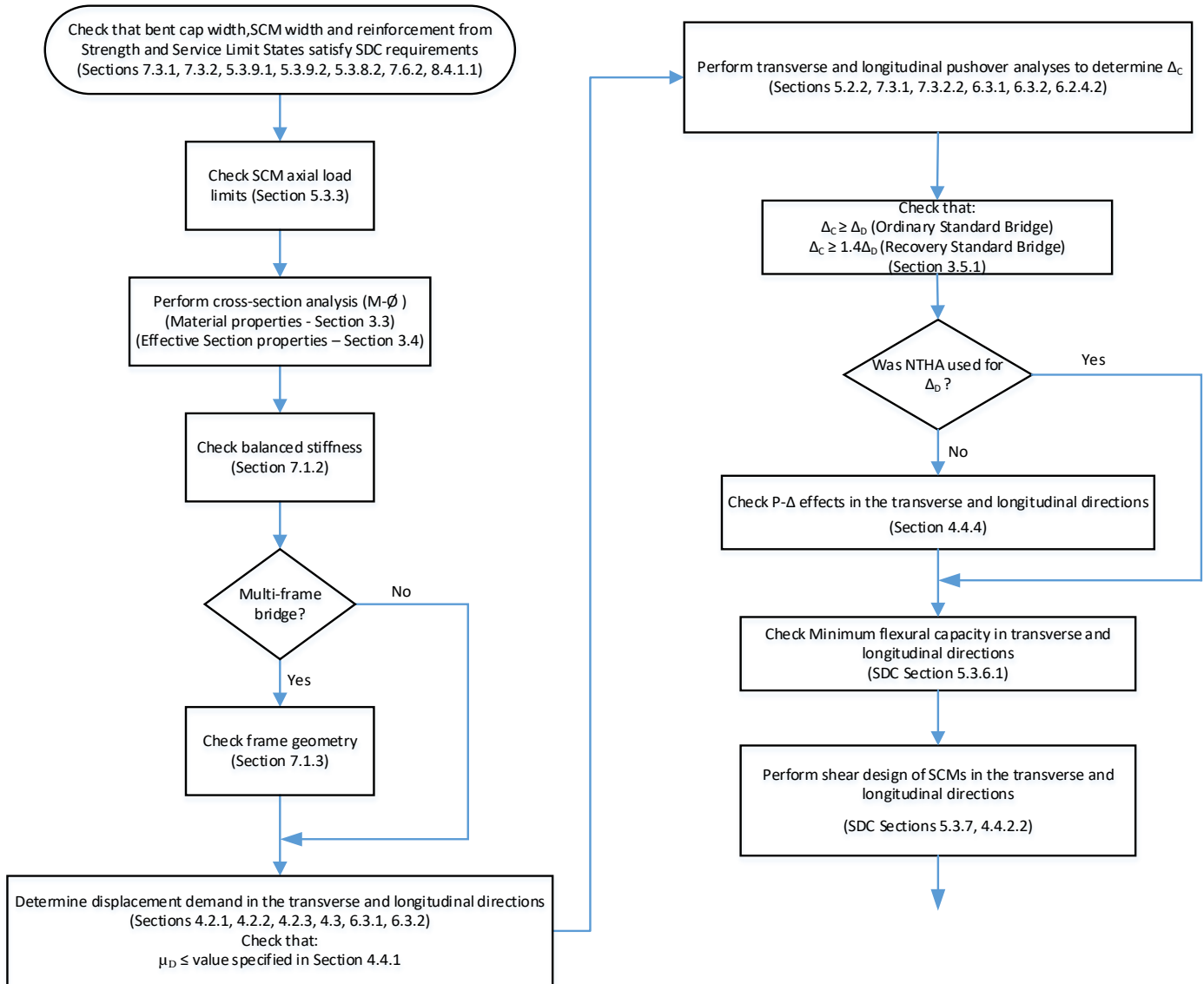
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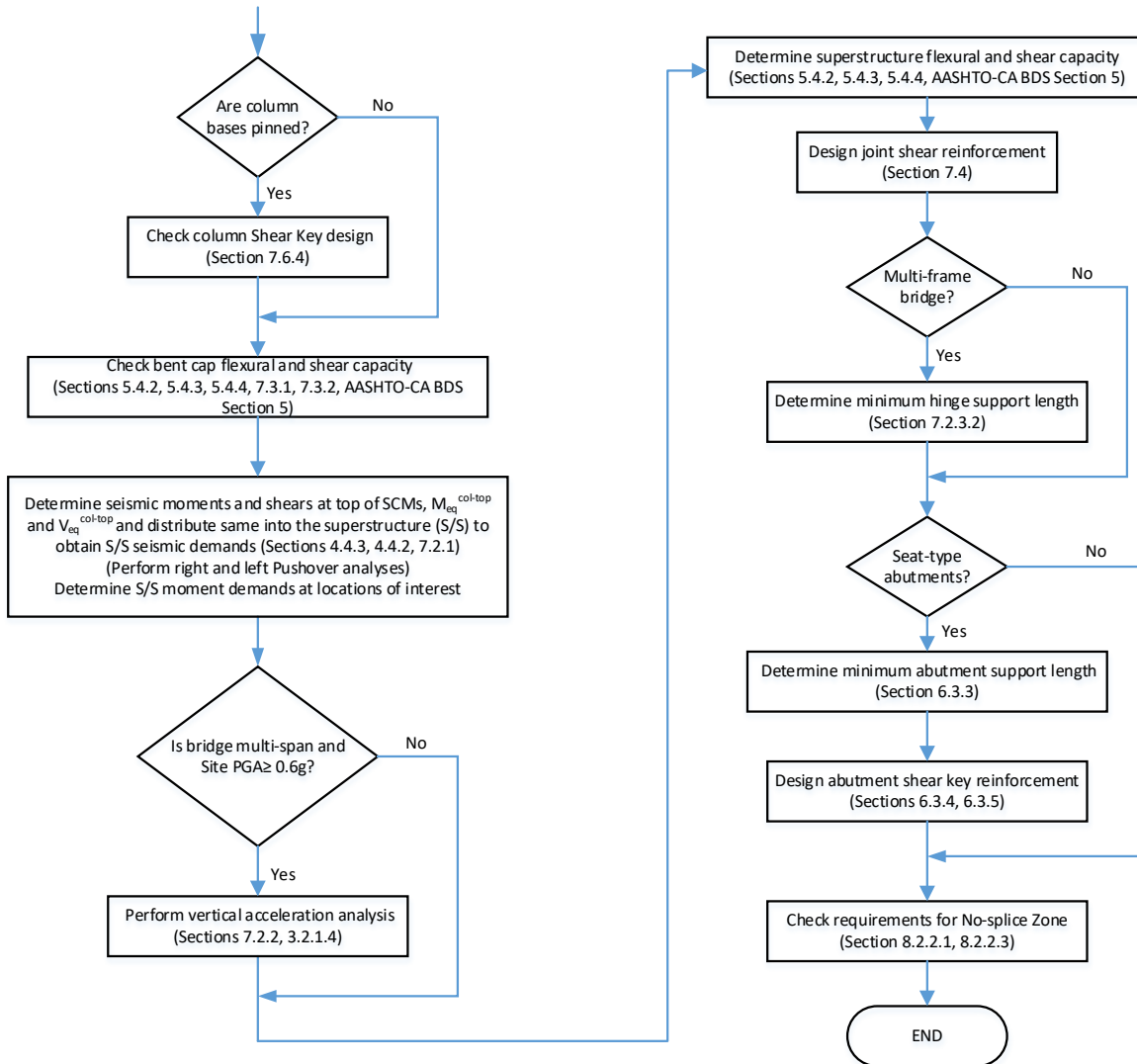


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SEISMIC DESIGN PROCEDURE FLOWCHART







DESIGN SPECTRUM DEVELOPMENT

California Seismic Hazard

Seismic hazard in California is governed by shallow crustal tectonics, with the sole exception of the Cascadia subduction zone along California's northern coastline. In both regimes, the Design Response Spectrum is based on the USGS Seismic Hazard Map (currently 2008 edition - Petersen, 2008) for the 5% in 50 years probability of exceedance (or 975-year return period). Adjustment factors for near-fault effects and basin amplification are also applied as described in the sections below.

