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 pushover 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.

CONTENTS

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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 capacities 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
$A_{b\ell}$ = 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_{ftg} = 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
$\mathrm{DO_H}$ = delayed operational performance state for columns with high transverse reinforcement
$\mathrm{DO_L}$ = 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 force, in. (mm)
d_b = longitudinal bar diameter, in. (mm)

d_{bb} = effective bar diameter of bundled bars, in. (mm)
d_c = depth of confined concrete measured to outside of perimeter hoop in the direction of the applied shear force, in. (mm)
e_{rx} = stiffness embedment factor for rotation about x-axis
e_y = stiffness embedment factor for horizontal translation (toward long side of footing)
e_x = stiffness embedment factor for horizontal translation (toward short side of footing)
$\mathrm{EQ_L}$ = effects of earthquake acting in the longitudinal direction, or related internal moments and forces, lb (kN)
$\mathrm{EQ_{TR}}$ = effects of earthquake acting in the transverse direction, or related internal moments and forces
F^* = force associated with lateral relative displacement of equivalent single degree of freedom system
F_a = short-period site coefficient (at T = 0.2 seconds)
F_v = long-period site coefficient (at T = 1.0 seconds)
FF = fully functional performance state
$flex$ = flexible
f_c' = specified 28-day compressive strength of concrete, psi (MPa)
f_{cc} = compressive strength of confined concrete, psi (MPa)
f_{ce}' = expected concrete compressive strength, psi (MPa)
f_{co}' = concrete compressive strength including effects of confinement and aging, psi (MPa)
f_s = stress in longitudinal steel, psi (MPa)
f_{tg} = footing
f_y = specified yield strength of reinforcing steel, psi (MPa)
f_{ye} = expected yield strength of reinforcing steel, psi (MPa)
f_{yl} = yield strength longitudinal reinforcement, psi (MPa)
f_{yo} = steel strength including effects of material overstrength and strain hardening, psi (MPa)
f_{ys} = yield strength of the transverse reinforcement, psi (MPa)
f_{yt} = specified yield strength transverse reinforcement, psi (MPa)
G = soil shear modulus, psi (MPa)
G_{max} = soil maximum shear modulus (for low shear strain), psi (MPa)
g = acceleration of gravity, in./s^2 (mm/s^2)
H = clear height from top of footing to bent cap soffit, in. (mm)
H_L = distance from column base to point of contraflexure determined for longitudinal response, in. (mm)
H_{TR} = distance from column base to point of contraflexure determined for transverse analysis, in. (mm)
H_{abut} = height of abutment
H_{emb} = embankment height, in. (mm)
h = deeper side of a rectangular cross section, in. (mm)
h_c = core width perpendicular to applied shear force, measured to outside edge of perimeter hoop, in. (mm)
h_{col} = depth of column in the direction of the shear, in. (mm)
I = gross section inertia, in.^{4} (mm^4)
I_{cr} = cracked section inertia, in.^{4} (mm^4)
$I_{x\,ftg}$ = moment of inertia of footing about the x-axis
J = gross torsion constant, in.^{4} (mm^4)
J_{cr} = cracked torsion constant, in.^{4} (mm^4)
K = footing stiffness, lb/in. (N/m)
K_{abut} = abutment stiffness, lb/in. (N/m)
K_{bent} = bent stiffness, lb/in. (N/m)
K_{deck} = deck stiffness, lb/in. (N/m)
K_{emb} = embankment stiffness, lb/in. (N/m)
K_o = footing stiffness without shape and embedment factors, lb/in. (N/m)
K_{rxp} = rotational stiffness about x-axis, lb·in. (N·m)
K_{tr} = transverse reinforcement index, in. (mm)
K_{xp} = horizontal translational stiffness (toward short side of footing), lb/in. (N/m)
K_{yp} = horizontal translational stiffness (toward long side of footing), lb/in. (N/m)
K_{zp} = vertical translational stiffness, lb/in. (N/m)
L = longitudinal direction of bridge
L' = embankment effective length, in. (mm)
L_c = embankment critical length, in. (mm)
L_{col} = distance from the column base to the point of contraflexure (also known as shear span), in. (mm)
L_{pr} = length of the plastic hinge region, in. (mm)
LC_1 = load case 1
LC_2 = load case 2
ℓ_{ac} = minimum anchorage length, in. (mm)
ℓ_c = length along column height between points of zero bending moment and maximum bending moment, in. (mm)
ℓ_d = basic development length of a straight bar, in. (mm)
ℓ_{dh} = development length of a standard hook, in. (mm)
ℓ_{hb} = basic development length of a standard hook, in. (mm)
ℓ_s = lap splice length, in. (mm)
ℓ_0 = plastic hinge length where special confinement reinforcement is required, in. (mm)
M = bending moment, in.·lb (N·mm)
M^* = mass of equivalent single degree of freedom system, lbm (g)
M_{center} = deck mass (center), lbm (g)
M_{cr} = cracking moment, in.·lb (N·mm)
M_{deck} = mass of bridge deck
M_{edge} = deck mass (edge), lbm (g)
