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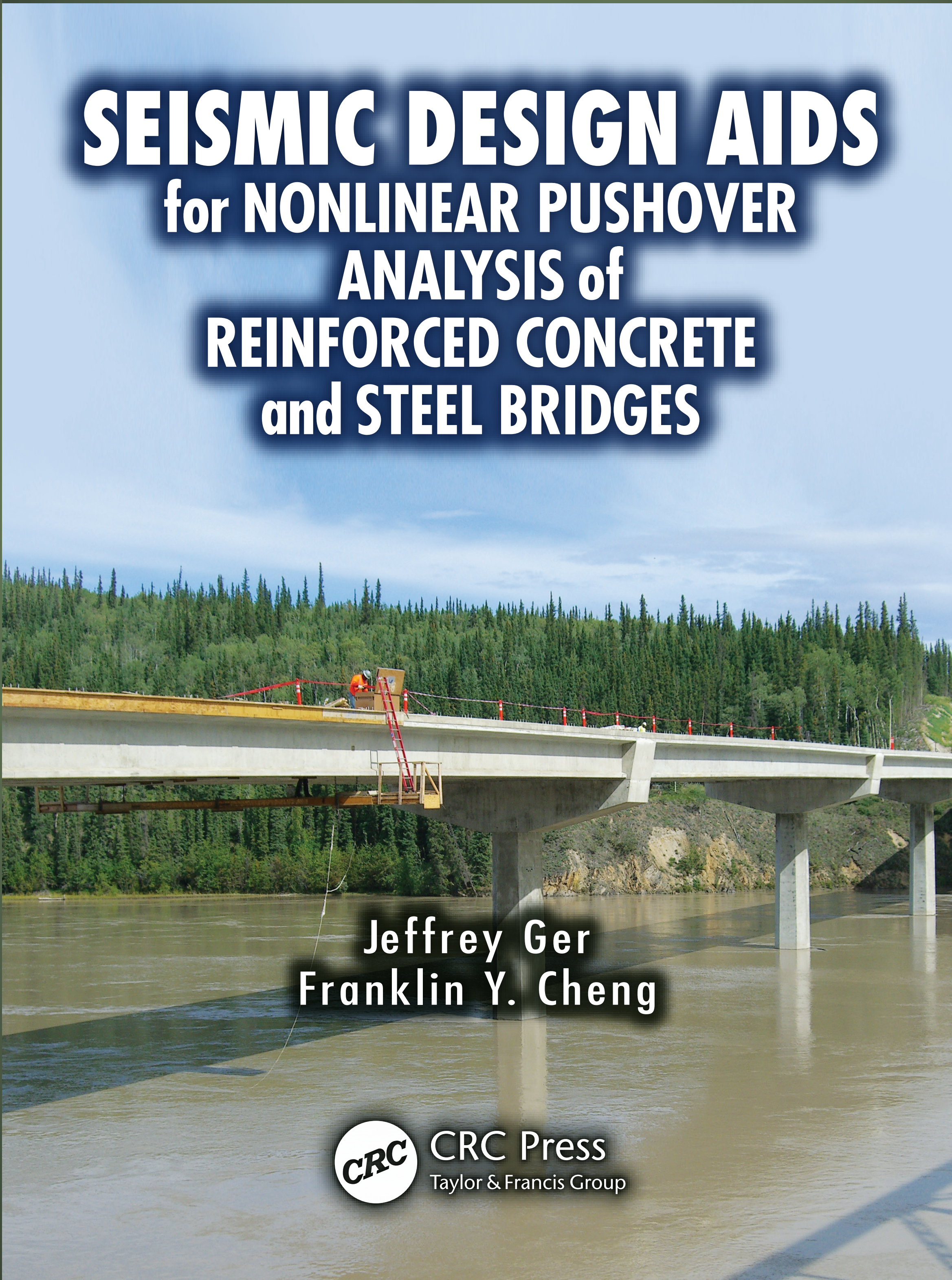
SEISMIC DESIGN AIDS for NONLINEAR PUSHOVER ANALYSIS of REINFORCED CONCRETE and STEEL BRIDGES