Spectrum Adjustment Factors

The design spectrum may need to be modified to account for seismological effects related to being in close proximity to a rupturing fault and/or placement on top of a deep sedimentary basin. These adjustments are discussed in the following sections.

Near-Fault Factor

Sites located near a rupturing fault may experience elevated levels of shaking at periods longer than 0.5 second due to phenomena such as constructive wave interference, radiation pattern effects, and static fault offset (fling). As a practical matter, these phenomena are commonly combined into a single "near-fault" adjustment factor. This adjustment factor, shown in Figure B.1, is fully applied at locations with a site to rupture plane distance (R_{Rup}) of 15 km (9.4 miles) or less and linearly tapered to zero adjustment at 25 km (15.6 miles). The adjustment consists of a 20% increase in spectral values with corresponding period longer than one second. This increase is linearly tapered to zero at a period of 0.5 second.

For application to a probabilistic spectrum, a deaggregation of the site hazard at a spectral period of 1-second should be performed to determine whether the "probabilistic" distance is less than 25 km. The "probabilistic" distance shall be calculated as the smaller of the mean distance and the mode distance (from the distance-magnitude combination that has the largest contribution to the hazard), but not less than the site to rupture plane distance corresponding to the nearest fault in the Caltrans Fault Database. This latter requirement reflects the intention not to apply a near-fault adjustment factor to a background seismic source used in the probabilistic seismic hazard analysis.

Basin Factor

Both the Campbell-Bozorgnia (2008) and Chiou-Youngs (2008) ground motion prediction models include a depth to rock (Z) parameter that allows each model to better predict ground motion in regions with deep sedimentary structure. The two models use different reference velocities for rock, with Campbell-Bozorgnia using a depth to 2.5 km/s shear wave velocity ($Z_{2.5}$) and Chiou-Youngs using a depth to 1.0 km/s shear wave velocity ($Z_{1.0}$). Numerical models suggest that ground shaking in sedimentary basins is impacted by phenomena such as trapped surface waves, constructive and destructive interference, amplifications at the basin edge, and heightened 1-D soil amplification due to a greater depth of soil. Since neither the Campbell-Bozorgnia nor Chiou-Youngs models consider these phenomena explicitly, it is more accurate to refer to predicted amplification due to the Z parameter as a "depth to rock" effect instead of a basin effect. However, since sites with large depth to rock are located in basin structures the term "basin effect" is commonly used.

Amplification factors for the two models are shown for various depths to rock in Figure B.2. These plots assume a shear wave velocity for the upper 30 m of the soil profile, v_{s30} of 270 m/s (typical for many basin locations) but are suitable for other v_{s30} values as well since the basin effect is only slightly sensitive to v_{s30} (primarily at periods less than 0.5 second). It should be noted that both models predict a decrease in long period energy for cases of shallow rock ($Z_{2.5} < 1$ km or $Z_{1.0} < 40$ m). Since $Z_{2.5}$ and $Z_{1.0}$ data are generally unavailable at non-basin locations, implementation of the basin amplification factors is restricted to locations with $Z_{2.5}$ larger than 3 km or $Z_{1.0}$ larger than 400 m.

Maps of $Z_{1.0}$ and $Z_{2.5}$

Figures B.3 through B.9 show contour maps of $Z_{1.0}$ and $Z_{2.5}$ for regions with sufficient depth to rock to trigger basin amplification. In Southern California, these maps were generated using data from the Community Velocity Model (CVM) Version 4 (http://sceec.usc.edu/sceecpedia/Community_Velocity_Model). In Northern California, the $Z_{2.5}$ contour map was generated using tomography data by Thurber (2009) and a generalized velocity profile by Brocher (2005). A $Z_{1.0}$ contour map could not be created in Northern California due to insufficient data.

Application of the models

For Southern California locations, an average of the Campbell-Bozorgnia and Chiou-Youngs basin amplification factors is applied. For Northern California locations, only the Campbell-Bozorgnia basin amplification factor is applied.

Directional Orientation of Design Spectrum

When recorded horizontal components of earthquake ground motion are mathematically rotated to different orientations, the corresponding response spectrum changes as well. The probabilistic spectra defined above reflect a spectrum that is equally probable in all orientations. The maximum response spectrum, occurring at a specific but unpredictable orientation, is approximately 15% to 25% larger than the equally probable spectrum calculated using the procedures described above. Since a narrow range of directional orientations typically define the critical loading direction for bridge structures, the equally probable component spectrum is used for design.

Selection of v_{s30} for Site Amplification

The Campbell-Bozorgnia (2008), Chiou-Youngs (2008), and Boore-Atkinson (2008) ground motion prediction models (the latter is included for application to the probabilistic spectrum) use the parameter v_{s30} to characterize near surface soil stiffness as well as infer broader site characteristics. v_{s30} represents the average small strain shear wave velocity in the upper 100 feet (30 meters) of the soil column. This parameter, along with the level of ground shaking, determines the estimated site amplification in each of the above models. If the shear wave velocity (v_s) is known (or estimated) for discrete soil layers, then v_{s30} can be calculated as follows:

$$v_{s30} = \frac{100 \text{ ft}}{\frac{D_1}{v_1} + \frac{D_2}{v_2} + \dots + \frac{D_n}{v_n}}$$

where, D_n represents the thickness of layer n (ft), v_n represents the shear wave velocity of layer n (fps), and the sum of the layer depths equals 100 feet. It is recommended that direct shear wave velocity measurements be used, or, in the absence of available field measurements, correlations to available parameters such as undrained shear strength, cone penetration tip resistance, or standard penetration test blow counts be utilized. Additional recommendations pertaining to determination of v_{s30} for development of the preliminary and final design spectrum are given in "Methodology for Developing Design Response Spectra" available in the Technical References link of the ARS Online V2 website (http://dap3.dot.ca.gov/shake_stable/v2/).

Figure B.10 provides a profile classification system that is published in Applied Technology Council–32 (1996) and was adopted in previous versions of SDC. This table includes general guidance on average shear wave velocity that may be useful for development of a preliminary design spectrum. The Campbell-Bozorgnia and Chiou-Youngs ground motion prediction equations are applicable for v_{s30} ranging from 150 m/s (500 fps) to 1500 m/s (5000 fps). For cases where v_{s30} exceeds 1500 m/s (very rare in California), a value of 1500 m/s should be used. For cases where either (1) v_{s30} is less than 150 m/s, (2) one or more layers of at least 5 feet thickness has a shear wave velocity less than 120 m/s, or (3) the profile conforms to Soil Profile Type E criteria per Figure B.10, a site-specific response analysis is required for determination of the final design spectrum.

For cases where the site meets the criteria prescribed for Soil Profile Type E, the response spectra presented in Figures B.11 - B.13, originally presented in ATC-32, can be used for development of a preliminary design spectrum. In most cases, however, Type E spectra will significantly exceed spectra developed using site response analysis methods. For this reason, it is preferred that a site response analysis be performed for the determination of the preliminary design spectrum in Type E soils.

When a soil profile meets the criteria prescribed for Soil Profile Type F (in Figure B.10), a site response analysis is required for both preliminary and final design.