M_{emb} = generalized embankment mass, lbm (g)
M_n = nominal flexural strength, in.·lb (N·mm)
M_{ne} = expected nominal flexural strength, in.·lb (N·mm)
M_p = plastic flexural strength, in.·lb (N·mm)
M_p^{column} = idealized plastic moment capacity of column calculated by moment-curvature analysis, in. ·lb (N·mm)
M_{pe} = expected plastic flexural strength, in.·lb (N·mm)
M_{pr} = probable flexural strength of plastic hinge, in.·lb (N·mm)
M_{tot}^* = generalized mass of the system, lbm (g)
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)
P_{EY} = probability of exceedance in Y years
P_b = nominal axial load strength at balanced strain conditions, lb (N)
$P_{d\ell}$ = axial load resulting from dead load, lb (N)
P_e = axial load determined by elastic analysis, lb (N)
P_n = nominal axial load strength at a given eccentricity, lb (N)
P_u = axial load including overturning effects, lb (N)
PI = plasticity index of embankment soil
R = strength reduction factor; seismic reduction factor in AASHTO (force-based design)
R_{eq} = equivalent circular footing radius, in. (mm)
r_P = axial load ratio ($P/A_g f_c'$)
r_s = shear span-depth ratio
S = site class coefficient
S_1 = long-period spectral acceleration (T = 1.0 seconds), g
S_a = spectral acceleration, g
S_s = short-period spectral acceleration (T = 0.2 seconds), g
S_{D1} = long-period design spectral acceleration, g
S_{DS} = short-period design spectral acceleration, g
s = center-to-center spacing of transverse steel or spiral pitch measured parallel to the column axis, in. (mm)
T = natural period of vibration of structure, s
T^* = predominant period of ground motion, s
T_o = reference period used to define spectral shape = $0.2T_s$, s
T_g = characteristic period of ground motion, s
T_s = corner period of spectrum, s
TR = transverse direction of bridge
u^* = lateral relative displacement, in. (mm)
u_1 = total transverse displacement, in. (mm)
u_b = displacement at bent, in. (mm)
u_{cn} = displacement of characteristic point, in. (mm)
u_g = imposed ground displacement, in. (mm)
$\ddot{u}_g$ = ground acceleration, in./s^2 (mm/s^2)
u_{tot} = total transverse displacement, in. (mm)
V = shear, lb (N)
V_{bent} = bent shear, lb (N)
V_c = concrete contribution to shear strength, lb (N)
V_e = design shear strength, lb (N)
V_{emb} = embankment shear, lb (N)
V_n = nominal shear strength, lb (N)
V_o = overstrength shear, lb (N)
V_p = plastic shear, lb (N)
V_s = steel contribution to shear capacity, lb (N)
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
α_1 = bent-abutment displacement ratio
β_1 = depth factor of rectangular compression stress block
Δ = displacement at contact embankment-abutment node, in. (mm)
Δ_{BOT} = displacement at base of column, in. (mm)
Δ_{CF} = displacement at contraflexure point, in. (mm)
Δ_{TOP} = displacement at top of column, in. (mm)
Δ_c = displacement capacity of the structure, in. (mm)
Δ_e = elastic spectral displacement, in. (mm)
Δ_u = peak displacement demand, in. (mm)
Δ_r = relative offset between point of contraflexure and base of plastic hinge, in. (mm)
Δ_{col} = displacement at the contraflexure point relative to a tangent at the end of the column, in. (mm)
$\Delta_{col,c}$ = displacement capacity of column, in. (mm)
$\Delta_{col,y}$ = yield displacement of column, in. (mm)
Δ_y = yield displacement, in. (mm)
δ = drift, in. (mm)
ε_c = strain at the outermost concrete compressive fiber
ε_{cu} = ultimate concrete compressive strain capacity
ε_s = strain in longitudinal steel
ε_{suh} = strain at maximum confinement reinforcement stress
ε_t = strain at outermost tensile steel layer
$\Phi(y,z)$ = embankment deformation shape to be evaluated based on imposed boundary conditions
ϕ = strength reduction factor
ϕ_{cp} = shape vector amplitude at characteristic point at deck level where largest lateral displacement is expected
ϕ_{emb} = shape vector amplitude at the top of embankment
γ = average embankment deformation level
γ_{soil} = unit weight of embankment soil, lb/ft^3 (N/mm^3)
Γ = modal participation factor
λ = lightweight aggregate concrete factor
λ_{mo} = overstrength magnifier
μ_d = global ductility demand
$\mu_{col,d}$ = member ductility demand
μ_δ = displacement ductility demand
ρ_l = longitudinal reinforcement ratio (= A_s/A_g)
ρ_{min_in} = minimum transverse steel volumetric ratio inside the plastic hinge zone
ρ_{min_out} = minimum transverse steel volumetric ratio outside the plastic hinge zone
ρ_s = volumetric ratio of transverse reinforcement
ψ_e = epoxy-coating factor
ψ_s = reinforcement size factor
ψ_t = reinforcement location factor (= 1 for column reinforcement)
$\Im_{emb}$ = embankment excitation factor
$\Im_{tot}$ = excitation factor for the entire model

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

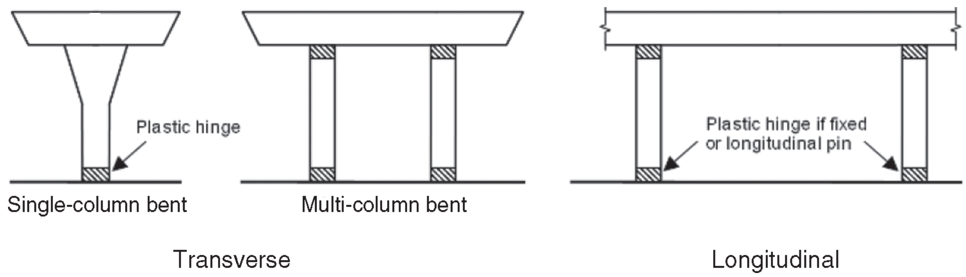

Fig. 3.2—Intended locations of inelastic response for forming mechanisms in single-level bridges.

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 Saiidi (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)