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Report on the Seismic Design of Bridge Columns Based on Drift

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Report on the Seismic Design of Bridge Columns Based on Drift

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This report provides a basis for evaluating bridge column drift demands and bridge column performance under simulated earthquake loading. It is intended for practicing engineers and academic researchers. Seismic performance objectives established for bridges are reviewed with an emphasis on bridge column performance states. Examples of column damage in past earthquakes are reviewed. Results from recent research on column performance are adapted to the case of bridge columns having a practical range of transverse reinforcement. These results are summarized in terms of drift limits associated with different performance states as a function of column shear span-depth ratio and axial load ratio,

for both rectangular and circular section columns. A static push-over method is presented that accounts for embankment flexibility. A two-span bridge is used as an example to illustrate the evaluation of column performance, the influence of changing column bent configurations (two 5 ft [1500 mm] diameter columns versus three 4 ft [1200 mm] diameter columns), and that larger column drift demands may result when embankment mass and flexibility are modeled.

Keywords: abutment; bridge; column; drift limit, embankment flexibility; performance objective, seismic analysis; seismic evaluation; seismic performance.

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CONTENTS

CHAPTER 1—INTRODUCTION AND SCOPE, p. 2

1.1—Introduction, p. 2

1.2—Scope, p. 2

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CHAPTER 2—NOTATION, p. 3**CHAPTER 3—DESIGN OBJECTIVES AND APPROACHES, p. 5**

- 3.1—Performance-based design philosophy, p. 5
- 3.2—Ductile mechanisms, p. 6
- 3.3—Performance states and objectives, p. 6
- 3.4—Earthquake ground motion, p. 9
- 3.5—Methods of analysis, p. 10
- 3.6—Design methods, p. 11

CHAPTER 4—PERFORMANCE LIMITS FOR REINFORCED CONCRETE BRIDGE COLUMNS, p. 14

- 4.1—Introduction, p. 14
- 4.2—Field observations of bridge column performance, p. 14
- 4.3—Laboratory observations of bridge column performance, p. 16
- 4.4—Performance expectations, p. 23

CHAPTER 5—CONSIDERATION OF EMBANKMENT FLEXIBILITY IN EVALUATION OF COLUMN DRIFT DEMANDS, p. 25

- 5.1—Introduction, p. 25
- 5.2—Structural modeling, p. 26
- 5.3—Analysis procedures, p. 27
- 5.4—Column design strengths, p. 31
- 5.5—Column displacements, p. 32
- 5.6—Evaluation of column displacement performance states, p. 32

CHAPTER 6—DESIGN OF COLUMNS FOR PERFORMANCE AND CONSTRUCTABILITY, p. 32

- 6.1—Introduction, p. 32
- 6.3—Design of transverse reinforcement, p. 35
- 6.4—Anchorage and splices of longitudinal reinforcement, p. 37
- 6.5—Splices of hoop and spiral reinforcement, p. 41
- 6.6— P - Δ effects, p. 41

CHAPTER 7—EXAMPLES, p. 41

- 7.1—Introduction, p. 41
- 7.2—Example: evaluation of performance of rectangular section column, p. 41
- 7.3—Evaluation of column performance in two-span bridge, p. 42
- 7.4—Conclusions, p. 59

CHAPTER 8—REFERENCES, p. 60

Authored documents, p. 60

APPENDIX A—NORMALIZED EMBANKMENT CAPACITY CURVES, p. 64**CHAPTER 1—INTRODUCTION AND SCOPE****1.1—Introduction**

Performance-based seismic design for bridges has come to the forefront after bridges subject to strong shaking in the 1989 Loma Prieta, 1994 Northridge, 1995 Hyogo-ken Nambu, and 1999 Marmara earthquakes were significantly damaged and collapsed. This damage, while not surprising, underscores the need to enhance design approaches to consider the damage to and functionality of bridges in the smaller, more frequent events. Key concepts of performance-based design were set forth for buildings in the Vision 2000 document of the Structural Engineers Association of California (SEAOC 1995) and were subsequently articulated for bridges in an Applied Technology Council report (ATC-32 1996) and National Cooperative Highway Research Program (NCHRP) Project 12-49 (NCHRP 2003). Bridges are designed to develop inelastic mechanisms distinct from those intended in modern buildings, often involving yielding of substructure columns. This report, therefore, addresses the design and evaluation of bridge columns for seismic performance. Material relevant to both design and analysis is included.

1.2—Scope

Current design practice, as reflected in Caltrans (2013) and AASHTO (2013), makes use of force-based design approaches. These approaches, which reduce elastic design forces by a factor to account for the intended ductile response of critical bridge components, have been used for many years. More recently, displacement-based design approaches, such as outlined by AASHTO (2011), have been advocated for performance-based seismic design. While promising, displacement-based design approaches do not have the support of decades of validation in the field. Uncertainty exists in estimates of demands and capacities, and at present it is difficult to implement a comprehensive treatment of uncertainty in routine design practice. Therefore, a deterministic approach for displacement-based seismic design is described herein. This approach is intended to more reliably achieve intended performance objectives than can be achieved with other approaches, and augments existing tools available to designers. The approach is developed in terms of performance objectives and associated column drift levels. Because embankment flexibility can have a significant effect on drift demands in the columns of ordinary bridges having one or several spans, a method to consider this effect is presented. The sensitivity of computed response to design and modeling assumptions is illustrated by example.

Column deformation capacity at any performance limit is dependent on the amount of longitudinal and transverse reinforcement, material properties, geometry and boundary conditions, and loading history. Experimental tests indicate substantial variability in the deformation capacity

ties associated with discrete performance limits (damage states). Combined loading—for example, bending moment combined with axial force and torsion—further influences drift capacity (Prakash et al. 2010).

Typical design approaches have relied on point estimates to compare capacity and demand. They are referred to as deterministic design approaches. Point estimates are single value estimates of values that have a statistical distribution. Recognizing the significant uncertainty in both demands and capacities, alternative approaches would establish an adequate level of confidence that demands do not exceed capacities at a specified hazard level. They might also seek to provide an acceptably small mean annual frequency of demands exceeding capacities. However, many challenges remain in adequately defining seismic hazard, site conditions, structural properties, and component hysteretic behavior, including component deformation capacities, to fulfill the theoretical potential of performance-based design. Furthermore, addressing these uncertainties in the context of realistic limitations in design practice presents a formidable challenge. This document considers point estimates of demands and capacities. Performance limits well short of collapse are considered, thereby providing a reserve margin.

Drift is the index used to compare capacity and demand as it is a direct measure of bridge performance, unambiguous, and easily identified. Performance states are established as a function of limiting drift demands for a range of transverse steel content relevant to practice. Only rectangular and circular solid, not hollow, reinforced concrete (RC) column sections are considered. Transverse reinforcement content can be varied within limits to affect drift capacity, thereby allowing the design approach to be used over regions of varied seismic hazard. Relatively little experimental data are available on the performance of columns made with high-strength concrete. One example is compressive strengths greater than 8000 psi (55 MPa). The drift capacity estimates made herein, therefore, are for concrete strengths less than 8000 psi (55 MPa), a strength range commonly used by most State Departments of Transportation.

Methods for evaluating drift demands are described, with emphasis on consideration of embankment response, which can be significant for common short-span bridges. Where conventional force-based design approaches are used, the drifts have a secondary role and generally need not be known with great accuracy. The emphasis herein on performance resulting from imposed drift demands places greater importance on the accuracy of drift estimates. Because computed drift demands are highly sensitive to analysis methods and modeling assumptions, as may be seen in the examples of Chapter 7, care should be taken in establishing expected demands and in interpreting the adequacy of a design to meet the intended performance objective.

Chapter 3 addresses performance objectives. Chapter 4 examines the performance of columns and establishes drifts associated with significant performance limits. Chapter 5 addresses the evaluation of drift demands and provides detailed information for treating embankment flexibility using a simplified pushover method of analysis. Chapter 6

summarizes requirements for proportioning and detailing column reinforcement. Chapter 7 illustrates the application of the drift performance chart and analyses used to evaluate column performance for an example bridge.

CHAPTER 2—NOTATION

A	= acceleration coefficient
A_{bl}	= area of longitudinal bar being spliced, in. ² (mm ²)
A_c	= area of confined core measured to outside of transverse reinforcement, in. ² (mm ²)
A_e	= effective concrete area, which may be taken as $0.8A_g$, in. ² (mm ²)
A_{fig}	= cross-sectional area of footing, in. ² (mm ²)
A_g	= gross area of concrete section, in. ² (mm ²)
A_s	= area of longitudinal reinforcement, in. ² (mm ²)
A_{sh}	= cross-sectional area of tie legs, in. ² (mm ²)
A_{shx}	= total cross-sectional area of steel running in the x-direction, in. ² (mm ²)
A_{shy}	= total cross-sectional area of steel running in the y-direction, in. ² (mm ²)
A_{sp}	= cross-sectional area of circular hoop or spiral bar, in. ² (mm ²)
A_{tr}	= total cross-sectional area of all transverse reinforcement that is within spacing s and that crosses the potential plane of splitting through the reinforcement being developed, in. ² (mm ²)
A_v	= effective area of shear reinforcement taken as the projected area of transverse tie bars on a plane perpendicular to the applied shear force, in. ² (mm ²)
B_c	= equivalent embankment width, equal to the width of the embankment at a height of two-thirds of H' above the base of the embankment, in. (mm)
C_1	= displacement amplification factor
C_{1TR}, C_{1L}	= peak displacement coefficient
C_{emb}	= lumped damper property attached on the deck to represent the embankment contribution (deck-pier-abutment substructure model)
C_s	= elastic seismic response coefficient
C_{tot}^*	= generalized damping coefficient
c_b	= spacing or cover dimension, in. (mm)
col	= column
cr	= cracked
D	= diameter of circular column, in. (mm)
D_c	= diameter or depth of column in direction of loading, in. (mm)
$D_{c\ max}$	= larger cross section dimension of the column, in. (mm)
D_{sp}	= diameter of spiral or circular hoop measured to outside face of spiral or circular hoop, in. (mm)
DC	= permanent load
DO_H	= delayed operational performance state for columns with high transverse reinforcement
DO_L	= delayed operational performance state for columns with low transverse reinforcement
d	= effective depth measured to centroid of tension steel; may be taken as $0.8h$, where h is section depth in direction of applied shear, in. (mm)
d_b	= longitudinal bar diameter, in. (mm)