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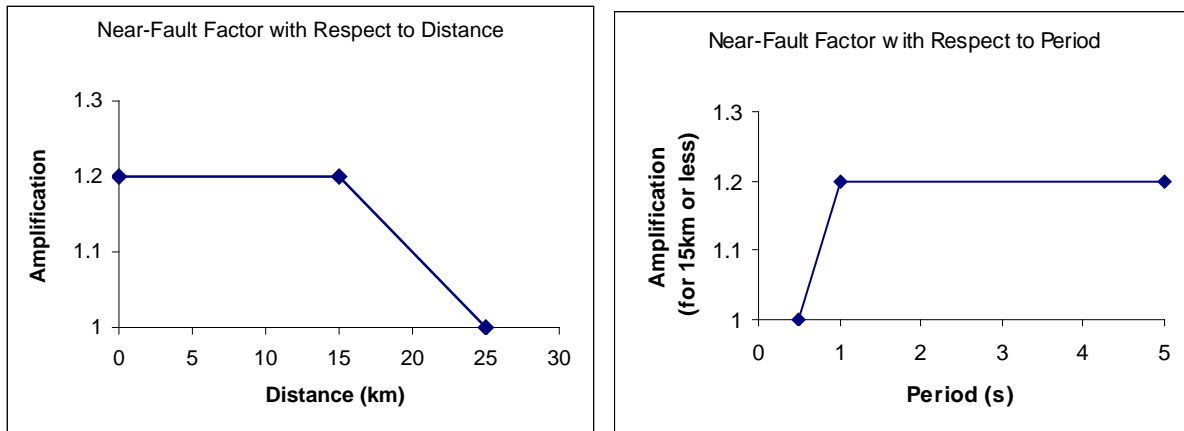


Figure B.1 Near-Fault adjustment factor as a function of distance and spectral period. The distance measure is based on the closest distance to any point on the fault plane

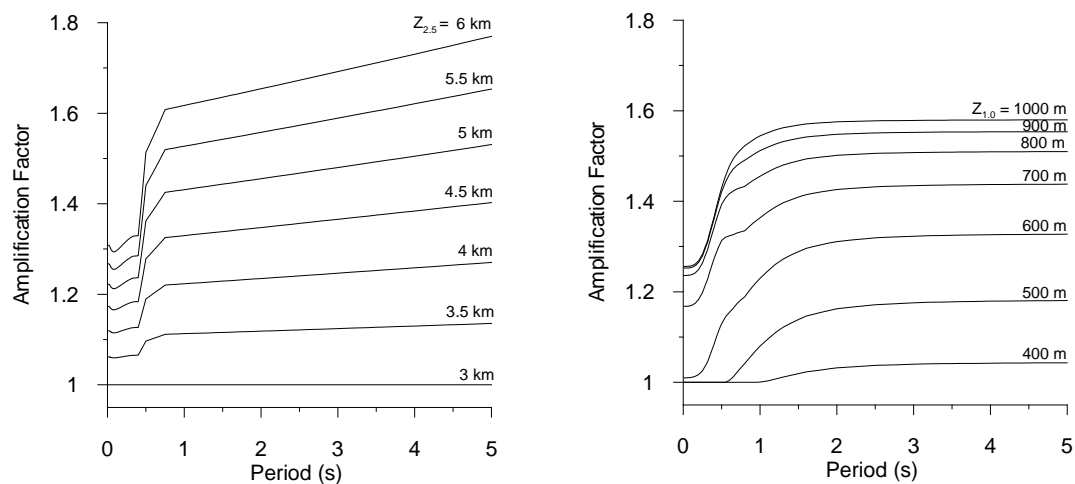


Figure B.2 Basin amplification factors for the Campbell-Bozorgnia (2008) and Chiou-Youngs (2008) ground motion prediction equations. Curves may be slightly conservative at periods less than 0.5 seconds

Los Angeles Basin $Z_{1.0}$

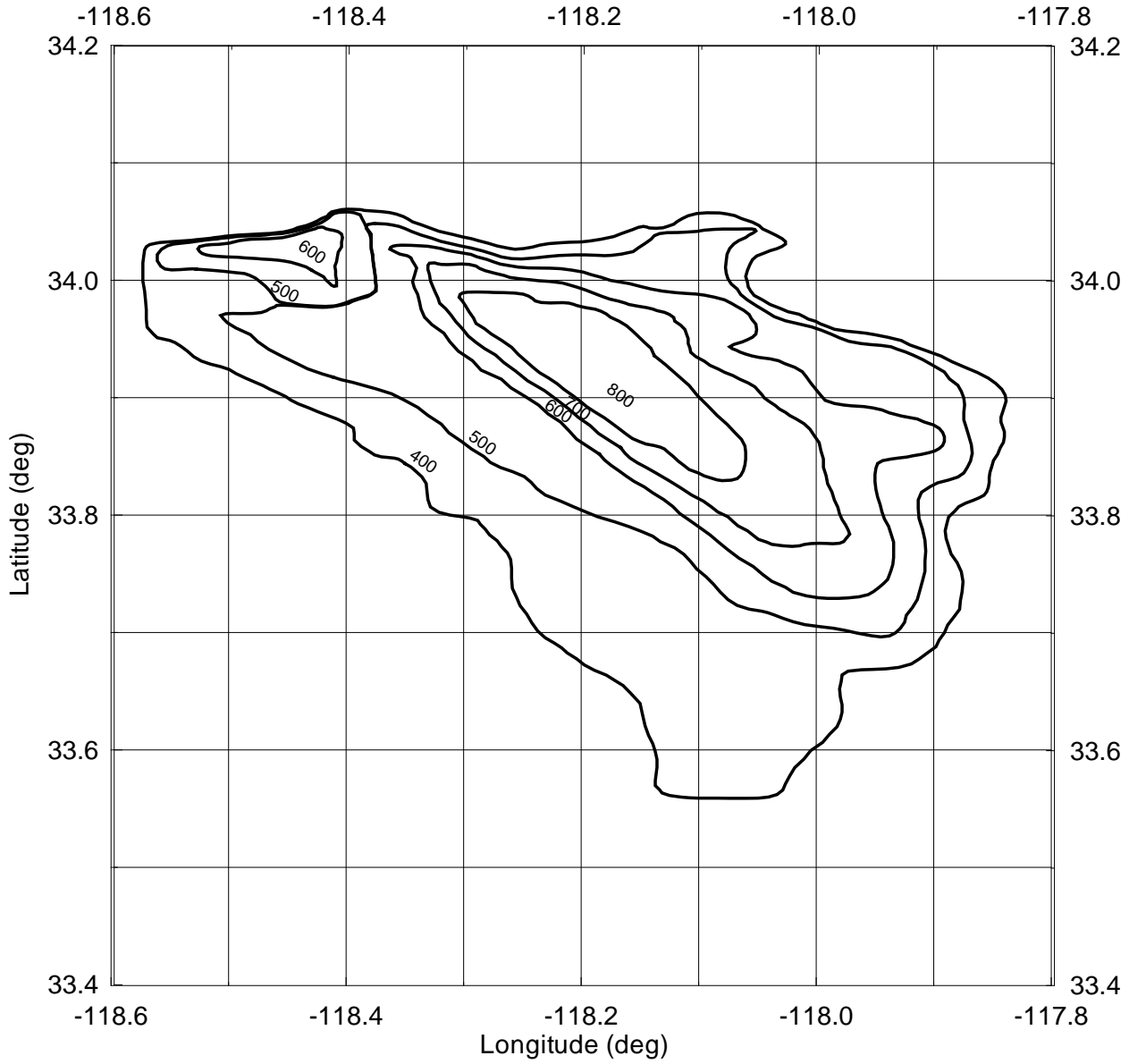


Figure B.3 Contours of depth (meters) to shear wave velocity 1 km/s ($Z_{1.0}$) in the Los Angeles Basin