Jeffrey Ger
Franklin Y. Cheng

CRC Press
Taylor & Francis Group

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Series Editor: Franklin Y. Cheng

Missouri University of Science and Technology

Seismic Design Aids for Nonlinear Pushover Analysis of Reinforced Concrete and Steel Bridges

Jeffrey Ger and Franklin Y. Cheng

Seismic Design Aids for Nonlinear Analysis of Reinforced Concrete Structures

Srinivasan Chandrasekaran, Luciano Nunziante, Giorgio Serino, and Federico Carannante

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CRC Press
Taylor & Francis Group
Boca Raton London New York

CRC Press is an imprint of the
Taylor & Francis Group, an informa business

CRC Press
Taylor & Francis Group
6000 Broken Sound Parkway NW, Suite 300
Boca Raton, FL 33487-2742

Version Date: 20110608

International Standard Book Number-13: 978-1-4398-3775-7 (eBook - PDF)

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In using the computer coding given in *Seismic Design Aids*, the reader accepts and understands that no warranty is expressed or implied by the authors on the accuracy or the reliability of the programs. The examples presented are only introductory guidelines to explain the applications of proposed methodology. The reader must independently verify the results and is responsible for the results.

To our families for their immense love and support

Jeffrey Ger

Father—Chia Chian

Mother—Mei Yu

Wife—Jenny

Son—Max

Daughter—Christie

Franklin Y. Cheng

Wife—Pi Yu (Beatrice)

Son and daughter-in-law—George and Annie

Daughter and son-in-law—Deborah and Craig

Grandchildren—Alex, Camille, and Natalie

Contents

Series Preface

The new *2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design* requires pushover analysis for Seismic Design Category D bridges. The pushover analysis can identify the failure modes with collapse sequence of damaged bridges for the limit state design of the system. This is a benchmark book that provides readers with an executable file for a computer program, INSTRUCT, to serve the engineering community's needs. The book includes step-by-step numerical procedures with five different nonlinear element stiffness formulation methods that vary from the most sophisticated to the simplest and are suitable for users with varying levels of experience in nonlinear analysis. Most of the numerical examples provided with the demonstration of the accuracy of analytical prediction conformed well with the full- or large-scale test results. The key features of this book are as follows:

1. A complete handbook for pushover analysis of reinforced concrete and steel bridges with confined and nonconfined concrete column members of either circular or rectangular cross sections as well as steel members of standard shapes
2. New technology for displacement-based seismic analysis with various in-depth, nonlinear member stiffness formulations
3. Step-by-step pushover analysis procedures and applications in bridge engineering
4. A computer execute file for readers to perform pushover analysis
5. Real engineering examples with performance-based bridge design
6. Detailed figures/illustrations as well as detailed input and output descriptions

This book is a useful reference for researchers and practitioners working in the field of structural engineering. It is also a key resource for senior undergraduates and all postgraduates that provides an organized collection of nonlinear pushover analysis applications.

Preface

Nonlinear static monotonic analysis, or pushover analysis, has become a common practice for performance-based bridge seismic design. The *2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design* (AASHTO, 2009) explicitly requires pushover analysis for Seismic Design Category D bridges. The *2006 FHWA Seismic Retrofitting Manual for Highway Structures: Part I—Bridges* (FHWA, 2006) adopted pushover analysis for bridges in Seismic Retrofit Categories C and D to assess bridge seismic capacity. The popularity of pushover analysis is mainly due to its ability to identify failure modes and design limit states of bridge piers and provide the progressive collapse sequence of damaged bridges when subjected to major earthquakes. Unfortunately, there is no complete technical reference in this field to give the practical engineer step-by-step procedures for pushover analyses and various nonlinear member stiffness formulations. This book includes step-by-step procedures for pushover analysis and provides readers an executable file for a computer program, INSTRUCT (INelastic STRUCTural Analysis of Reinforced-Concrete and Steel Structures) to perform pushover analysis. The readers can download the INSTRUCT executable file from the website at http://www.crcpress.com/product/isbn/9781439837634. Many examples are provided to demonstrate the accuracy of analytical prediction by comparing numerical results with full- or large-scale test results.