d_{bb}	= effective bar diameter of bundled bars, in. (mm)	K	= footing stiffness, lb/in. (N/m)
d_c	= depth of confined concrete measured to outside of perimeter hoop in the direction of the applied shear force, in. (mm)	K_{abut}	= abutment stiffness, lb/in. (N/m)
e_{rx}	= stiffness embedment factor for rotation about x-axis	K_{bent}	= bent stiffness, lb/in. (N/m)
e_y	= stiffness embedment factor for horizontal translation (toward long side of footing)	K_{deck}	= deck stiffness, lb/in. (N/m)
e_x	= stiffness embedment factor for horizontal translation (toward short side of footing)	K_{emb}	= embankment stiffness, lb/in. (N/m)
EQ_L	= effects of earthquake acting in the longitudinal direction, or related internal moments and forces, lb (kN)	K_o	= footing stiffness without shape and embedment factors, lb/in. (N/m)
EQ_{TR}	= effects of earthquake acting in the transverse direction, or related internal moments and forces	K_{rxp}	= rotational stiffness about x-axis, lb·in. (N·m)
F^*	= force associated with lateral relative displacement of equivalent single degree of freedom system	K_{tr}	= transverse reinforcement index, in. (mm)
F_a	= short-period site coefficient (at $T = 0.2$ seconds)	K_{xp}	= horizontal translational stiffness (toward short side of footing), lb/in. (N/m)
F_v	= long-period site coefficient (at $T = 1.0$ seconds)	K_{yp}	= horizontal translational stiffness (toward long side of footing), lb/in. (N/m)
FF	= fully functional performance state	K_{zp}	= vertical translational stiffness, lb/in. (N/m)
$flex$	= flexible	L	= longitudinal direction of bridge
f'_c	= specified 28-day compressive strength of concrete, psi (MPa)	L'	= embankment effective length, in. (mm)
f_{cc}	= compressive strength of confined concrete, psi (MPa)	L_c	= embankment critical length, in. (mm)
f_{ce}'	= expected concrete compressive strength, psi (MPa)	L_{col}	= distance from the column base to the point of contraflexure (also known as shear span), in. (mm)
f_{co}'	= concrete compressive strength including effects of confinement and aging, psi (MPa)	L_{pr}	= length of the plastic hinge region, in. (mm)
f_s	= stress in longitudinal steel, psi (MPa)	LC_1	= load case 1
f_{ig}	= footing	LC_2	= load case 2
f_y	= specified yield strength of reinforcing steel, psi (MPa)	ℓ_{ac}	= minimum anchorage length, in. (mm)
f_{ye}	= expected yield strength of reinforcing steel, psi (MPa)	ℓ_c	= length along column height between points of zero bending moment and maximum bending moment, in. (mm)
f_{yl}	= yield strength longitudinal reinforcement, psi (MPa)	ℓ_d	= basic development length of a straight bar, in. (mm)
f_{yo}	= steel strength including effects of material overstrength and strain hardening, psi (MPa)	ℓ_{dh}	= development length of a standard hook, in. (mm)
f_{ys}	= yield strength of the transverse reinforcement, psi (MPa)	ℓ_{hb}	= basic development length of a standard hook, in. (mm)
f_{yt}	= specified yield strength transverse reinforcement, psi (MPa)	ℓ_s	= lap splice length, in. (mm)
G	= soil shear modulus, psi (MPa)	ℓ_o	= plastic hinge length where special confinement reinforcement is required, in. (mm)
G_{max}	= soil maximum shear modulus (for low shear strain), psi (MPa)	M	= bending moment, in·lb (N·mm)
g	= acceleration of gravity, in./s ² (mm/s ²)	M^*	= mass of equivalent single degree of freedom system, lbm (g)
H	= clear height from top of footing to bent cap soffit, in. (mm)	M_{center}	= deck mass (center), lbm (g)
H_L	= distance from column base to point of contraflexure determined for longitudinal response, in. (mm)	M_{cr}	= cracking moment, in·lb (N·mm)
H_{TR}	= distance from column base to point of contraflexure determined for transverse analysis, in. (mm)	M_{deck}	= mass of bridge deck
H_{abut}	= height of abutment	M_{edge}	= deck mass (edge), lbm (g)
H_{emb}	= embankment height, in. (mm)	M_{emb}	= generalized embankment mass, lbm (g)
h	= deeper side of a rectangular cross section, in. (mm)	M_n	= nominal flexural strength, in·lb (N·mm)
h_c	= core width perpendicular to applied shear force, measured to outside edge of perimeter hoop, in. (mm)	M_{ne}	= expected nominal flexural strength, in·lb (N·mm)
h_{col}	= depth of column in the direction of the shear, in. (mm)	M_p	= plastic flexural strength, in·lb (N·mm)
I	= gross section inertia, in. ⁴ (mm ⁴)	M_p^{column}	= idealized plastic moment capacity of column calculated by moment-curvature analysis, in·lb (N·mm)
I_{cr}	= cracked section inertia, in. ⁴ (mm ⁴)	M_{pe}	= expected plastic flexural strength, in·lb (N·mm)
I_{xftg}	= moment of inertia of footing about the x-axis	M_{pr}	= probable flexural strength of plastic hinge, in·lb (N·mm)
J	= gross torsion constant, in. ⁴ (mm ⁴)	M_{tot}^*	= generalized mass of the system, lbm (g)
J_{cr}	= cracked torsion constant, in. ⁴ (mm ⁴)	M_u	= bending moment due to factored loads, in·lb (N·mm)
		M_y	= first yield moment, in·lb (N·mm)
		n	= number of bars being developed along the plane of splitting
		n_b	= number of longitudinal bars confined by spiral or circular hoops
		OP	= operational performance state

P	= axial load, lb (N)	α_1	= bent-abutment displacement ratio
P_{EY}	= probability of exceedance in Y years	β_1	= depth factor of rectangular compression stress block
P_b	= nominal axial load strength at balanced strain conditions, lb (N)	Δ	= displacement at contact embankment-abutment node, in. (mm)
P_{dl}	= axial load resulting from dead load, lb (N)	Δ_{BOT}	= displacement at base of column, in. (mm)
P_e	= axial load determined by elastic analysis, lb (N)	Δ_{CF}	= displacement at contraflexure point, in. (mm)
P_n	= nominal axial load strength at a given eccentricity, lb (N)	Δ_{TOP}	= displacement at top of column, in. (mm)
P_u	= axial load including overturning effects, lb (N)	Δ_c	= displacement capacity of the structure, in. (mm)
PI	= plasticity index of embankment soil	Δ_e	= elastic spectral displacement, in. (mm)
R	= strength reduction factor; seismic reduction factor in AASHTO (force-based design)	Δ_u	= peak displacement demand, in. (mm)
R_{eq}	= equivalent circular footing radius, in. (mm)	Δ_r	= relative offset between point of contraflexure and base of plastic hinge, in. (mm)
r_P	= axial load ratio ($P/A_g f'_c$)	Δ_{col}	= displacement at the contraflexure point relative to a tangent at the end of the column, in. (mm)
r_s	= shear span-depth ratio	$\Delta_{col,c}$	= displacement capacity of column, in. (mm)
S	= site class coefficient	$\Delta_{col,y}$	= yield displacement of column, in. (mm)
S_1	= long-period spectral acceleration ($T = 1.0$ seconds), g	Δ_y	= yield displacement, in. (mm)
S_a	= spectral acceleration, g	δ	= drift, in. (mm)
S_s	= short-period spectral acceleration ($T = 0.2$ seconds), g	ϵ_c	= strain at the outermost concrete compressive fiber
S_{D1}	= long-period design spectral acceleration, g	ϵ_{cu}	= ultimate concrete compressive strain capacity
S_{DS}	= short-period design spectral acceleration, g	ϵ_s	= strain in longitudinal steel
s	= center-to-center spacing of transverse steel or spiral pitch measured parallel to the column axis, in. (mm)	ϵ_{suh}	= strain at maximum confinement reinforcement stress
T	= natural period of vibration of structure, s	ϵ_t	= strain at outermost tensile steel layer
T^*	= predominant period of ground motion, s	$\Phi(y,z)$	= embankment deformation shape to be evaluated based on imposed boundary conditions
T_o	= reference period used to define spectral shape = $0.2T_s$, s	ϕ	= strength reduction factor
T_g	= characteristic period of ground motion, s	ϕ_{cp}	= shape vector amplitude at characteristic point at deck level where largest lateral displacement is expected
T_s	= corner period of spectrum, s	ϕ_{emb}	= shape vector amplitude at the top of embankment
TR	= transverse direction of bridge	γ	= average embankment deformation level
u^*	= lateral relative displacement, in. (mm)	γ_{soil}	= unit weight of embankment soil, lb/ft ³ (N/mm ³)
u_1	= total transverse displacement, in. (mm)	Γ	= modal participation factor
u_b	= displacement at bent, in. (mm)	λ	= lightweight aggregate concrete factor
u_{cn}	= displacement of characteristic point, in. (mm)	λ_{mo}	= overstrength magnifier
u_g	= imposed ground displacement, in. (mm)	μ_d	= global ductility demand
$\ddot{u}_g$	= ground acceleration, in./s ² (mm/s ²)	$\mu_{col,d}$	= member ductility demand
u_{tot}	= total transverse displacement, in. (mm)	μ_δ	= displacement ductility demand
V	= shear, lb (N)	ρ_l	= longitudinal reinforcement ratio ($= A_s/A_g$)
V_{bent}	= bent shear, lb (N)	ρ_{min_in}	= minimum transverse steel volumetric ratio inside the plastic hinge zone
V_c	= concrete contribution to shear strength, lb (N)	ρ_{min_out}	= minimum transverse steel volumetric ratio outside the plastic hinge zone
V_e	= design shear strength, lb (N)	ρ_s	= volumetric ratio of transverse reinforcement
V_{emb}	= embankment shear, lb (N)	ψ_e	= epoxy-coating factor
V_n	= nominal shear strength, lb (N)	ψ_s	= reinforcement size factor
V_o	= overstrength shear, lb (N)	ψ_t	= reinforcement location factor ($= 1$ for column reinforcement)
V_p	= plastic shear, lb (N)	$\mathfrak{F}_{emb}$	= embankment excitation factor
V_s	= steel contribution to shear capacity, lb (N)	$\mathfrak{F}_{tot}$	= excitation factor for the entire model
V_u	= factored design shear force, lb (N)		
V_{un}	= normalized force coordinate corresponding to a constant reference displacement ductility, lb (N)		
V_y	= yield shear, lb (N)		
W	= embankment crest width, in. (mm)		
W_E	= effective weight of embankment, lb (N)		
W_s	= total weight of structure, lb (N)		
w'	= embankment width at abutment base, in. (mm)		
w_{avg}	= embankment width at midheight, in. (mm)		
Y	= time period corresponding to a mean return period and probability of exceedance, years		
Z	= response modification factor		

CHAPTER 3—DESIGN OBJECTIVES AND APPROACHES

3.1—Performance-based design philosophy

Performance-based seismic design relates damage, loss of function, and societal consequences anticipated for an infrastructure component such as a bridge or highway system to a defined seismic hazard. For bridge design, this involves the