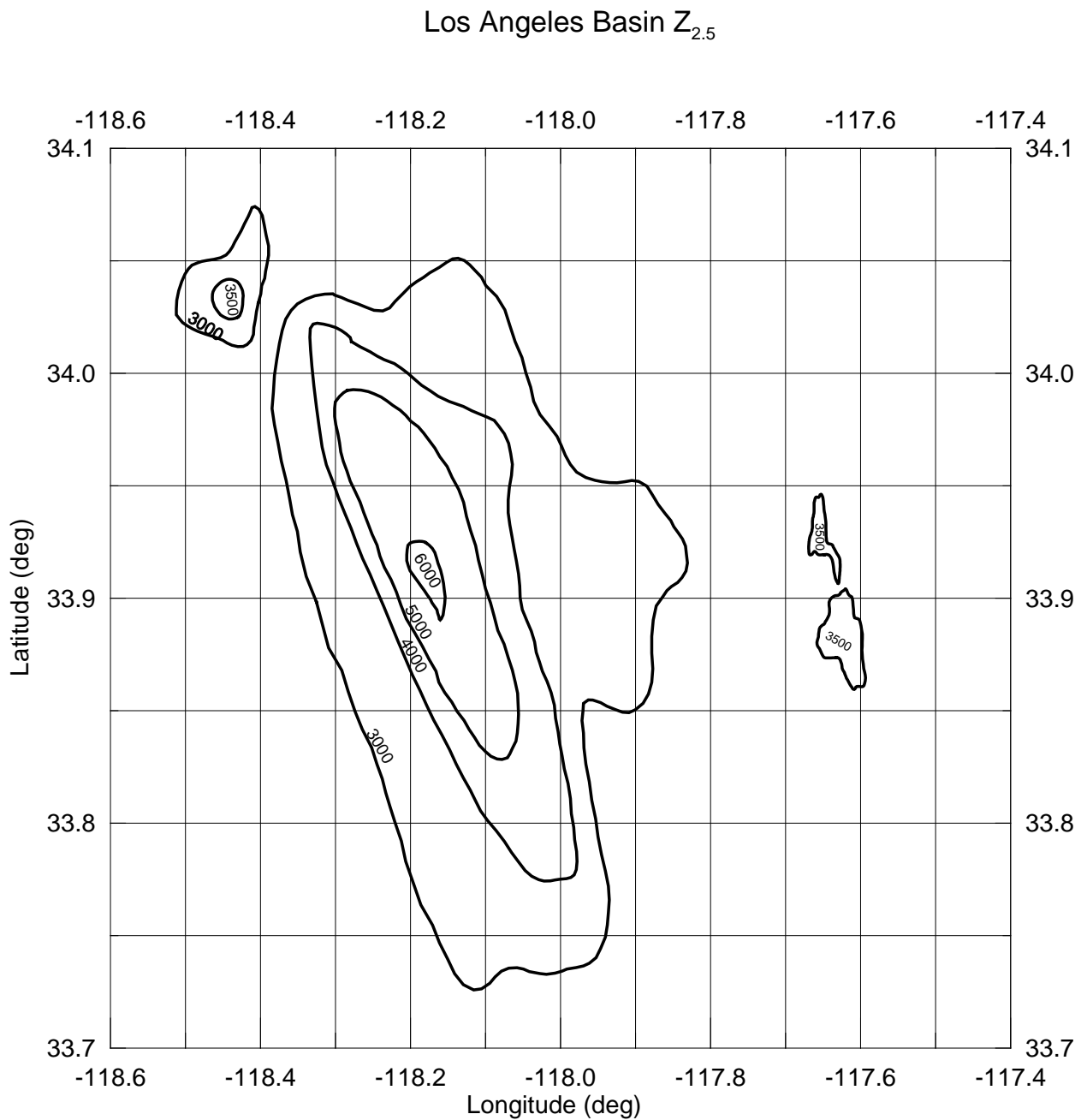


Figure B.4 Contours of depth (meters) to shear wave velocity 2.5 km/s ($Z_{2.5}$) in the Los Angeles Basin

Ventura Basin $Z_{1.0}$

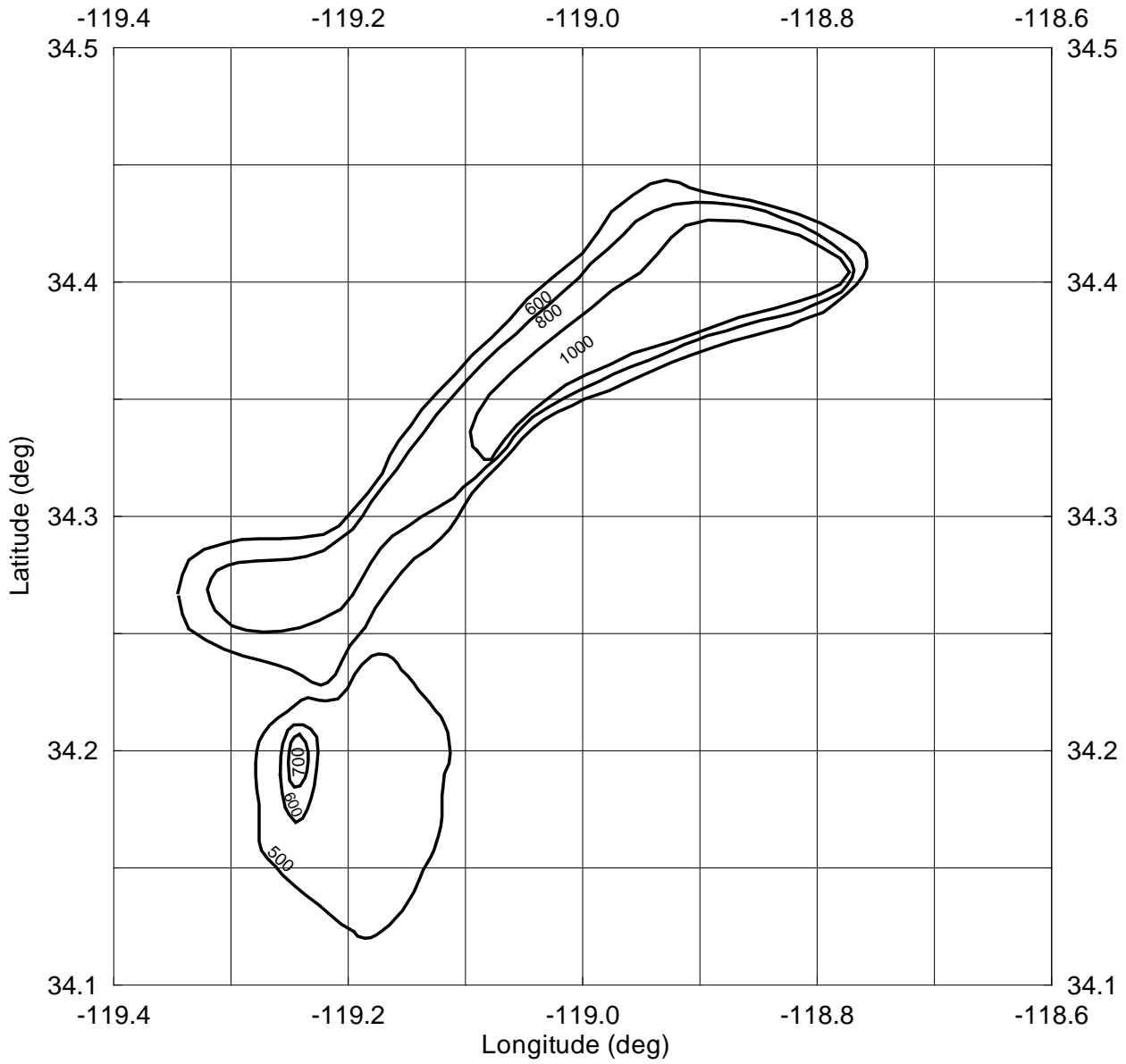


Figure B.5 Contours of depth (meters) to shear wave velocity 1 km/s ($Z_{1.0}$) in the Ventura Basin