The computer program INSTRUCT was developed based on a microcomputer program INRESB-3D-SUPII (Cheng et al., 1996a and b) and mainframe program INRESB_3D-SUP (Cheng and Mertz, 1989a). INRESB-3D-SUPII was a modular computer program consisting of six primary blocks. The first block (STRUCT) defines the structural model. The remaining five blocks (SOL01, SOL02, SOL03, SOL04, and SOL05) are independent solutions for static loading, seismic loading, natural frequency and buckling loading, static cyclic or pushover loading, and response spectrum analysis, respectively. Since the purpose of INSTRUCT is mainly to perform nonlinear pushover analysis of reinforced concrete and steel bridge bents, it includes only SOL01 and SOL04. During the development of INSTRUCT, SOL04 was enhanced significantly, and it includes five different nonlinear element stiffness formulation methods for pushover analysis. They are finite segment–finite string (FSFS), finite segment–moment curvature (FSMC), axial load–moment interaction (PM), constant moment ratio (CMR), and plastic hinge length (PHL) methods. These range from the most sophisticated to the simplest and are suitable for engineers with varying levels of experience in nonlinear structural analysis. The results from these methods have been compared during the development of the program. They generally exhibit reasonable differences due to the different numerical operation of individual methods, but are consistent in general. SOL04 is capable of performing not only unidirectional pushover analysis but also cyclic pushover analysis. Depending on future needs, SOL02, SOL03, and SOL05 can be incorporated into future versions of INSTRUCT.

Chapter 1 describes the evolution of seismic bridge design codes in the United States over the past 70 years and includes a comparison between force-based and displacement-based design approaches. Regardless of the design approach being used, it demonstrates the importance of using pushover analysis for seismic bridge design and retrofitting evaluation.

Chapter 2 summarizes the application of pushover analysis in force-based bridge design as well as in displacement-based seismic bridge design. Other applications such as capacity/demand analysis for the evaluation of existing bridges, quantitative bridge redundancy evaluation, moment–curvature analysis, and estimation of inelastic response demand for buildings are also described in this chapter.

Nonlinear pushover analysis procedure is described in Chapter 3. The flowchart for structural modeling and the procedures for solutions SOL01 and SOL04 are described. Material and element libraries are provided, including 12 material and 7 element types. The material library covers elastic material and hysteresis models of bilinear, Takeda, gap/restrainer, hinge, interaction axial load–moment, finite-segment (steel), finite-segment (reinforced concrete), FSMC, plate, point, and brace materials. The element library includes elastic three-dimensional (3D) beam, spring, inelastic 3D beam, finite-segment, plate, point, and brace elements.

The nonlinear bending stiffness matrix formulations for reinforced concrete members are described in Chapter 4, including the above-mentioned FSFS, FSMC, PM, CMR, and PHL methods. Since most bridge columns in the United States are reinforced concrete columns, it is necessary to check all the possible concrete column failure modes in the pushover analysis. Possible concrete column failure modes include

1. Compression failure of unconfined concrete due to fracture of transverse reinforcement
2. Compression failure of confined concrete due to fracture of transverse reinforcement
3. Compression failure due to buckling of the longitudinal reinforcement
4. Longitudinal tensile fracture of reinforcing bars
5. Low cycle fatigue of the longitudinal reinforcement
6. Failure in the lap-splice zone
7. Shear failure of the member that limits ductile behavior
8. Failure of the beam–column connection joint

INSTRUCT is capable of checking all the possible concrete column failure modes. The approaches used to check individual failure modes are also described in this chapter.

Chapter 5 describes how to combine bending, shear, axial, and torsional stiffnesses to form the 3D element stiffness matrices for bridge columns and cap beams. The stiffness matrix formulation for other elements such as brace and plate elements is introduced in this chapter. Once all the element stiffness matrices are formulated, a 3D structural system subjected to both static and nonlinear pushover loadings can be analyzed. The definitions of structural joints and degrees of freedom (dofs), including free, restrained, condensed, or constrained dofs, are also described in detail.