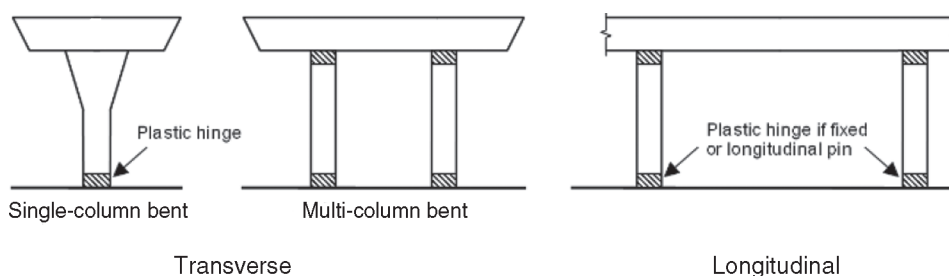


Fig. 3.2—Intended locations of inelastic response for forming mechanisms in single-level bridges.

selection of a suitable structural system and suitable materials, the designation of intended locations of inelasticity, and the comparison of anticipated demands with the capacities associated with the desired performance. In many cases, inelastic response will be intended in reinforced concrete (RC) components, involving the formation of plastic flexural hinges in RC columns. The substantial uncertainty in anticipated future ground motions can be addressed by ensuring capacity for ductile behavior, even where the performance objectives aim for little or no damage. Therefore, the proportioning and detailing of a bridge system to provide sufficient strength and drift capacity, while developing an acceptable ductile flexural mechanism, is a primary objective in performance-based seismic design. This chapter summarizes performance objectives and analytical approaches pertaining to bridge columns and bridge systems, and discusses design approaches applicable to short bridges.

3.2—Ductile mechanisms

The design of bridges has emphasized the development of ductile mechanisms as an alternative to proportioning for elastic response, just as in building design. The types of mechanisms that are encouraged in bridges, however, differ from those sought in buildings. In buildings, the formation of plastic hinges in the columns is discouraged because these elements are critical to the stability of the overlying floors; instead, the formation of plastic hinges in the beams is encouraged. In bridges, longer spans and the need to maintain traffic flow have discouraged the use of mechanisms involving plastic hinging in the beams. Instead, mechanisms that involve plastic hinge formation in the columns are typically preferred, particularly for the vast majority of bridges that have only a single deck level. The column hinges protect the beams from severe damage; damage is easily identified and access for repair is not impeded by traffic. By carefully proportioning member strengths and detailing for ductility, the engineer can force the structure to develop a ductile mode of response (Paulay 1977). The more commonly desired mechanisms of inelastic response for bridges are illustrated in Fig. 3.2.

3.3—Performance states and objectives

While a comprehensive view of performance-based seismic design would consider the continuum of performance anticipated over a probabilistic description of hazard (Moehle and Deierlein 2004), most practical renderings

of performance-based seismic design concepts require the explicit evaluation of performance at a number of discrete hazard levels. Furthermore, while the societal consequences of structural response can be evaluated with appropriate tools and models, such evaluations may be more useful for public policy and institutional decision-making purposes rather than for structural engineers in routine design practice. Consequently, the evaluation and design to limit structural response quantities to acceptable limits is emphasized in this document.

Thus, a performance objective may be considered as a statement of the degree of damage and disruption of service allowed for different (discrete) levels of shaking intensity. The appropriate performance objective for a bridge depends on the consequences of the damage and loss of function. Critical or important bridges are those for which the potential for loss of function is to be minimized because the consequences are deemed unacceptable. In contrast, a reduction in service due to damage by relatively strong ground motion is considered acceptable for standard or ordinary bridges.

In practice, the performance objective is usually evaluated at only one or two intensities of ground shaking. Where two intensities are used, the smaller intensity or more frequently-occurring shaking intensity is described as a serviceability or functional-evaluation ground motion. The stronger intensity or more rare ground motion is known as a maximum-considered or safety-evaluation ground motion, and is also used in cases where only one intensity is considered. The maximum considered earthquake (MCE) is the largest earthquake that is considered reasonable to design structures to resist. In some standards, the MCE is a ground motion having a 2 percent probability of being exceeded in a 50-year exposure, subjected to a cap based on a deterministic assessment of the motion that can be generated by known faults. The deterministic motions are limited by geologic parameters such as fault length and stress drop. The MCE terminology has replaced the maximum-credible earthquake phrasing that had been used at an earlier time.

A critical bridge is one whose continued function is critical to post-earthquake operations. All other bridges are classified as standard. Three possible states are considered for performance-based design. A fourth level, at collapse, never should be a design objective. The degree of damage and disruption to service associated with these performance states is described in Table 3.3a.



Table 3.3a—Performance states of critical bridge elements

Performance state	Damage description
I—Fully functional (FF)	Residual cracks are small enough that no repair is required. The cracks may be caused by flexure and shrinkage, not shear or bond.
II—Operational (O)	Limited damage occurs to structural components, not affecting their structural integrity. Examples include settlement of approach slabs, pounding at expansion joints, yielding of restrainer cables, and spalling of concrete cover. Some yielding of column longitudinal reinforcement is acceptable, but nothing approaching buckling or fracture. Closure of the bridge may be required until an inspection is completed, and partial lane closures may be required to repair damage. Repairs should be completed in the days and weeks following an earthquake.
III—Delayed operation (DO)	Severe damage to columns occurs, such as onset of buckling of longitudinal reinforcement or limited fracture of transverse reinforcement. Some loss of core concrete may occur. Ductile details allow the components to maintain their gravity-load-carrying capacity. Complete replacement of the structure is not anticipated, but repair and replacement of columns requires closure to all but emergency traffic.
IV—Collapse prevention (CP)	Partial or total collapse is imminent, possibly associated with extensive crushing of the concrete core, buckling and fracture of longitudinal steel reinforcement, and extensive fracture of transverse reinforcement. The bridge is closed to traffic, and complete replacement is required.

Table 3.3b—Performance objectives

Shaking intensity at site	Critical bridges	Standard bridges
Serviceability-evaluation shaking intensity	I—Fully functional	II—Operational
Safety-evaluation shaking intensity	II—Operational	III—Delayed operation

These damage states, when associated with various intensities of ground shaking, result in the performance objectives shown in Table 3.3b. The ground motion recurrence intervals to be used for the serviceability evaluation and safety evaluation typically are established considering societal needs and expectations. The shaking intensity or design response spectrum associated with the recurrence intervals may be based on regional hazard maps or may be established on a site-specific basis.

In areas of low seismicity, even the safety evaluation motion may be too small to induce significant damage to a bridge structure. Under such conditions, the bridge can be designed on the basis of elastic analyses. To ensure robustness in the face of the significant uncertainties in hazard evaluation and response analysis, however, consideration should be given to identifying an inelastic mechanism and detailing inelastic regions for ductile behavior.

For comparison, Table 3.3c summarizes the performance objectives described in [ATC-32 \(1996\)](#), and Table 3.3d summarizes those contained in NCHRP 12-49 ([NCHRP 2003](#)). NCHRP 12-49 refers to life-safety and operational performance levels rather than designating a bridge to be critical, standard, important, or ordinary.

The lower shaking intensity level—for example, functional-evaluation ground motion—is defined in [ATC-32 \(1996\)](#) as that which has a probability of approximately 60 percent of not being exceeded during the life of the bridge. The lower shaking intensity level for NCHRP 12-49 ([NCHRP 2003](#))—for example expected earthquake—uses a 50 percent probability of exceedance during an assumed 75-year life of the bridge, corresponding approximately to a 100-year return period. The higher shaking intensity level is defined in [ATC-32 \(1996\)](#) as a maximum credible event, corresponding to a return period of 1000 to 2000 years,

while the NCHRP objectives—for example, Rare Earthquake—use a 3 percent probability of exceedance during the 75-year life of the bridge for the higher shaking intensity, corresponding approximately to a 2500-year return period.

In [AASHTO \(2011\)](#), bridges are designed for the life-safety performance objective for a seismic hazard corresponding to a 7 percent probability of exceedance in 75 years.

The performance states described in Table 3.3a characterize damage for the bridge system. Table 3.3e interprets these performance states in terms of damage to columns and expected repair methods. The damage descriptions in Table 3.3e are explicit yet qualitative in nature. Quantitative limits are needed for design. However, many different indexes are available to quantify damage to columns. Various measures of ductility, such as curvature rotation and displacement; drift; plastic rotation; strain; and measures of energy are commonly used.

Local indexes, such as strains in concrete and reinforcement, are mostly used in research rather than in design practice. Strains in the concrete and reinforcement typically are calculated assuming plane sections remain plane and neglecting bar slip, tension stiffening, and other complex behaviors that are known to occur in RC members, but are too complex to be represented in common structural analysis codes. Choices made to include or exclude these behaviors in modeling affect the calculated results.

Examples of performance limits based on strains at critical sections are given in Tables 3.3f and 3.3g. Table 3.3f summarizes strain limits corresponding to the [ATC-32 \(1996\)](#) limit states, as defined by [Zelinski and Alameddine \(1998\)](#). Table 3.3g summarizes strain limits associated with serviceability and damage control states, as defined by [Kowalsky \(2000\)](#) and [Priestley \(2000\)](#). These values are reasonably consistent with those determined by [Silva and Sritharan \(2011\)](#). [Vosooghi and Saïdi \(2010a\)](#) determined strain limits associated with various performance states on the basis of shaketable tests of scaled models. Results for columns detailed for ductile flexural response are reported in Table 3.3h, for:

1. Damage States DS-1 (described as flexural cracks and corresponding approximately to fully functional performance)



Table 3.3c—Seismic performance criteria (ATC-32 1996)

Ground motion at site	Level of damage and post-earthquake service levels			
	Important bridge		Ordinary bridge	
	Service	Damage	Service	Damage
Functional—Evaluation ground motion*	Immediate	Minimal	Immediate	Repairable
Safety—Evaluation ground motion†	Immediate	Repairable	Limited	Significant

*Functional—Evaluation ground motion: This ground motion may be assessed either deterministically or probabilistically. The determination of this event is to be reviewed by a Caltrans-approved consensus group (ATC-32 1996). This motion is intended to represent an event that has a reasonable probability of not being exceeded (approximately 60 percent) during the life of the bridge.

†Safety—Evaluation ground motion: This ground motion may be assessed either deterministically or probabilistically. The deterministic assessment corresponds to the maximum credible earthquake. The probabilistic ground motion for the safety evaluation typically has a long return period (approximately 1000 to 2000 years). The maximum credible earthquake is the largest earthquake capable of occurring along an earthquake fault, based on current geologic information as defined by the 1996 Caltrans Seismic Hazard Map.