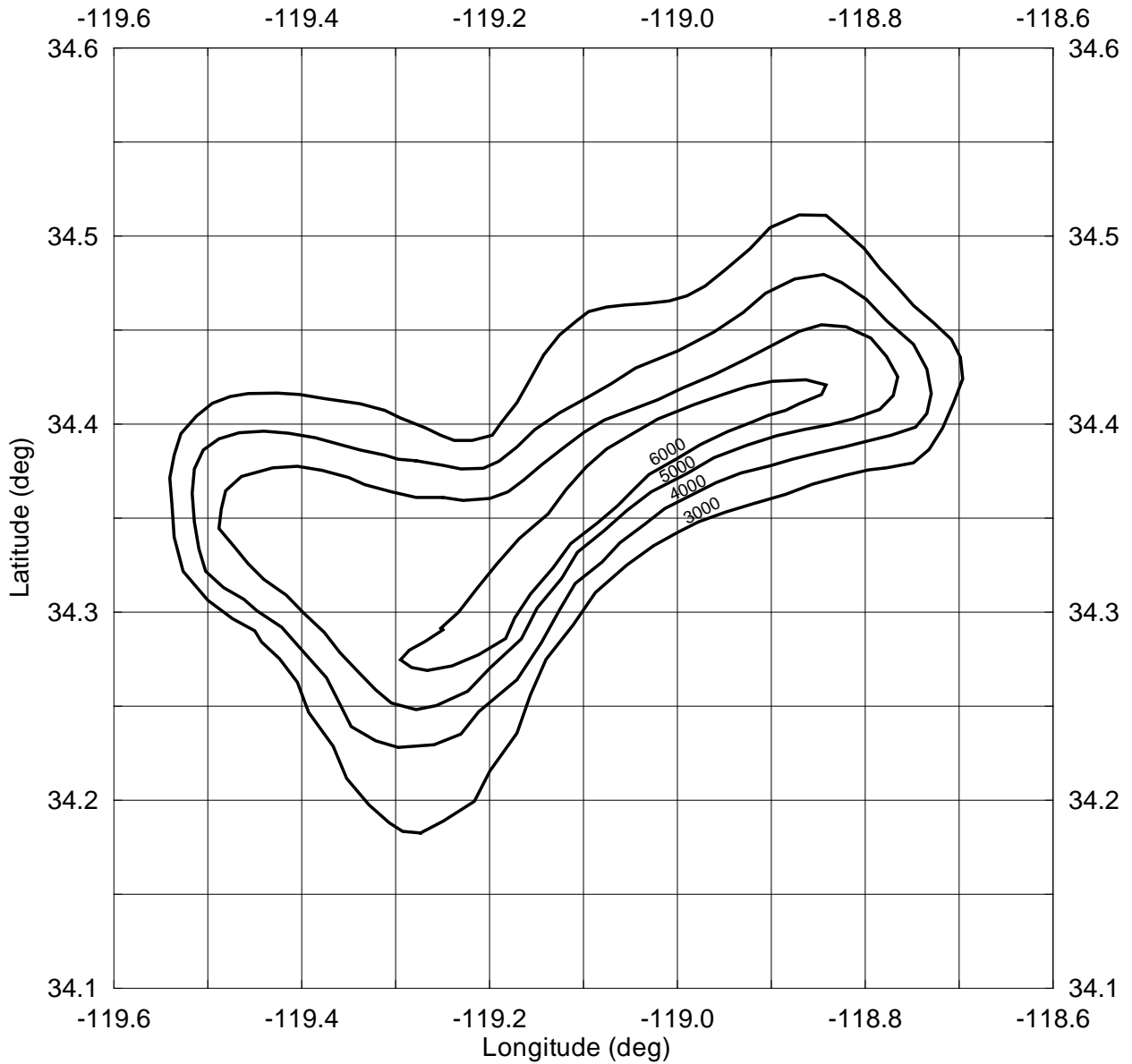
Ventura Basin $Z_{2.5}$ 

Figure B.6 Contours of depth (meters) to shear wave velocity 2.5 km/s ($Z_{2.5}$) in the Ventura Basin

Salton Basin $Z_{1.0}$

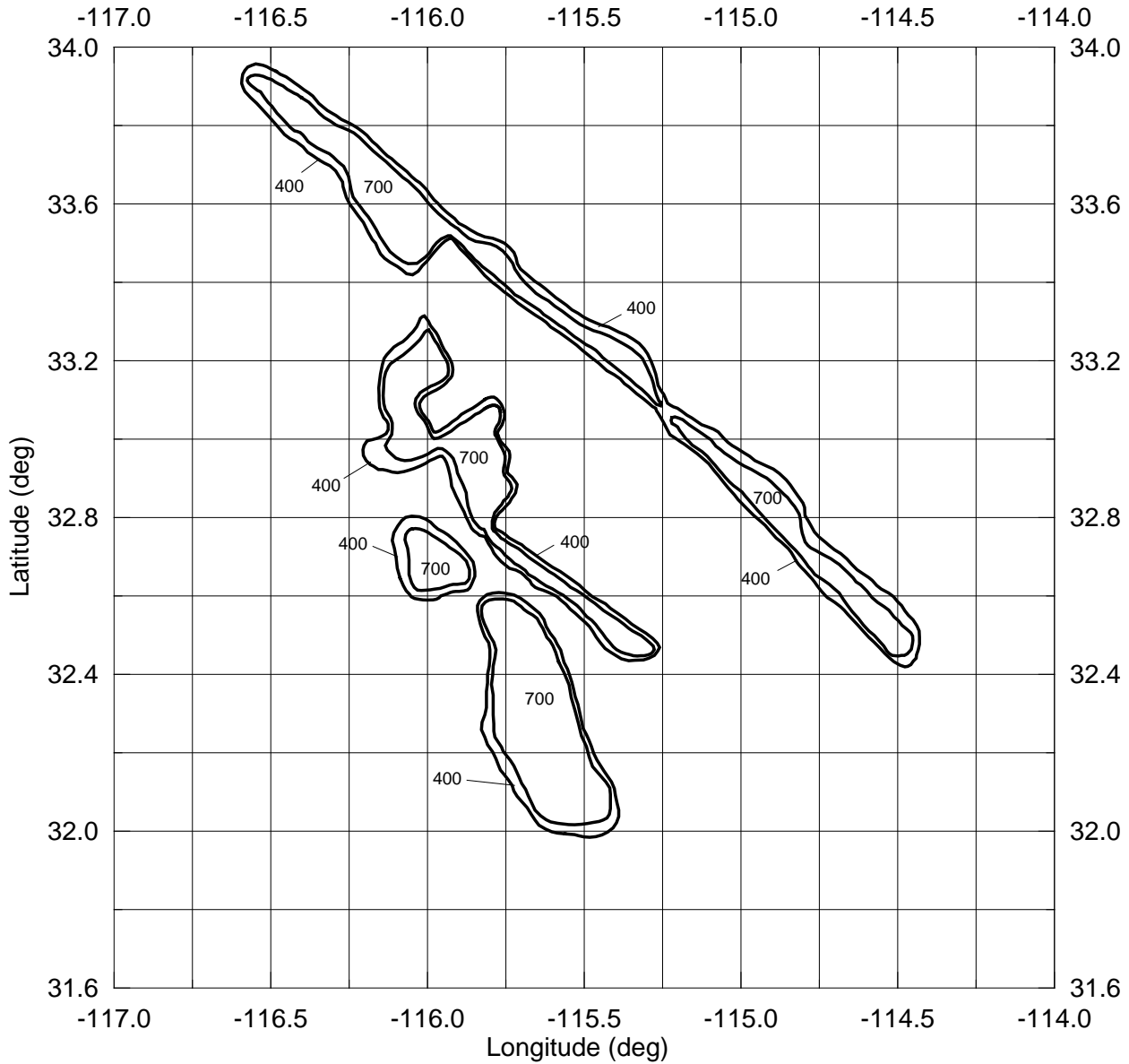


Figure B.7 Contours of depth (meters) to shear wave velocity 1 km/s ($Z_{1.0}$) in the Salton Basin (Imperial Valley)