Chapter 6 contains detailed input data instructions. The modular form of INSTRUCT allows the addition of new materials and/or new elements into the program depending on future needs. The structural analysis adopted in the program is based on the matrix method. The system formulation in INSTRUCT has the following attributes: (1) joint-based degrees of freedom, (2) rigid body and planar constraints, (3) material and geometric stiffness matrix formulation, and (4) unbalanced load correction. INSTRUCT has been developed to achieve efficiency in both computation and data preparation. The output solutions include the results of joint forces and displacements, member forces and deformations, member ductility factors, and structural displacement capacities corresponding to different performance-based limit states.

Chapter 7 provides 13 numerical examples to illustrate the preparation of input data and the output solutions for the bridge pushover analysis of reinforced concrete and steel bridge bents. Most examples provide a comparison between the numerical results and available experimental test results. Many existing steel diaphragms (cross frames) in steel or prestressed concrete girder bridges were not designed for high seismic loads, and the inelastic buckling of brace members could occur when subjected to lateral loads. For steel pile cap bents, the steel piles may develop plastic hinges and the diagonal brace members may buckle due to lateral seismic load. As shown in some of the examples, INSTRUCT is capable of performing pushover analysis for steel pile cap bents and steel diaphragms, with consideration of post-buckling effects of steel members.

The majority of the mathematic derivations for the nonlinear stiffness matrices of various structural elements, nonlinear member cross-sectional properties, and different numerical analyses described in this book are included in Appendices A through E, I, and J. Although this book is mainly for readers who have fundamental earthquake engineering and structural dynamics background, Appendices F through H provide structural engineers with basic knowledge of dynamic analysis of structures, including elastic and inelastic time history analyses, damped free vibration, damped vibration with dynamic force, the development elastic and inelastic response spectra, equivalent viscous damping, and the response spectrum analysis of the multiple-degrees-of-freedom system.

The photo shown on the book cover is of the Tanana River Bridge near Tok, Alaska, which was one of the first bridges in Alaska designed using the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (AASHTO, 2009) and pushover analysis to ensure that the displacement capacities of individual piers are greater than the corresponding seismic displacement demands. The authors wish to thank Derek Soden, the former Alaska DOT structural designer who designed this bridge, for providing this photo cover and proofreading a majority of the book manuscripts.

Series Editor

Franklin Y. Cheng, PhD, PE, is a distinguished member (formerly honorary member) of ASCE; member of the Academy of Civil Engineers, Missouri University of Science and Technology (MST); and curators' professor emeritus of civil engineering, MST. He is one of pioneers in allying computing expertise to large, complex, seismic-resistant structures with major impact as follows: (1) development of computer algorithms and programs for supercomputers, PCs, and executable files on the web available worldwide; response results that conform well with field observations such as the 22-story Pino-Suraez steel buildings in Mexico City and River Crossing reinforced concrete bridges in California; (2) optimum design of 2D and 3D tall buildings in the United States and abroad for both practice and parametric investigations such as design logic procedures and criteria of the Tentative Provisions of ATC-3 for improvement recommendations; (3) development of a smart HDABC (hybrid damper actuator braced control) system considering soil–structure interaction effective for various earthquake magnitudes with shaking-table verification; (4) leadership in integrating frontier design and retrofitting techniques through international workshops. Dr. Cheng has numerous publications to his credit, the most recent being *Structural Optimization—Dynamic and Seismic Applications*, *Smart Structures—Innovative Systems for Seismic Response Control*, and *Matrix Analysis of Structural Dynamic—Applications and Earthquake Engineering*.