Service Levels:

Immediate: Full access to normal traffic is available following the earthquake.

Limited: Limited access (for example, reduced lanes, light emergency traffic) is possible within days of the earthquake. Full service is restorable within months.

Damage Levels:

Minimal: Although minor inelastic response may occur, post-earthquake damage is limited to narrow cracking in concrete and no apparent permanent deformations; minor inelastic response may occur.

Repairable: Inelastic response may occur, resulting in concrete cracking, reinforcement yielding, and minor spalling of concrete cover.

Significant: There is a minimum risk of collapse. Permanent offsets may occur and damage consisting of cracking, reinforcement yielding, and major spalling of concrete may require closure to repair.

Table 3.3d—Seismic performance objectives (NCHRP 12-49 [NCHRP 2003])

Ground motion at site	Level of damage and post-earthquake service levels			
	Life safety		Operational	
	Service	Damage	Service	Damage
Expected earthquake, 50 percent in 75 years*	Immediate	Minimal	Immediate	Minimal to none
Rare earthquake, 3 percent in 75 years†	Significant disruption	Significant	Immediate	Minimal

*Frequent or expected ground motion: This motion is intended to represent an event that has 50 percent probability of exceedance in 75 years, corresponding to a 100-year return period. This is the design level under which the bridge should remain essentially elastic.

†Rare ground motion: This ground motion has 3 percent probability of exceedance in 75 years, corresponding to a 2500-year return period. This design level is recommended to assure that a “no-collapse” performance criteria for maximum considered earthquake is satisfied.

Service levels:

Immediate: Full access to normal traffic is available following the earthquake.

Significant disruption: Limited access, such as reduced lanes and light emergency traffic, is possible after shoring; however, the bridge may need to be replaced.

Damage levels:

None: Evidence of movement may be present but there is no notable damage.

Minimal: Some visible signs of damage. Inelastic response may occur, resulting in narrow flexural cracking of concrete, the onset of yielding in steel, and no apparent permanent deformations.

Significant: Although there is no collapse, permanent offsets may occur and damage consisting of cracking, major spalling of concrete, reinforcement yielding, local buckling of steel columns, global and local buckling of steel braces, and cracking in the bridge deck slab at shear studs on the seismic load path is possible. The bridge may require closure to repair. Partial or complete replacement may be required in some cases.

Table 3.3e—Reinforced concrete bridge column performance states

Performance state	Column damage state	Expected repair	Expected traffic impact
I—Fully functional (FF)	Negligible residual crack widths, caused by shrinkage or flexure	None	None
II—Operational (O)	Cover spalling acceptable; core intact Limited yielding of longitudinal reinforcement	Epoxy injection Concrete patching	Bridge can be closed to traffic until inspection completed
III—Delayed operation (DO)	Severe damage including crushing of core concrete Limited bar buckling Limited transverse reinforcement fracture	Partial replacement of column(s)	Bridge is closed to all but emergency vehicles
IV—Collapse prevention (CP)	Loss of core concrete Extensive bar buckling Extensive transverse reinforcement fracture Longitudinal reinforcement fracture	Complete replacement of column(s) required	Bridge is closed to all traffic

Table 3.3f—ATC-32 (1996) Limit state strains (Zelinski and Alameddine 1998)

Concrete strain limit	Minimal damage, (1/3) ϵ_{cu} or 0.004	Repairable damage, (2/3) ϵ_{cu} or 0.008	Significant damage, ϵ_{cu} or 0.012
Steel strain limit*			
Grade 420, 10 to 25 mm diameter	0.03	0.08	0.12
Grade 420, 29 to 57 mm diameter	0.03	0.06	0.09

*Grade 420 steel has a nominal yield strength of approximately 60 ksi.

2. DS-4 (spalling and cracking exposing lateral and longitudinal bars, corresponding approximately to the limit of operational performance)

3. DS-5 (corresponding approximately to the limit of delayed operational [DO] performance). Some overlap is apparent in the strain ranges provided in Table 3.3h.

Effects of cover on crack sizes of scaled RC members should be considered. Additional performance descriptors and evaluators are discussed by [Hose et al. \(2000\)](#) and [Vosooghi and Saiidi \(2010a\)](#).

To use the limits provided in Tables 3.3f and 3.3g, the ultimate concrete compressive strain demand is computed from moment-curvature analysis using a confined concrete model—for example, [Mander et al. \(1988\)](#). The ultimate compressive strain capacity of the confined concrete, ϵ_{cu} , depends on parameters such as the amount of transverse reinforcement, the yield strength of the transverse reinforcement, the strain at fracture of the reinforcement, and the compressive strength of the concrete, and may be estimated as

$$\epsilon_{cu} = 0.004 + 1.4\rho_s f_{yh} \epsilon_{suh} / f_{cc} \quad (3.3)$$

The strains ϵ_{suh} and ϵ_{cu} are the steel tensile strain at peak and ultimate, respectively.

Performance states can be defined at the member level. For example, Table 3.3i illustrates a simple approach to quantifying the performance states of columns in terms of the yield and reliable displacement capacities, applicable to columns developing plastic flexural hinges. While various models exist for estimating the reliable deformation capacity of such columns, significant disagreement exists among the models, with estimates often deviating by up to a factor of 2 or more from experimental results (for example, [Inel \(2002\)](#)).

Experimentally determined displacement capacities vary significantly relative to estimates made with various predictive models. Conservative point estimates of displacement capacity, therefore, are required in a deterministic design approach. Experimentally determined drift capacities of cantilever columns are easily established. For addressing columns with different boundary conditions, the approach developed in this report evaluates deformation capacities and demands in terms of drift ratio, where the drift ratio is defined as the displacement normalized by shear span. Drift demands are readily identified in structural analysis results, are unambiguous, and directly meaningful.

Experimental data are reviewed in [Chapter 4](#) to develop recommendations for the drift capacities associated with the performance states of Tables 3.3a and 3.3e. The recommended drift limits are given in [4.4](#) as a function of shear span-depth ratio for columns having transverse reinforce-

Table 3.3g—Limit state strains (Kowalsky 2000)

	Serviceability	Damage control
Concrete strain limit (compression)	0.004	0.018 or ϵ_{cu}
Steel strain limit (tension)	0.015	0.060 or $0.6\epsilon_{su}$

Table 3.3h—Mean tensile strains in longitudinal reinforcement corresponding to visual damage states (Vosooghi and Saiidi 2010a)

Ductile flexural column condition	Visual damage state		
	DS-1	DS-4	DS-5
	Approximate performance state		
	Fully functional	Operational	Delayed operation
Far-field ground motions Shear stress < $4\sqrt{f'_c}$	0.012 ± 0.006	0.035 ± 0.007	0.045 ± 0.014
Far-field ground motions Shear stress > $6\sqrt{f'_c}$	0.005 ± 0.003	0.035 ± 0.008	0.051 ± 0.014
Near-field ground motions Shear stress < $4\sqrt{f'_c}$	0.013 ± 0.003	0.033 ± 0.016	0.038 ± 0.015

Table 3.3i—Performance states expressed in terms of member displacement capacity

Limit state	Deformation limit
Fully functional	1 to 1.5 times the yield displacement
Operational	Concrete spalling (approximately three times the yield displacement)
Delayed operation	Approximately 3/4 of a lower-bound estimate of the displacement capacity

ment that satisfies shear strength and confinement requirements summarized in that section.

The extent of repair associated with different performance states can vary substantially. For further information, the reader is referred to [Vosooghi and Saiidi \(2010b\)](#).

3.4—Earthquake ground motion

The ground motions or shaking intensity associated with a performance objective may be determined using deterministic or probabilistic approaches. Although average recurrence intervals of 1000 to 2500 years are used for the safety evaluation level, written historical records are available for only a few hundred years in the United States, and large numbers of instrumental records have been obtained in recent decades. Information obtained from earlier events

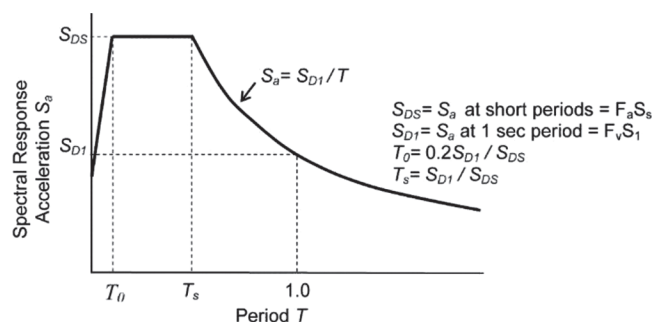


Fig. 3.4—Design response spectrum (AASHTO 2011).

through paleoseismological studies is limited. Projections of future design shaking intensities are based on a relatively narrow historical window for many regions of the country. Records obtained for events of similar magnitudes recorded at similar distances on similar site conditions exhibit substantial variability. Even the computed response to ground motions that represent a given spectral intensity can exhibit substantial variability relative to the mean demands estimated using a smoothed spectrum. Variability, due to randomness and uncertainty in computed responses, requires the design engineer exercise an unusual degree of care in establishing that the performance objectives are likely to be satisfied. Therefore, robust structural systems that are capable of withstanding significant uncertainties in ground motion input, accurate estimates of structural properties, appropriate structural models, and reliable estimates of member drift capacities are sought in even routine designs.

Performance objectives are based on the premise that the earthquake hazard representation for a given recurrence interval is known or can be established for the site in question. The U.S. Geological Survey has prepared maps that establish spectral ordinates having stated probabilities of occurrence. The spectral ordinates at 0.2 and 1 seconds are used to construct a site-specific hazard curve (Fig. 3.4) that represents spectral accelerations that have a desired probability of exceedance in a stated time frame. This is described more fully in [AASHTO \(2011\)](#).

The relationship between the return period, P_R , and probability of exceedance is given by

$$P_R = \frac{-Y}{\ln(1 - P_{EV})} \quad (3.4)$$

where P_{EV} is the probability of exceedance (expressed as a decimal) in Y years for the desired hazard level. The relationship between the intensity of shaking and the recurrence interval is a function of the site-specific hazard.

3.5—Methods of analysis

To establish the adequacy of a design, quantitative assessments of the forces and drifts induced in the components of a bridge are necessary. Both linear and nonlinear analysis methods are available as described in the following, and their applicability to a specific bridge depends on its configuration and the performance state of interest. The proce-

dures, termed linear static procedure (LSP), linear dynamic procedure (LDP), nonlinear static procedure (NSP), and nonlinear dynamic procedure (NDP), generally will give different results due to the attributes of the analysis method, degree of nonlinearity, variability of ground motions, and assumptions introduced by the analyst. Dynamic procedures are generally applied in cases where higher modes may be significant, where the longitudinal and transverse responses may be coupled (as may occur due to pounding at skewed expansion joints), or where the bridge lacks regularity. The static procedures are appropriate for regular bridges where the drift response can be captured by a predominant mode of vibration. Current provisions require that the dynamic effects of gravity loads be explicitly accounted for by using nonlinear response history analysis unless a calculated stability coefficient is below a limiting value, as summarized in [6.6](#).