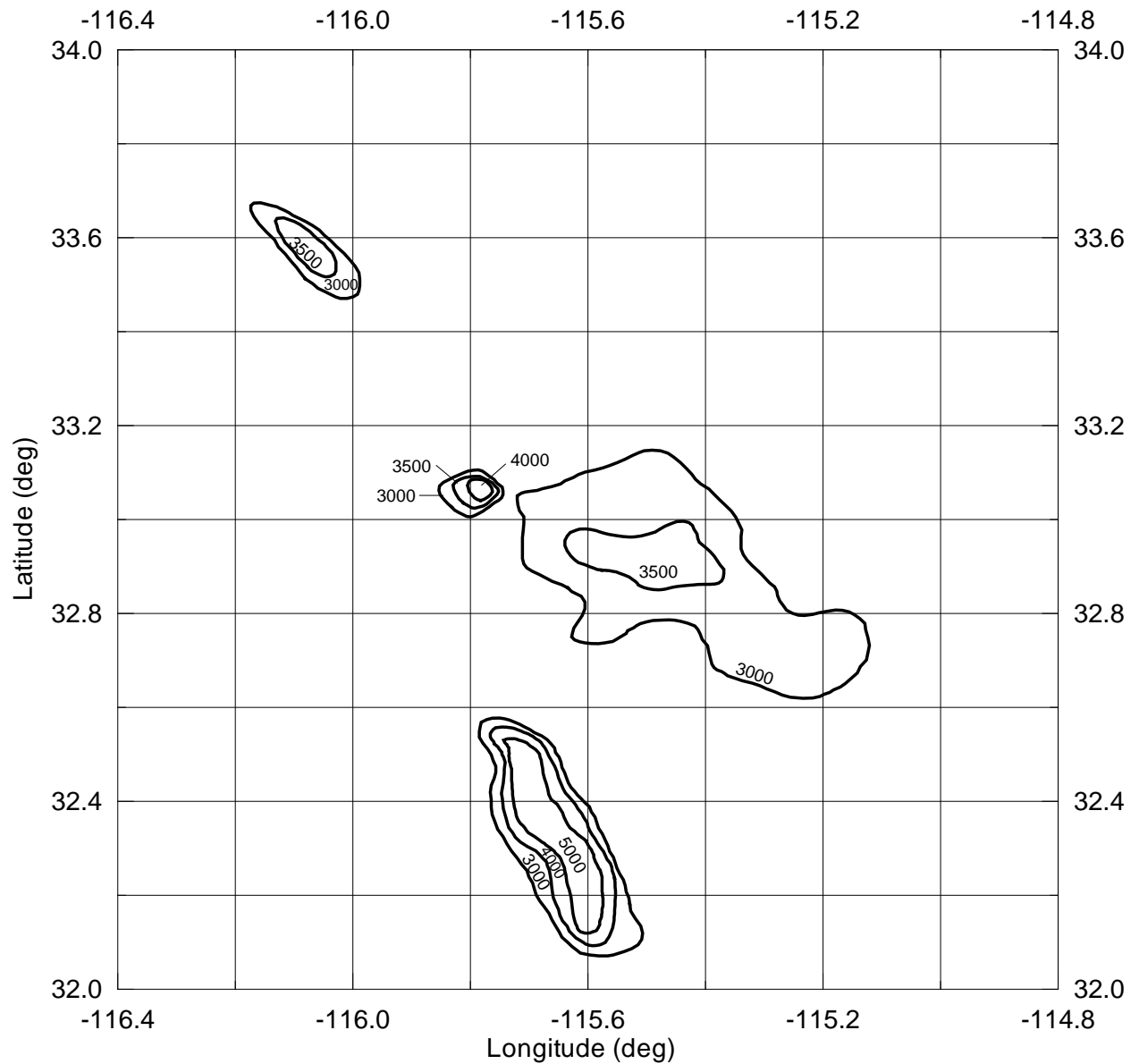
Salton Basin $Z_{2.5}$ 

Figure B.8 Contours of depth (meters) to shear wave velocity 2.5 km/s ($Z_{2.5}$) in the Salton Basin (Imperial Valley)