Authors

Jeffrey Ger is the Federal Highway Administration (FHWA) division bridge engineer in Florida, Puerto Rico, and U.S. Virgin Islands. Before joining FHWA, he worked with the Missouri Department of Transportation where he supervised 10 bridge designers and was extensively involved with projects for the seismic retrofitting and strengthening of bridges in St. Louis, Missouri. He received his PhD in civil engineering from the University of Missouri-Rolla. His research experience has been in the field of earthquake engineering, nonlinear structural response, and building and highway bridge design. He has published more than 40 technical papers in structural engineering. Dr. Ger is a professional engineer and is a member of FHWA's National Seismic Virtual Team. He is also the FHWA Ex-Officio for the Technical Committee T-4 (Construction) of the American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Bridges and Structures. He is a panel member of current NCHRP 12-86 and 20-07 research projects on "Bridge System Safety and Redundancy," and "Update AASHTO Guide Spec. for Bridge Temporary Works," respectively. Dr. Ger received the U.S. Secretary of Transportation's Team Award in 2004 "for providing extraordinary transportation services to move food, water and shelter materials to relieve the pain and suffering by millions of victims of the 2004 Hurricanes." He provided critical support in the wake of Florida's 2004 hurricanes, completing an emergency interstate bridge repair project 26 days ahead of schedule. In 2006, he received the FHWA Bridge Leadership Council's Excellent Award, recognizing his outstanding customer service in carrying out the bridge program in Florida. In 2007, he received the FHWA Engineer of the Year Award as well as an award from the National Society of Professional Engineers, as one of the top 10 federal engineers of the year among the 26 federal agencies. In 2008, he received the Civil Engineering Academy Award from the Department of Civil Engineering at the University of Missouri-Rolla. Recently, Dr. Ger was appointed as one of the seven members of the U.S. Transportation Infrastructure Reconnaissance Team traveling to Chile in April 2010 to assess the bridge damage condition due to the February 27, 2010 Chile earthquake.

Franklin Y. Cheng was appointed Curators' Professor of Civil Engineering at the University of Missouri-Rolla (now Missouri University of Science and Technology, MST) in 1987, the highest professorial position in Missouri's university system, and is the senior investigator, Intelligent Systems Center, University of Missouri-Rolla. Dr. Cheng has received 4 honorary professorships abroad and chaired 7 of his 24 National Science Foundation (NSF) delegations to various countries for research and development cooperation. He has served as either chairman or member of 37 professional societies and committees, 12 of which are American Society of Civil Engineers (ASCE) groups. He was the first chair of the Technical Administrative Committee on Analysis and Computation and initiated the Emerging Computing Technology Committee and Structural Control Committee. He also initiated and chaired the Stability Under Seismic Loading Task Group of the Structural Stability Research Council. Dr. Cheng has served as a consultant for Martin Marietta Energy Systems Inc., Los Alamos National Laboratory, and Martin & Huang International, among others. The author, coauthor, or editor of 26 books and over 250 publications, Dr. Cheng is the recipient of numerous honors, including the MSM-UMR Alumni Merit, ASCE State-of-the-Art (twice), the Faculty Excellence, and the Haliburtan Excellence awards. In 2007, he was elected as the 565th honorary member of ASCE since 1852. Dr. Cheng gained industrial experience with C.F. Murphy and Sargent & Lundy in Chicago, Illinois. He received his PhD (1966) in civil engineering from the University of Wisconsin-Madison.

1 Overview of Seismic Design of Highway Bridges in the United States

1.1 INTRODUCTION

The nonlinear static monotonic analysis, or pushover analysis, has become a common procedure in current structural engineering practice (ATC-40, 1996; FEMA-273, 1997; FEMA-356, 2000). The American Association of State Highway and Transportation Officials (AASHTO) Guide Specifications for load and resistance factors design (LRFD) Seismic Bridge Design explicitly require pushover analysis for seismic design category D (SDC D) bridges. The *2006 FHWA Seismic Retrofitting Manual for Highway Structures: Part I—Bridges* (FHWA, 2006) adopted pushover analysis in evaluation method D2 for bridges of seismic retrofit categories C and D (SRC C and SRC D) to assess bridge seismic performance.

This chapter describes the evolution of seismic bridge design codes in the United States. The intent is not to introduce the seismic design codes in detail, but to illustrate the differences among these codes and discuss major code improvements over the past 70 years. The history of code development can explain why the current *AASHTO Guide Specifications for LRFD Seismic Bridge Design* and the *FHWA Seismic Retrofitting Manual* require using nonlinear pushover analysis for bridge design and retrofit, respectively. This chapter also provides a discussion of possible future code improvement.