Guidance for selecting a method of analysis for evaluating column drift demands in straight bridges is given in Table 3.5 on the basis of skew, the presence or absence of expansion joints, and fixity at the abutment. For bridges with no skew, the abutments are oriented perpendicular to the alignment of the bridge. An example is for zero skew angle. Where substantial skew is present, there is significant interaction between the longitudinal and transverse motions, typically resulting in torsional rotations of the deck, in plan, and opening of joints between the deck and abutments. For the purposes of Table 3.5, this can be assumed to occur for skew angles in excess of 20 degrees.

Discussion of analysis procedures follows. Assumptions used in modeling can significantly affect response, and column drift demands in particular. Embankment flexibility, which has been overlooked in the past, can significantly increase column drift demands ([Inel 2002](#)), particularly for columns in bents located within a few spans of the embankment. Methods for modeling embankment flexibility are described in [Chapter 5](#). Additional information on the modeling and analysis of bridge structures can be found in [ACI 341.2R](#).

3.5.1 Linear static procedure—The LSP is the simplest analysis procedure. Response is computed for a static force applied laterally to the structure. The magnitude of this force is determined from an elastic design response spectrum at the calculated fundamental period of vibration. The force is applied at the vertical center of mass and distributed laterally in proportion to the product of the bridge mass and displaced shape.

The LSP is useful for estimating displacement demands on structures where a more sophisticated dynamic procedure will not provide significant additional insight into demands. This procedure may be used for structures whose response is dominated by a single mode, at Performance States I, II, and III in Table 3.3a. Softening of abutment wall resistance should be represented using equivalent linear techniques as described by [Caltrans \(2010\)](#).

3.5.2 Linear dynamic procedure—The LDP includes both response spectrum methods, in which multiple mode response peaks are combined according to a combination

Table 3.5—Influence of configuration on minimum recommended analysis method*

Configuration parameter			Analysis direction	
Skew	Intermediate expansion joints (not at abutments)	Abutment fixity in skew direction	Skew (parallel to abutments)	Perpendicular to skew
Minor (skew < 20 degrees)	One or zero	Fixed	NSP with embankment flexibility	LSP
		Free	NSP	LSP
	Two or more	Fixed	†	LSP
		Free	‡	LSP
Substantial (skew ≥ 20 degrees)	Any number	Fixed	NDP with embankment flexibility	LSP
		Free	NDP	LSP

*For straight bridge alignments where significant softening is expected.

†Column drift demands may be estimated using the LSP or NSP for the middle deck segment, assuming it to be free of the adjacent deck segments. In other cases and for complex bridges, the NDP should be used, and embankment flexibility should be considered.

‡Column drift demands may be estimated using the LSP or NSP for each deck segment assuming each to be free of the adjacent deck segments. In other cases and for complex bridges, the NDP should be used.

Note: Nonlinear or dynamic methods may be used in place of linear or static methods. For example, NSP may be used in place of LSP, and NDP may be used in place of NSP.

rule and response history or time-step methods. Enough modes should be included in a response spectrum analysis to capture at least approximately 90 percent of the mass. Peak modal response quantities can be combined using the square root of sum of squares combination when the modal frequencies are well separated. The complete quadratic combination should be used where two or more modal frequencies are close in value.

The LDP is appropriate for bridges in which higher modes contribute significantly to response. This procedure is applicable to all bridges at Performance State I in Table 3.3a. The forces calculated using the LDP may exceed those associated with the development of a mechanism at Performance States II and III in Table 3.3a.

3.5.3 Nonlinear static procedure—The NSP, also known as a pushover analysis, is a procedure that directly accounts for nonlinear load-deformation responses of the bridge components. The structure is analyzed under gradually increasing lateral forces or displacements. The total lateral force, or base shear, is plotted against the displacement at a control point located on the deck. As displacements increase, components of the model soften and yield until a mechanism forms. The internal forces and deformation demands throughout the structure are evaluated at one or more target displacements. Each target displacement is determined for a hazard level of interest that is associated with each performance objective considered. Estimates of the target displacements can be determined using various procedures. In general, the target displacement is approximately equal to the peak displacement of an elastic system for long-period structures; target displacements of short-period systems undergoing inelastic response exceed the peak displacements of corresponding elastic systems.

The NSP is suitable for structures at the operational and delayed operation performance states whose response is dominated by a single mode. Inelastic deformation that develops in the nonlinear static analysis clearly indicates locations where large inelastic deformations are likely to occur, and provides a more realistic assessment of behavior relative to linear procedures. Design approaches that use the

NSP have been proposed (Fajfar 2000) and have been modified to account for embankment flexibility (Inel 2002).

3.5.4 Nonlinear dynamic procedure (or time history procedure)—The NDP accounts directly for nonlinear load-deformation characteristics of the bridge components by analysis of response to individual ground motion records in the time domain. Because “nonlinearity” fundamentally implies a nonproportional and path-dependent response, a suite of ground motions are used to establish the range of predicted responses. The selection of ground motions and methods used to scale them to achieve appropriate intensities is still unresolved despite significant advances in recent years (Iervolino and Cornell 2005; Baker and Cornell 2006); some discussion is provided in Section 3.4.4 of AASHTO (2013).

The NDP is expected to provide a more accurate measure of behavior compared to linear procedures, accounts for higher modes (unlike the NSP), and potentially can account for multiple support and noncoherent excitations. As an analysis method, the NDP is best suited for determining ranges of predicted demands after a preliminary design has been established.

3.6—Design methods

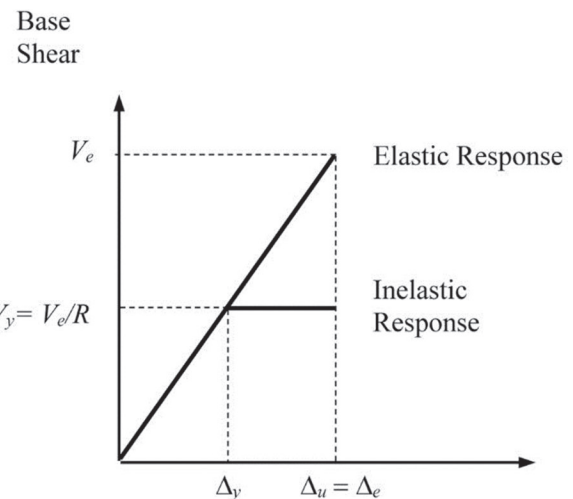
Although force-based approaches have been widely used for seismic design, displacement-based design approaches are gaining favor as a means to ensure adequate performance. In force-based approaches, the strength required to resist the seismic demands is the primary focus, and design level forces are obtained by reducing elastic demands by a strength reduction factor whose value is related to the degree of inelastic response or damage to be expected. With force-based approaches, estimates of displacement demands can be compared with acceptable values toward the end of the design process. Performance-based design makes damage a primary concern, and kinematic relationships between displacement demands and damage allow expected damage to be assessed directly. Member proportions and strengths are established to limit damage within acceptable levels. In principle, either approach can be used, although the directness of a displacement-based approach makes such an approach attractive for performance-based design.

3.6.1 Force-based design method—In force-based design approaches, design forces are obtained by reducing the elastic response forces by a response modification factor. This factor, also termed a strength reduction factor or ductility and risk reduction factor, can be constant or can vary as a function of the fundamental period of vibration and other parameters. Larger response modification factors are associated with larger degrees of inelastic response. With force-based design approaches, engineers can establish the locations where inelastic demands will develop by controlling the distribution of member strengths and then detail these locations to produce adequate ductile behavior. Sufficient deformation capacity has to be available to absorb uncertainty in the computed demands that result from the approximations and assumptions made in analysis and variability in the inelastic response to ground motions.

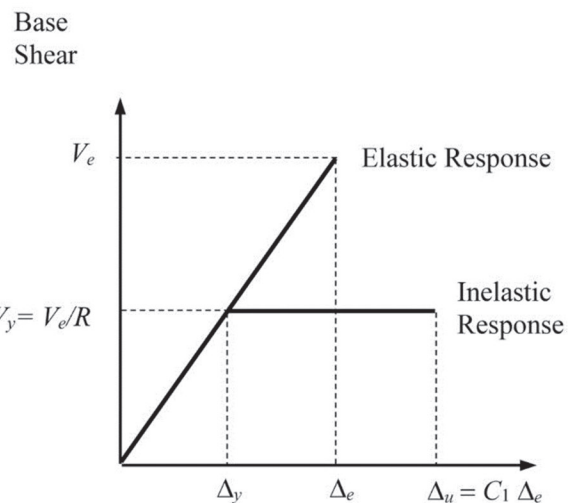
Relationships between the force-reduction factor, expected displacements, and ductility demands have been established based on the computed response of single-degree-of-freedom oscillators. Many seismic design criteria contain an implicit assumption known as the equal displacement rule, which is illustrated in Fig. 3.6.1(a). This rule constitutes an observation that that an upper bound to the peak displacement of a ductile system, having strength V_y , which is less than the strength V_e that would be required for elastic response, is given by the peak displacement, Δ_e , of the elastic system having the same initial stiffness and period of vibration. With reference to Fig. 3.6.1(a), by similar triangles, the displacement ductility demand, $\mu_\delta = \Delta_u/\Delta_y$, is identical to the strength reduction factor R . Although substantial variability exists for individual ground motion excitations, the equal displacement rule does represent mean trends for structures whose fundamental periods are greater than a characteristic period of the ground motion, T_g .

The characteristic period has been evaluated in different ways. Shimazaki and Sozen (1984) identified the characteristic period as the period corresponding to the peak of the equivalent velocity spectrum of a single-degree-of-freedom oscillator. Miranda (1993) identified a predominant period as one at which the pseudo-velocity is a maximum for a single-degree-of-freedom system having damping equal to 5 percent of viscous damping. Cuesta and Aschheim (2001) suggested simply evaluating T_g as $2\pi(S_v)_{max}/(S_a)_{max}$. Where smoothed design spectra are available, the characteristic period can be taken equal to the corner period located at the transition from the constant acceleration portion of the elastic response spectrum to the constant velocity portion of the spectrum. Therefore, with reference to Fig. 3.4, T_s can be considered a reasonable estimate of T_g .