Northern California $Z_{2.5}$

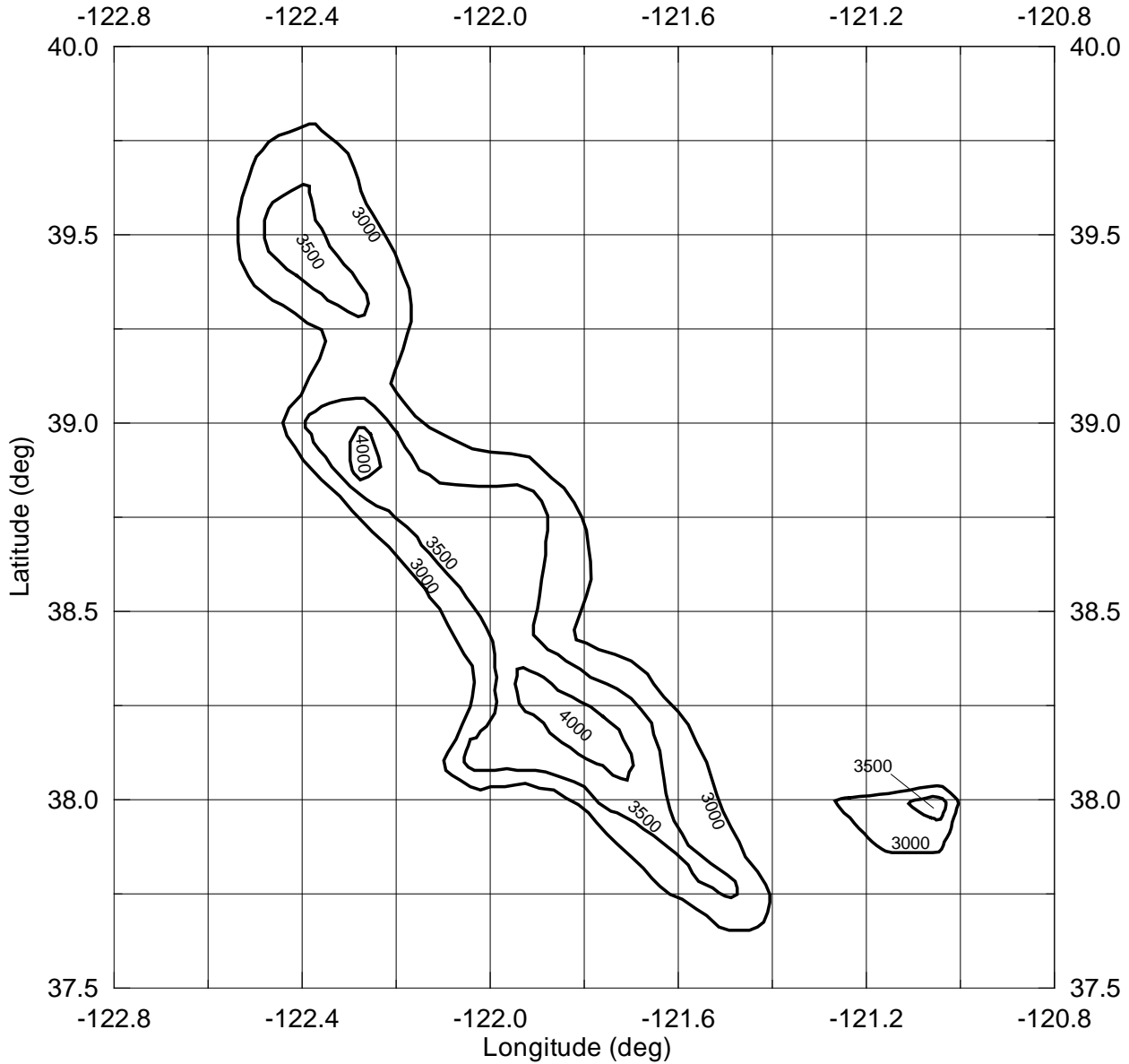


Figure B.9 Contours of depth (meters) to shear wave velocity 2.5 km/s ($Z_{2.5}$) in Northern California

Soil Profile Type	Soil Profile Description ^a
A	Hard rock with measured shear wave velocity $v_{S30} > 5000$ ft/s (1,500 m/s)
B	Rock with shear wave velocity $2,500 < v_{S30} < 5000$ ft/s ($760\text{m/s} < v_{S30} < 1,500$ m/s)
C	Very dense soil and soft rock with shear wave velocity $1,200 < v_{S30} < 2,500$ ft/s ($360\text{m/s} < v_{S30} < 760$ m/s) or with either standard penetration resistance $N > 50$ or undrained shear strength $s_u \geq 2,000$ psf (100 kPa)
D	Stiff soil with shear wave velocity $600 < v_{S30} < 1,200$ ft/s ($180 \text{ m/s} < v_{S30} < 360$ m/s) or with either standard penetration resistance $15 \leq N \leq 50$ or undrained shear strength $1,000 < s_u < 2,000$ psf ($50 < s_u < 100$ kPa)
E	A soil profile with shear wave velocity $v_{S30} < 600$ ft/s (180 m/s) or any profile with more than 10 ft (3 m) of soft clay, defined as soil with plasticity index $PI > 20$, water content $w \geq 40$ percent, and undrained shear strength $s_u < 500$ psf (25 kPa)
F	<p>Soil requiring site-specific evaluation:</p> <ol style="list-style-type: none"> 1. Soils vulnerable to potential failure or collapse under seismic loading; i.e. liquefiable soils, quick and highly sensitive clays, collapsible weakly-cemented soils 2. Peat and/or highly organic clay layers more than 10 ft (3 m) thick 3. Very high-plasticity clay ($PI > 75$) layers more than 25 ft (8 m) thick 4. Soft-to-medium clay layers more than 120 ft (36 m) thick

^a The soil profile types shall be established through properly substantiated geotechnical data.

Figure B.10 Soil profile types (after Applied Technology Council-32-1, 1996)

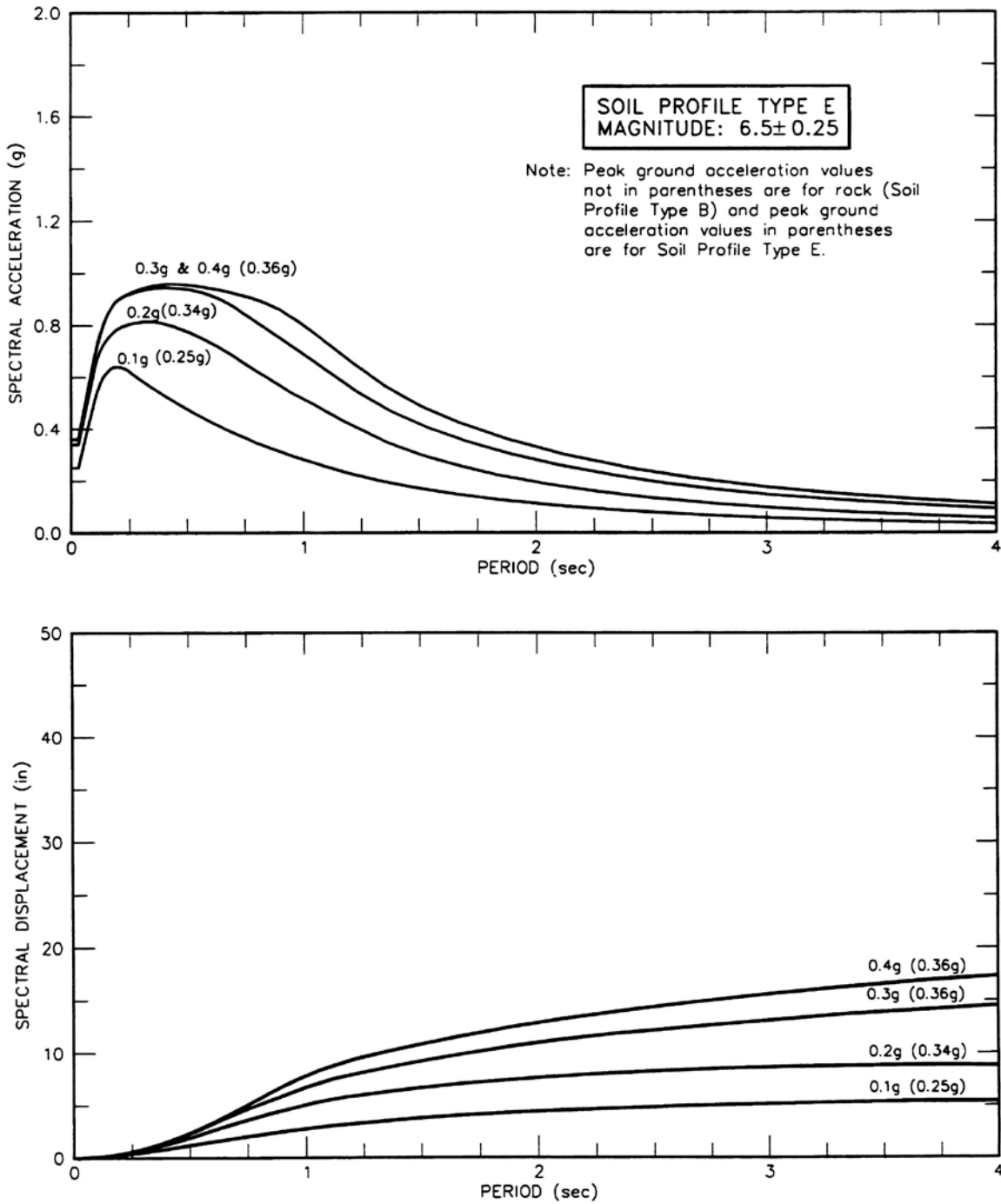


Figure B.11 Spectral Acceleration and Displacement for Soil Profile E ($M = 6.5 \pm 0.25$)