1.2 AASHTO BRIDGE SEISMIC DESIGN PHILOSOPHY

The highway bridge design code in the United States has evolved several times over the past 70 years. The first highway bridge design code was published in 1931 by the American Association of State Highway Officials (AASHO), later by the AASHTO. From 1931 through 1940, AASHO codes did not address seismic design. The 1941 edition of the AASHO code required that bridges be designed for earthquake load; however, it did not specify how to estimate that load. In 1943, the California Department of Transportation (Caltrans) developed various levels of equivalent static lateral forces for the seismic design of bridges with different foundation types, with individual members designed using the working stress design (WSD) method (Moehle et al., 1995).

Following Caltrans' criteria, the 1961 edition of the AASHO specifications for the first time specified an earthquake loading for use with the WSD design approach. This seismic provision, used until 1975, did not include a national seismic map. The AASHO design code provisions from this period are briefly described as follows.

1.2.1 AASHO Elastic Design Procedures (1961–1974)

In regions where earthquakes may be anticipated, the equivalent earthquake static lateral force was calculated (AASHO, 1969) as follows:

$$EQ = CD \tag{1.1}$$

where
- EQ is the lateral force applied horizontally at the center of gravity of the structure
- D is the dead load of structure
- $C = 0.02$ for structures founded on spread footings on material rated as 4 t or more per square foot
- $C = 0.04$ for structures founded on spread footings on material rated as less than 4 t per square foot
- $C = 0.06$ for structures founded on piles

The earthquake force, EQ, calculated from Equation 1.1 was part of the Group VII loading combination given by

$$\text{Group VII} = D + E + B + SF + EQ \tag{1.2}$$

in which D, E, B, and SF are dead load, earth pressure, buoyancy, and stream flow, respectively. With WSD, the code allowed a $33\frac{1}{3}\%$ increase in the allowable stress for member design due to earthquake consideration. For reinforced concrete columns subjected to bending, the allowable compression stress at the extreme fiber was $0.4f'_c$, and tension stress at the extreme fiber of the member was not permitted.

Despite the Caltrans design criteria, many highway bridges were severely damaged or collapsed during the 1971 San Fernando earthquake. The post-earthquake damage assessment indicated that the elastic WSD provisions for bridges subjected to earthquake were not adequate. This event illustrated the drawbacks of elastic design, such as (1) the seismic lateral force levels of 2%, 4%, and 6% of the total structural dead load were too low in California, (2) the actual column moment demand reached the column moment capacity, (3) columns were not designed for ductility, which resulted in brittle failure during the earthquake, and (4) energy dissipation was very small.

Following the San Fernando earthquake, Caltrans developed a new force-based seismic design procedure for highway bridges. The new design criteria included soil effects on seismic load and the dynamic response characteristics of bridges. It increased the amount of column transverse reinforcement for ductility, and beam seat lengths were increased to minimize the risk of unseating of the superstructure. In 1975, AASHTO adopted an interim seismic design specification, which was based on the Caltrans' design criteria. The same design criteria were used in the 1977, 1983, 1989, and 1992 AASHTO Standard Specifications. The following describes the design criteria during this time period.

1.2.2 AASHTO Force-Based Design Procedures (1975–1992)

The equivalent static force method was used to calculate the design earthquake loading. The design earthquake load is given as follows:

$$EQ = CFW \tag{1.3}$$

where

- EQ is the equivalent static horizontal force applied at the center of gravity of the structure
- F is the framing factor
- $F = 1.0$ for structures where single columns or piers resist the horizontal forces
- $F = 0.8$ for structures where continuous frames resist the horizontal forces applied along the frame
- W is the total dead weight of the structure
- C is the combined response coefficient, expressed as

$$C = A \times R \times \frac{S}{Z} \tag{1.4}$$

where

- A is the maximum expected peak ground acceleration (PGA) as shown in the seismic risk map of the United States in Figure 1.1
- R is the normalized acceleration response (PGA = 1 g) spectral value for a rock site
- S is the soil amplification factor
- Z is the force-reduction factor, which accounts for the ductility of various structural components