Systems with fundamental periods less than the characteristic period generally have peak displacements larger than those determined by the equal displacement approximation, as illustrated in Fig. 3.6.1(b). Peak displacements can be several times the elastic displacements on average, depending on the period and force reduction factor. A simple expression of this trend is given by the following relations for systems with $T < T_g$,



(a) Displacement Relative to Ground



(b) Displacement Relative to Ground

Fig. 3.6.1—(a) Equal displacement approximation for $T > T_g$; and (b) response for $T < T_g$.

$$\Delta_u = C_1 \Delta_e \quad (3.6.1a)$$

$$C_1 = \left(1 - \frac{1}{R}\right) \frac{T_g}{T} + \frac{1}{R} \geq 1 \quad (3.6.1b)$$

where C_1 modifies the elastic spectral displacement Δ_e to estimate the peak displacement of a yielding system, and R is the ratio of V_e and V_y .

ATC-32 (1996) relates the response modification factor to the fundamental period of the structure, and applies smaller response modification factors to structures where the degree of inelastic response is to be reduced. Ordinary bridges can be designed as full ductility structures, whereas important bridges be designed as limited ductility structures. The response modification factors inferred from ATC-32 (1996) are shown in Table 3.6.1a for performance objectives given in ATC-32 (1996).

Table 3.6.1a—ATC-32 (1996) response modification factors

	Performance objective			
	Ordinary bridges		Important bridges	
Ground motion	Functional evaluation	Safety evaluation	Functional evaluation	Safety evaluation
Z-factor (columns)	Eq. (3.6.1d)	Eq. (3.6.1c)	1	Eq. (3.6.1d)

Table 3.6.1b—Predominant periods of ground motions T^* (ATC-32 1996)

g	Magnitude = 6.5 ± 0.25				Magnitude = 7.25 ± 0.25				Magnitude = 8.0 ± 0.25			
	B	C	D	E	B	C	D	E	B	C	D	E
0.1	0.32	0.45	0.46	0.44	0.41	0.53	0.56	0.56	0.51	0.69	0.71	0.71
0.2	0.37	0.44	0.49	0.64	0.42	0.53	0.55	0.74	0.47	0.61	0.65	0.85
0.3	0.35	0.43	0.50	0.73	0.38	0.51	0.55	0.76	0.48	0.64	0.65	0.98
0.4	0.39	0.47	0.50	0.87	0.42	0.56	0.59	0.93	0.46	0.62	0.66	1.04
0.5	0.37	0.46	0.50	—	0.42	0.53	0.62	—	0.45	0.59	0.70	—
0.6	0.35	0.44	0.50	—	0.43	0.54	0.64	—	0.46	0.60	0.76	—
0.7	—	—	—	—	0.50	0.66	0.76	—	0.54	0.71	0.80	—

Table 3.6.1c—NCHRP 12-49 (NCHRP 2003) response modification factors

	Performance objective			
	Life safety		Operational	
Ground motion	Expected	Rare	Expected	Rare
R-factor (columns)	4	6	1.5	2.5

$$Z = 3 \frac{T}{T^*} + 1 \leq 4 \text{ for full ductility structures} \quad (3.6.1c)$$

$$Z = 2 \frac{T}{T^*} + 1 \leq 3 \text{ for limited ductility structures} \quad (3.6.1d)$$

Note that the response modification factor, termed a Z factor in the [Caltrans \(2010\)](#) provisions, is termed R -factor in the [ATC-32 \(1996\)](#) recommended provisions in the context of building design. Values of predominant period given in ATC-32 (1996) for different soil types B, C, D, and E as a function of earthquake magnitude, peak ground acceleration, and site class are reproduced in Table 3.6.1b.

The response modification factors given in NCHRP 12-49 ([NCHRP 2003](#)) (Table 3.6.1c) neglect the known influence of period on the ratio of the peak and elastic displacements. Peak displacements of short-period systems responding inelastically to ground motions tend to exceed those of corresponding elastic systems. Therefore, the use of period-invariant response modification factors in NCHRP 12-49 (NCHRP 2003) can be expected to result in larger displacements and greater damage in short-period structures, relative to that expected for short-period structures designed with the ATC-32 (1996) response modification factors.

3.6.2 Displacement-based design method—Because deformations of members responding inelastically are a better indicator of damage than the strength reduction factor used in design, displacement-based design approaches

are often favored for performance-based seismic design. Displacement-based design approaches aim to limit the displacement response of a structure consistent with the performance objectives for a given earthquake hazard level. In a displacement-based design approach, system strength is established to limit drift demands to acceptable levels, and related to the strength of plastic hinges associated with the intended mechanism (Fig. 3.2). Sufficient strength has to be provided elsewhere to ensure that the intended mechanism develops, whereas ductile details are provided to ensure adequate deformation capacity at intended locations.

Displacement-based design approaches generally rely on an approximate equivalent single-degree-of-freedom system to establish the strengths and member proportions to control the expected displacement demands as well as the deformation demands in yielding members. For example, [Calvi and Kingsley \(1995\)](#) and [Kowalsky \(2002\)](#) use concepts of equivalent damping to establish the stiffness and strength required to control displacement response. [Vidic et al. \(1994\)](#) describes an approach called the N2 Method that relies on R - μ - T relations (taking the place of equivalent damping) applied to elastic spectra. This approach can make use of the approximate inelastic spectra described by [Chopra and Goel \(1999\)](#) and [Fajfar \(1999\)](#), which emphasize peak displacement, or can use an alternative form of spectra known as yield point spectra ([Aschheim and Black 2000](#)). An example illustrating the direct treatment of P - Δ effects in the performance-based design of a bridge column is provided by [Aschheim and Hernández-Montes \(2003\)](#).

Choices made in modeling the components of a bridge directly affect the results obtained with the aforementioned approaches. Traditional models have emphasized the flexibility of the structural components of the bridge. In the last two decades, abutment flexibility has been modeled routinely, but the abutment bases and bent foundations were assumed to move uniformly together. More recent work has identified that deformation of the foundations can contribute significantly to the displacements of the abut-

ments, affecting the displacement demands imposed on substructure columns. Movement of the embankment can be the major source of column deformation demand in bridges that have relatively stiff decks that are integral with the abutments, typically for bridges that lack intermediate expansion joints and are subjected to strong ground motion (McCallen and Romstad 1994; Werner et al. 1987; Inel 2002; Kotsoglou and Pantazopoulou 2009). Displacement-based design methods that account for embankment flexibility have been proposed by Inel (2002) and Kotsoglou and Pantazopoulou (2008). One finding (Inel 2002) is that for bridges with relatively stiff decks and no intermediate expansion joints, the columns need only be designed to withstand the displacements resulting from embankment deformation. This stands in contrast to traditional design approaches that presume the column strength and stiffness are the primary factors in the degree of inelasticity developed in the columns of such bridges.

CHAPTER 4—PERFORMANCE LIMITS FOR REINFORCED CONCRETE BRIDGE COLUMNS

4.1—Introduction

Performance-based seismic design of reinforced concrete (RC) bridge columns requires that performance states be quantified. Performance states used in this report are introduced in Chapter 3. Performance states are defined by the damage sustained by the column, the repair required, and the action to be taken by the responsible authority, such as the local or state department of transportation.

The behavior exhibited by a column undergoing inelastic response is a consequence of many parameters. Experimental studies have aided in developing an understanding of the effects of various parameters on observed behavior and deformation capacity. The most important parameters are column shear span-depth ratio ($r_s = L_{col}/D$ for cantilever columns), the quantity and detailing of transverse reinforcement, and axial load ratio ($r_p = P/A_g f'_c$). For skewed or torsionally unbalanced bridge configurations, column torsion may be sufficient to affect behavior (Li et al. 2010).

Squat columns are prone to fail in shear, whereas more slender columns may develop ductile flexural behavior. Flexural failures encompass several commonly observed failure modes that characterize the collapse prevention performance limit: compressive failure of concrete, low cycle fatigue of longitudinal bars due to repeated cycles of yielding in tension and buckling in compression, and loss of confinement due to fracture of transverse steel or loss of anchorage, such as hooks not anchored in core. Engineers attempt to avoid shear failures by using heavy transverse reinforcement and light longitudinal reinforcement, and in some cases by making the column longer or making one end a pin connection.

For the operational or delayed operational performance states, failure simply refers to the failure to satisfy a performance objective, which may mean spalling of concrete cover, excessive yielding of steel, or the development of cracks larger than a specified width (Table 3.3a).



Fig. 4.2.1—East face of Pier 33, Hanshin Expressway, 1995 Kobe Earthquake. (Courtesy of NISEE, University of California, Berkeley, CA [Thewalt 1995a].)

Various modes of failure of bridge columns are illustrated in this chapter to better clarify the intentions and objectives of the performance-based approach presented in this document. Specifically, this chapter illustrates both desirable and undesirable types of column behavior observed in past earthquakes, describes the main trends observed in experimental studies, and establishes drift ratios corresponding to the design performance states for columns having varying amounts of confinement as a function of column shear span-depth ratio and axial load ratio.

4.2—Field observations of bridge column performance

Devastating earthquakes throughout the world have provided opportunities to observe the actual performance of bridge columns. Desirable failure modes are associated with ductile response, while undesirable failure modes are associated with a loss of seat support or rapid loss of strength at low levels of ductility, characteristic of a brittle failure mode. Changes in design provisions, partly in response to these earthquakes, have improved design practice and presumably have reduced the likelihood of occurrence and severity of undesirable modes of behavior. Damage observed in past earthquakes is illustrated in this section. Included are photographs representing the performance states of Table 3.3e as well as undesirable failure modes.

4.2.1 Fully functional performance state—Figure 4.2.1 shows light flexural cracking on a square pier after the 1995 Kobe Earthquake. This type of damage is classified as fully functional (FF) performance.

4.2.2 Operational performance state—Figures 4.2.2a through 4.2.2d show examples of damage consistent with the operational performance state. Figure 4.2.2a shows substantial horizontal cracks at the base of a circular column after the Kobe Earthquake, indicative of flexural hinging. Figure 4.2.2b shows spalled concrete at the base of a large-diameter circular concrete bridge column after the Kobe Earthquake. Figure 4.2.2c shows spalled cover concrete after the 1994 Northridge Earthquake. Figure 4.2.2d shows spalled cover



Fig. 4.2.2a—Cracking and spalling, circular column, Pier 40 of the Hanshin Expressway, 1995 Kobe Earthquake. (Courtesy of NISEE, University of California, Berkeley, CA [Thewalt 1995b].)



Fig. 4.2.2b—Spalled concrete on the northwest face of Pier 34, Hanshin Expressway, 1995 Kobe Earthquake. (Courtesy of NISEE, University of California, Berkeley, CA [Thewalt 1995c].)

concrete in a bridge pylon after the 1999 Chi-Chi Earthquake in Taiwan.