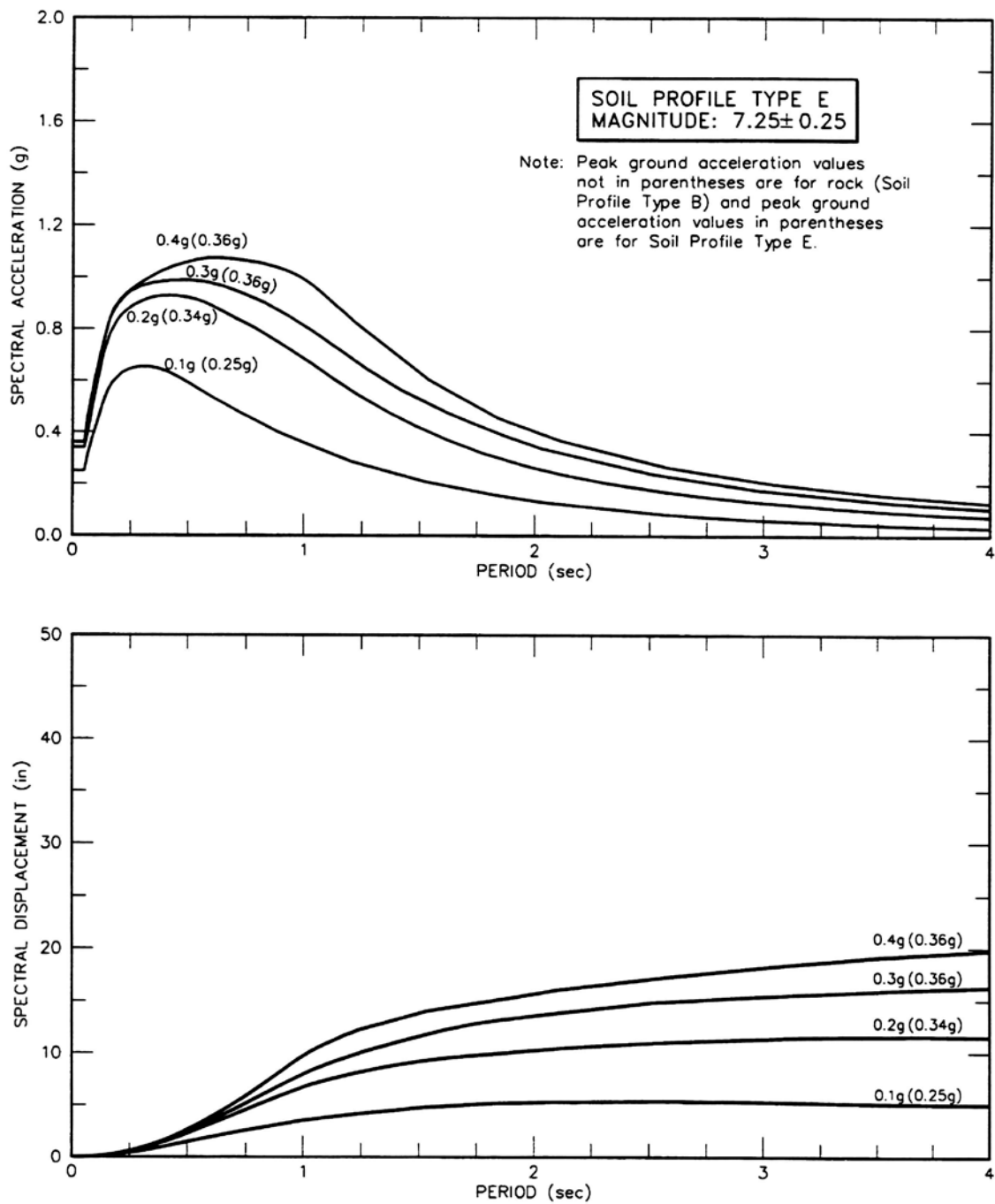


Figure B.12 Spectral Acceleration and Displacement for Soil Profile E ($M = 7.25 \pm 0.25$)

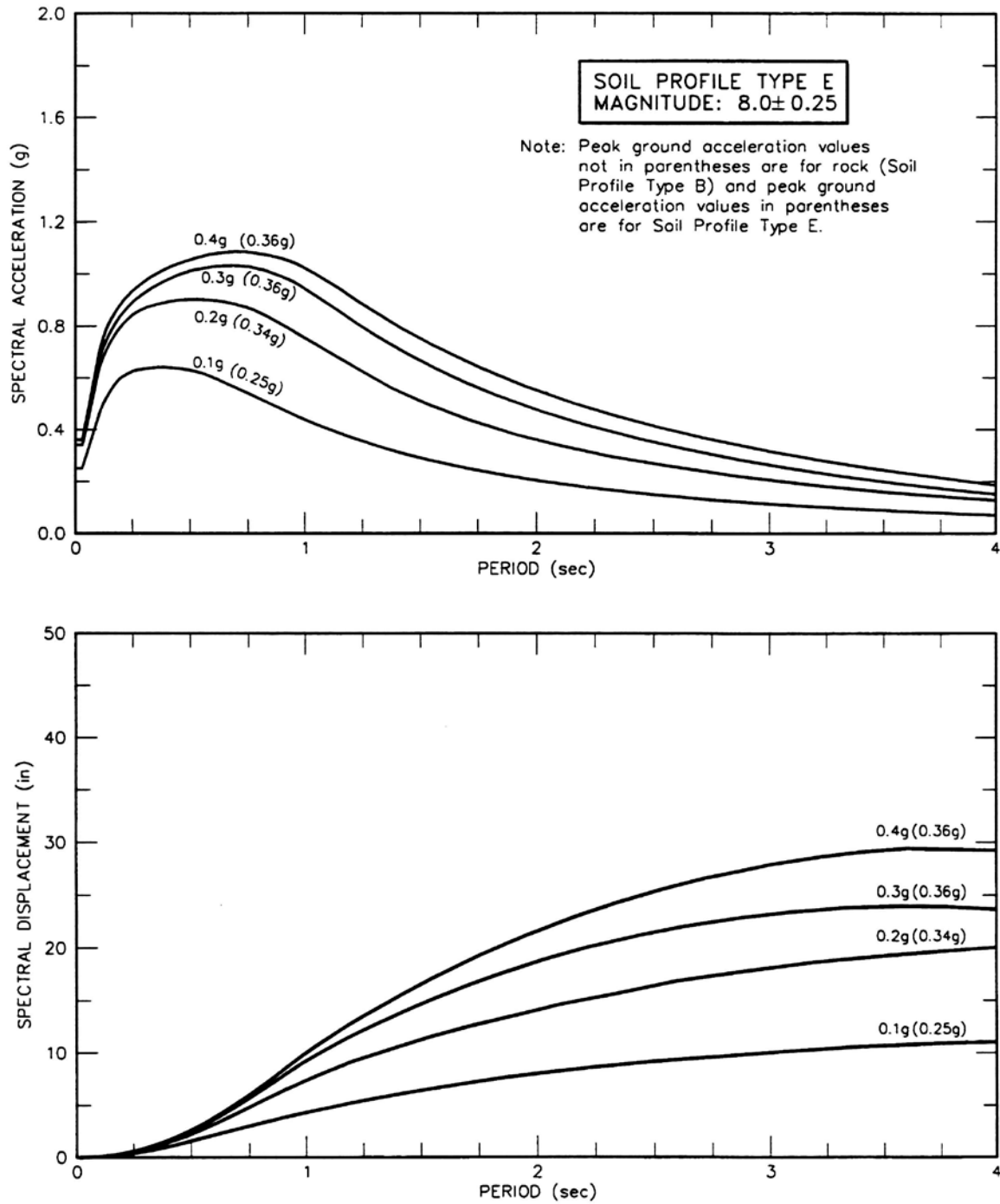


Figure B.13 Spectral Acceleration and Displacement for Soil Profile E ($M = 8.0 \pm 0.25$)



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