The first U.S. seismic map, as shown in Figure 1.1, was included in this version of the AASHTO code. Although the definitions of R, S, and Z were described in the code, the numerical values of R, S, and Z were not provided. Instead, four plots of C as a function of structural period were provided with each plot representing a certain depth range of alluvium to rocklike material. One of the combined response coefficient plots is shown in Figure 1.2. The PGA values corresponding to three seismic zones (zones 1, 2, and 3) in the seismic map are shown in Table 1.1.

The same Group VII load combination given by Equation 1.2 was used for WSD with a $33\frac{1}{3}\%$ increase in the allowable stress. From the lessons learned in the 1971 San Fernando earthquake, for the first time, AASHTO provided the option of using

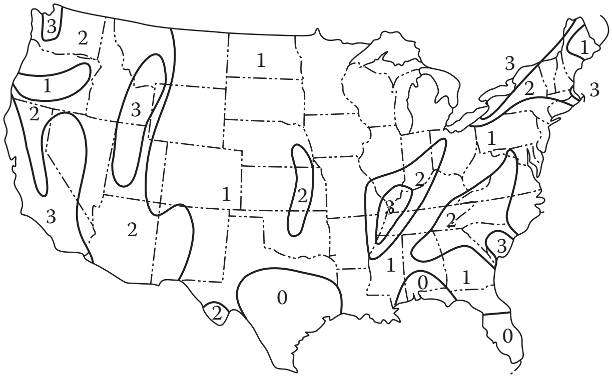

FIGURE 1.1 National seismic risk map. (From American Association of State Highway Transportation Officials (AASHTO), *Standard Specifications for Highway Bridges*, 12th edn., Washington, DC, 1977.)

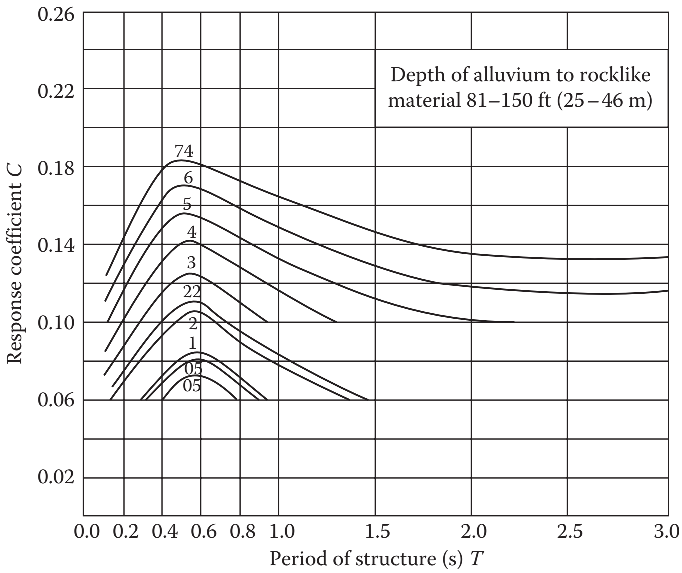

FIGURE 1.2 Combined response coefficient C for different rock acceleration A. (From American Association of State Highway Transportation Officials (AASHTO), *Standard Specifications for Highway Bridges*, 12th edn., Washington, DC, 1977.)

load factor strength design (LFD) and allowed inelastic deformations in ductile column members. For LFD, the Group VII load combination was

$$\text{Group VII} = \gamma[\beta_D D + \beta_E E + B + SF + EQ] \tag{1.5}$$

in which the load factor $\gamma = 1.3$, $\beta_D = 0.75$ for checking the column for minimum axial load and maximum moment, $\beta_D = 1.0$ for checking the column for maximum axial

TABLE 1.1
Maximum Expected PGA for Different Zones

PGA Value (g)	Zone
0.09	1
0.22	2
0.5	3

load and minimum moment, $\beta_E = 1.3$ for lateral earth pressure and 0.5 for checking positive moments in rigid frames, and B and SF are the buoyancy and stream flow pressure, respectively.