4.2.3 Delayed operational performance state—Figure 4.2.3 shows a bridge column observed in Kobe, Japan following the 1995 earthquake, where fully spalled cover and some damage to the core concrete is observed; the column had permanent drift after the earthquake equal to approximately 2 percent of its height. This damage is consistent with a delayed operation performance state.

4.2.4 Undesirable failure modes—Figures 4.2.4a through 4.2.4l provide examples of undesirable failure modes. Figure 4.2.4a shows a shear failure that occurred just below a column flare in the 1994 Northridge Earthquake. Figure 4.2.4b shows shear failures that occurred during the 1971 San Fernando Earthquake. Figures 4.2.4c and 4.2.4d show failed transverse reinforcement and damage to core concrete of bridge piers that occurred in the 1995 Kobe Earthquake. Figures 4.2.4e and 4.2.4f show shear failures. Figures 4.2.4g and 4.2.4h show a collapsed bridge near Watsonville, CA, after the 1989 Loma Prieta Earthquake. The collapse resulted from the piles punching through the deck after failing in shear. Figure 4.2.4i shows an example of column failure at the Cypress viaduct during the Loma Prieta Earthquake. Figure 4.2.4j shows apparent shear failures resulting in collapse of bridges after the Chi Chi Earthquake. Figure



Fig. 4.2.2c—Mission-Gothic Bridge cover spalling, State Route 118, 1994 Northridge Earthquake. (Courtesy of NISEE, University of California, Berkeley, CA [Draher 1994].)



Fig. 4.2.2d—Spalling of concrete at main pylon of the Jyi-Lu Bridge, Chi-Chi Earthquake 1999. (Courtesy of NISEE, University of California, Berkeley, CA [Moehle 1999a].)

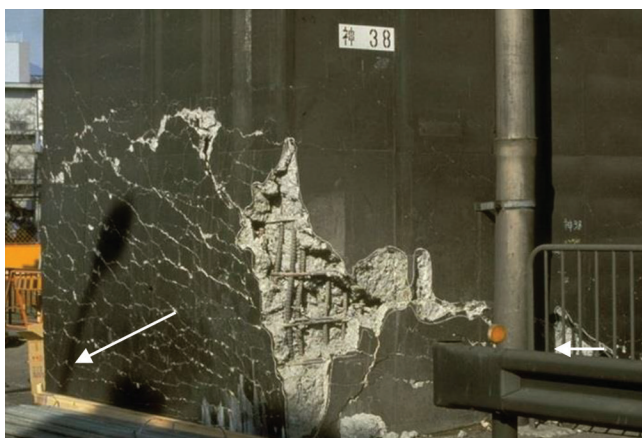


Fig. 4.2.3—Heavy flexural cracking and crushing of concrete, Pier 38 of the Hanshin Expressway, 1995 Kobe Earthquake. (Courtesy of NISEE, University of California, Berkeley, CA [Thewalt 1995d].)

4.2.4k shows the complete loss of freeway structures due to column failures and inadequate seat lengths. Figure 4.2.4l shows an example of anchorage failure in a column during the San Fernando Earthquake.

4.3—Laboratory observations of bridge column performance

This section describes column performance observed in laboratory studies, paying particular attention to the difference between shear-critical and flexurally dominated behavior. The progression of damage observed in experiments on flexurally dominated bridge columns is briefly summarized. Finally, the effect of variation of shear span-depth ratio, transverse reinforcement ratio, longitudinal reinforcement ratio, axial load ratio, and lateral displacement history on column performance is reviewed. Recent studies have also evaluated the influence of torsion in combination with flexure and shear (Prakash et al. 2010).

4.3.1 Shear-critical versus flexurally dominated columns—

Ductile behavior is ensured by relying on the development



Fig. 4.2.4a—Shear failure at base of column flare, Mission-Gothic Bridge, 1994 Northridge Earthquake. (Courtesy of NISEE, University of California, Berkeley, CA [Aschheim 1994].)



Fig. 4.2.4b—Shear failure of several columns, possibly associated with deck rotation, Foothills Freeway overpass, 1971 San Fernando Earthquake. (Courtesy of NOAA/NGDC, U.S. Geological Survey [Leyendecker 1971a].)

of yielding in flexure. Ductile flexural response can be obtained only if sufficient shear strength is provided. The shear demand on the column is a function of the flexural strength, length of the column, and degree of fixity. Column lengths could vary over the length of the bridge to accommodate changes in bridge elevation and terrain. Column cross section dimensions are usually determined for the longest columns, causing some columns to be relatively stocky or squat (short relative to section depth). These relatively squat columns are stiffer than the more slender columns



Fig. 4.2.4c—Failure of transverse reinforcement and damage to column core at base of Pier 86 of the Hanshin Expressway, 1995 Kobe Earthquake. (Courtesy of NISEE, University of California, Berkeley, CA [Thewalt 1995e].)



Fig. 4.2.4e—Shear failure of Pier 165, Hanshin Expressway, 1995 Kobe Earthquake. (Courtesy of NISEE, University of California, Berkeley, CA [Thewalt 1995g].)



Fig. 4.2.4d—Failure of transverse reinforcement and damage to column core at upper portion of Pier 374 of the Hanshin Expressway, 1995 Kobe Earthquake. (Courtesy of NISEE, University of California, Berkeley, CA [Thewalt 1995f].)



Fig. 4.2.4f—Column shear failure, Wu-Hsi Bridge, 1999 Chi Earthquake. (Courtesy of NISEE, University of California, Berkeley, CA [Moehle 1999b].)



Fig. 4.2.4g—Overview of collapse of Highway 101 at Struve Slough, 1989 Loma Prieta Earthquake. (Courtesy of NISEE, University of California, Berkeley, CA [Boulanger 1989a].)



Fig. 4.2.4h—Pile failures, Highway 101 at Struve Slough, 1989 Loma Prieta Earthquake. (Courtesy of NISEE, University of California, Berkeley, CA [Boulanger 1989b].)



Fig. 4.2.4i—Column and joint failure of double-deck viaduct, I-880, 1989 Loma Prieta Earthquake. (Courtesy of NISEE, University of California, Berkeley, CA [Dickenson 1989].)



Fig. 4.2.4j—Column shear failure and loss of seat support, Wu-Hsi Bridge, 1999 Chi-Chi Earthquake. (Courtesy of NISEE, University of California, Berkeley, CA [Moehle 1999c].)



Fig. 4.2.4k—Column failures and loss of seat support, Interstates 5 and 210 in Southern California, 1971 San Fernando Earthquake. (Courtesy of NISEE, University of California, Berkeley, CA [Steinbrugge 1971].)



Fig. 4.2.4l—Anchorage failure, Interstate 5/210 interchange, 1971 San Fernando Earthquake. (Courtesy of NOAA/NGDC, U.S. Geological Survey [Leyendecker 1971b].)

and, therefore, subjected to higher shear forces. As a result of their relatively low shear span-depth ratio, large shear forces are required to develop flexural hinging at the ends of squat columns, and could exceed the inherent shear capacity of the column, leading to a shear failure that takes place at a displacement that could be many times smaller than the displacement associated with plastic hinging of more slender columns. While flexural behavior can be detailed to be ductile and typically does not compromise support for gravity loads, shear failures are brittle and often compromise the ability of a column to carry gravity loads. The failure of short columns in shear has been cited as the cause of collapse of bridges in many earthquakes. A good example is the collapse of both separation ramps on the I5-14 interchange in Los Angeles in the 1994 Northridge Earthquake.

Experimental studies indicate that the shear that can be carried within a flexural plastic hinge decreases with increasing inelastic deformation demands (FHWA 1983; Aschheim and Moehle 1992). A column that is yielding in flexure, therefore, could abruptly fail in shear if an insufficient amount of transverse reinforcement is present. The

design guidelines in Chapter 6 promote the development of flexural hinges. Design engineers attempt to avoid shear failure by using heavy transverse reinforcement and light longitudinal reinforcement, and can extend the length of the column, introduce a pin, or both, at one end to reduce the shear demands and increase the shear capacity.

4.3.2 Observed damage states—Reinforced concrete columns are designed from different perspectives of performance intentions and objectives, but may experience unexpected damage states involving complicated failure modes from combinations of flexure, shear, and torsion. Therefore, the damage states should be clearly defined according to various failure modes for appropriate loading conditions (Prakash and Belarbi 2010). A significant body of experimental research has been conducted focusing on the ultimate capacity of bridge columns (for example, the most severe performance level) under combined flexure and shear. As interest in performance-based earthquake engineering has grown, more recent research has begun to focus on the progression of damage from initial cracking through ultimate failure (Calderone et al. 2000; Choi et al. 2007; Correia and Saiidi 2005; Correia et al. 2007b; Johnson et al. 2008; Kunnath et al. 1997; Lehman and Moehle 2000; Phan et al. 2007; Pulido-Collantes et al. 2001; Saiidi and Mortensen 2002; Saiidi et al. 2004). Interpretation of crack widths observed in tests of scaled RC members should account for the effect of cover thickness. Researchers also have been culling and reevaluating previous experimental data to quantify intermediate performance states (Berry and Eberhard 2003, 2005; Berry et al. 2004; Vosooghi and Saiidi 2010a).

Damage states of interest include concrete cracking, longitudinal reinforcement yielding, initial spalling of cover concrete, complete spalling of cover concrete, buckling of longitudinal reinforcement, and fracture of spiral and longitudinal reinforcement (Fig. 4.3.2). Column failure, defined as significant loss in lateral load-carrying capacity, typically results from bar buckling, bar fracture, or both. In some cases, loss of lateral load-carrying capacity occurs together with loss of gravity load support; in others, gravity load support is maintained well beyond loss of lateral load-carrying capacity.

4.3.3 Influence of column parameters on seismic performance—Implementation of performance-based design procedures require one or more methods to estimate performance at the element level. Engineering demand parameters refer to various indexes that can be associated with important damage states. Engineering demand parameters can describe performance at a global level, using indexes such as drift or displacement ductility or, at a local level, using parameters such as curvature or strain. Local engineering parameters have been used to evaluate seismic performance. An example is Lehman and Moehle (2000). Use of local parameters, however, is difficult because: 1) most researchers do not report values of local parameters corresponding to a given damage state such as compressive strain related to concrete spalling; and 2) current models that are used in practice do not provide reliable estimates of local engineering demand parameters. Damage index models have also been suggested

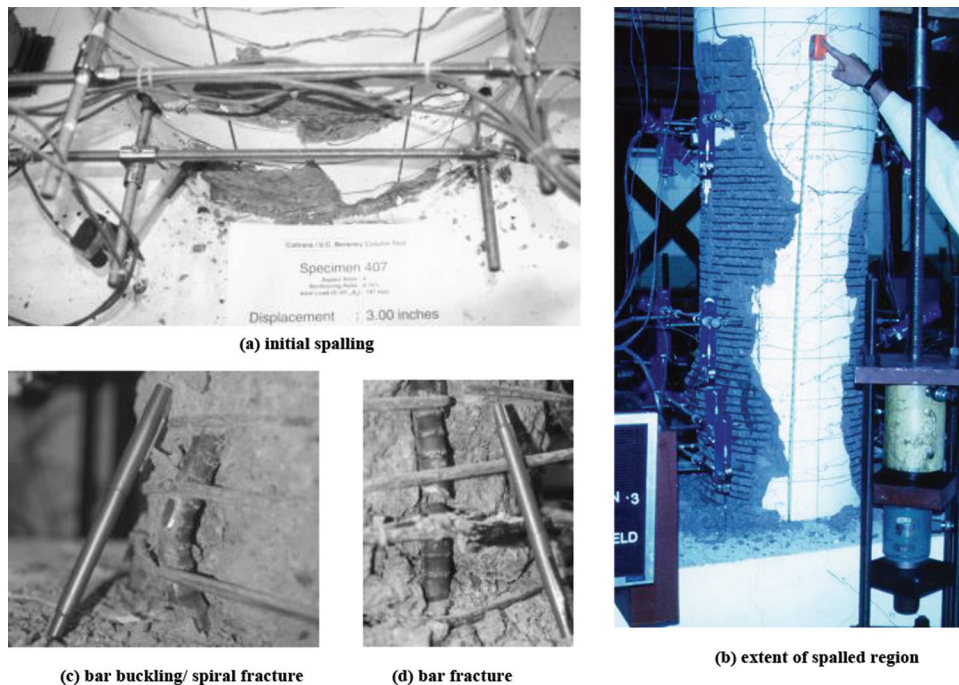


Fig. 4.3.2—Damage states observed during laboratory testing (Lehman et al. 2004).

such as for the case of combined flexure, shear, and torsion in the model put forward by [Prakash and Belarbi \(2010\)](#). Damage index models, however, are not easily implemented in design office settings and not yet widely accepted. For these reasons, the drift ratio corresponding to the onset of particular damage states is evaluated in this section. This parameter is used for simplicity, and is preferred over ductility because of the uncertainty associated with estimates of the yield displacement.

[Berry and Eberhard \(2003\)](#) evaluated the influence of key parameters such as column geometry; reinforcement; and loading on the drift ratio, displacement ductility, plastic rotation, and longitudinal strain corresponding to specific damage states in RC columns. A large comprehensive database of RC columns tested under constant axial load and uniaxial bending was compiled as part of this research effort. The database ([Berry et al. 2004](#)) identifies column geometry, material properties, and reinforcement details for each test specimen. Digital force-displacement histories are also available, along with the observed displacements corresponding to the onset of cover spalling and buckling of longitudinal bars. These damage states approximately correspond to the operational and delayed operation performance states, respectively. Note that the displacements reported in the histories and the cover spalling and bar buckling limit states are the tip displacements of an equivalent cantilever column; data for each column in the database has been reduced to represent the response of a cantilever column with fixed base rotations. The process used for transforming the data is reported in [Berry et al. \(2004\)](#).

A subset of the entire column dataset was used by [Berry and Eberhard \(2003, 2005\)](#). The subset was obtained by selecting specimens that satisfied the following criteria.

- a) There was sufficient transverse reinforcement to make the column flexure-critical, as defined by [Berry et al. \(2004\)](#).
- b) The shear span-depth ratio exceeded 1.9.
- c) The longitudinal reinforcement was continuous, not spliced.
- d) The displacement at or preceding cover spalling or bar buckling was documented.

These criteria were satisfied by 114 rectangular-reinforced and 52 spiral-reinforced column specimens. Of the 114 rectangular-reinforced columns, 102 had cover spalling documented and 62 had bar buckling documented. Of the 52 spiral-reinforced columns, 40 had cover spalling documented and 42 reported bar buckling. Key column parameters for the sets of rectangular and spirally reinforced columns are summarized in Tables 4.3.3a and 4.3.3b, respectively. A detailed list of the column specimens is provided in [Berry and Eberhard \(2003, 2005\)](#).

These data subsets are used to illustrate the effects of key parameters on column performance as follows. [Berry and Eberhard \(2003\)](#) combined plastic-hinge analysis with approximations for the column yield displacement, plastic curvature, buckling strain, and plastic-hinge length to develop three relationships linking column damage to three commonly used engineering demand parameters: plastic rotation, drift ratio, and displacement ductility. Results for drift ratio are described as follows.

An estimate of the mean drift ratio at the onset of cover concrete spalling is given by

$$\frac{\Delta_{spall_calc}}{L_{col}} (\%) = 1.6 \left(1 - \frac{P}{A_g f'_c} \right) \left(1 + \frac{L_{col}}{10h_{col}} \right) \quad (4.3.3a)$$

An estimate of the mean drift ratio at the onset of bar buckling, as determined by [Berry and Eberhard \(2005\)](#) is

Table 4.3.3a—Summary statistics for key column parameters (spirally reinforced)*

	L_{col} , mm	L_{col}/h_{col}	ρ_{eff}	$P/A_g f'_c$	S/d_b	f_{yt} , MPa	f'_c , MPa
Mean	2238	4.7	0.13	0.15	2.81	4.36	35
COV	0.72	0.41	0.40	0.95	0.53	0.15	0.32
Minimum	750	2.0	0.01	0.00	0.83	294	23
Maximum	9140	10.0	0.27	0.70	8.44	565	90

* $\rho_{eff} = \rho_s f_{ys}/f'_c$, where ρ_s is the volumetric transverse reinforcement ratio.

Note: 1 in. = 25.4 mm, and 1000 psi = 6.895 MPa.

Table 4.3.3b—Summary statistics for key column parameters (rectilinear transverse reinforcement)*

	L_{col} , mm	L_{col}/h_{col}	ρ_{eff}	$P/A_g f'_c$	S/d_b	f_{yt} , MPa	f'_c , MPa
Mean	1482	4.5	0.23	0.32	4.33	455	59
COV	0.31	0.32	0.50	0.48	0.38	0.09	0.51
Minimum	450	2.0	0.02	0.09	2.54	363	21
Maximum	2335	7.6	0.64	0.70	14.40	521	118

* $\rho_{eff} = \rho_s f_{ys}/f'_c$, where ρ_s is the volumetric transverse reinforcement ratio.

Note: 1 in. = 25.4 mm, and 1000 psi = 6.895 MPa.

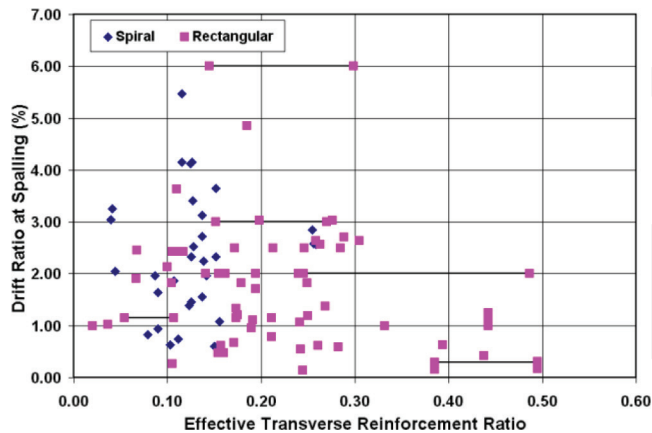


Fig. 4.3.3a—Drift ratio versus effective transverse reinforcement ratio at spalling.

$$\frac{\Delta_{bb_calc}}{L_{col}} (\%) = 3.25 \left(1 + k_{e_bb} \rho_{eff} \frac{d_b}{h_{col}} \right) \left(1 - \frac{P}{A_g f'_c} \right) \left(1 + \frac{L_{col}}{10 h_{col}} \right) \quad (4.3.3b)$$

where $k_{e_bb} = 40$ for all rectangular-reinforced columns and 150 for all spiral-reinforced columns, and $\rho_{eff} = \rho_s f_{ys}/f'_c$. Because little data are available for large values of s/d_b , it is recommended that k_{e_bb} be taken as 0 for columns with s/d_b exceeding 6. Note these equations were calibrated by setting the mean measured-to-calculated drift ratio at the onset of the various damage states to 1.0 and minimizing the coefficient of variation.

The forms of Eq. (4.3.3a) and Eq. (4.3.3b) were selected to be consistent with the expected and observed behavior of RC columns. The axial load ratio term is consistent with the results of moment-curvature analyses, which indicate that, for the same maximum compressive strain, columns with higher axial loads have lower curvatures at the performance states of interest. The shear span-depth ratio, L_{col}/h_{col} , affects the drift ratio at the onset of damage—as L_{col}/h_{col} increases,

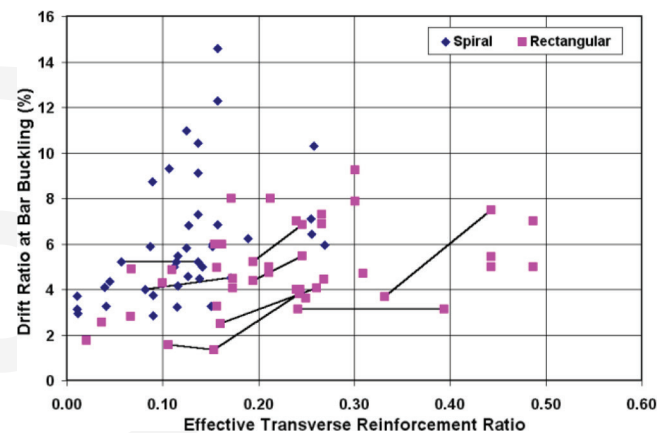


Fig. 4.3.3b—Drift ratio versus effective transverse reinforcement ratio at bar buckling.

yield displacement and length of plastic hinge increase. In Eq. (4.3.3b), the effective confinement ratio ρ_{eff} accounts for the restraint against bar buckling that the transverse reinforcement provides. The values of k_{e_bb} for spiral- and rectangular-reinforced columns differ because spiral reinforcement is more effective than rectangular ties at confining the core concrete and restraining the longitudinal reinforcement from buckling. The data do not show an appreciable influence of s/d_b , in part because bars can buckle over more than one tie spacing and because the dataset had relatively few columns with large s/d_b values. The normalized bar diameter, d_b/D , reflects the increased stability of larger bars and the influence of strain penetration into the column base.

The experimental data support the form of Eq. (4.3.3a) and Eq. (4.3.3b). Figures 4.3.3a through 4.3.3f plot drift ratios at the onset of spalling or bar buckling as functions of effective confinement ratio, axial load ratio, and shear span-depth ratio. Because many parameters are involved, two-dimensional plots do not reveal the influence of all important parameters. Consequently, some of the scatter apparent in these figures is attributable to the influence