Since values for Z were not provided in the specifications, a designer did not have a clear idea what column ductility demand was required. Without knowing the ductility demand, the ductility capacity of the design column was of questionable adequacy. This drawback was improved in the 1992 AASHTO specifications as described in the next section.

1.2.3 AASHTO Force-Based Design Procedures (1992–2008)

The 1992 edition of the AASHTO Standard Specifications was based on the Applied Technology Council (ATC) publication entitled "Seismic Design Guidelines for Highway Bridges" (ATC-6, 1981). The primary departure from the previously mentioned AASHTO specification (1975–1992) is described as follows:

1. Instead of the equivalent static force method, structures were analyzed by elastic response spectrum analysis. The detailed description of response spectrum analysis is given in Appendix H.
2. The design acceleration spectrum included consideration of soil type at the bridge site, ranging from hard (S_1) to very soft (S_4).
3. The elastic member forces calculation considered two horizontal seismic components. The combination of structural responses due to multicomponent seismic input is described in Appendix H.
4. The elastic member forces from the response spectrum analysis were reduced by a response modification factor, R, which mainly represented the column ductility demand with consideration of the redundancy of the structure.
5. The specifications emphasized the ductile detailing of columns via a minimum transverse reinforcement requirement.

As mentioned above, the elastic force demand of the ductile member is divided by the code-provided response modification factor R (also called force-reduction factor or strength-reduction factor). The intent of R is to estimate the column ductility

TABLE 1.2
Response Modification Factors

Substructure	***R***
Wall-type pier	2
Reinforced concrete pile bents	
1. Vertical piles only	3
2. One or more battered piles	2
Single columns	3
Steel or composite and steel	
Concrete pile bents	
1. Vertical piles only	5
2. One or more battered piles	3
Multiple column bent	5

Source: American Association of State Highway Transportation Officials (AASHTO), *Standard Specifications for Highway Bridges*, 16th edn., Washington, DC, 1996.

demand. The response modification factors in the 1992 and 1996 editions of the AASHTO Standard Specifications are shown in Table 1.2.

Based on these specifications, the LFD Group VII load combination for seismic performance categories (SPCs) C and D was

$$\text{Group VII} = 1.0[D + E + B + SF + EQM] \tag{1.6}$$

in which

$$EQM = \frac{EQ}{R} \tag{1.7}$$

where

- EQ is the elastic seismic member force calculated from the response spectrum analysis
- EQM is the elastic seismic member force modified by the appropriate R-factor given in Table 1.2

In the response spectrum analysis, the design spectrum value corresponding to the mth mode shape is in terms of the elastic seismic response coefficient, C_{sm}, expressed by

$$C_{sm} = \frac{1.2AS}{T_m^{2/3}} \tag{1.8}$$

where

A is the acceleration coefficient from the seismic PGA map

S is the site coefficient having the values of 1.0, 1.2, 1.5, and 2.0 for soil types of S_1, S_2, S_3, and S_4 (or called soil types I, II, III, and IV), respectively

T_m is the structural period corresponding to the mth mode

Figure 1.3 shows the AASHTO 500-year return period seismic contour map, which is much more refined than the previous AASHTO map shown in Figure 1.1. The design spectrum with soil types of S_1, S_2, S_3, and S_4 is shown in Figure 1.4, which was determined from the generation of many response spectra based on many earthquake records, primarily from earthquakes in the western United States (Seed et al., 1976). A description of how to generate response spectra is given in Appendix G. The specifications defined four SPCs (A, B, C, and D) on the basis of the acceleration coefficient, A, for the site, and the importance classification (IC) of the bridge to be designed, as shown in Table 1.3, in which IC = I for essential bridges and IC = II for other bridges. An essential bridge is one that must be designed to function during and after an earthquake. The specifications provided different degrees of sophistication of seismic analysis and design for each of the four SPCs.

In 1994, AASHTO published the first edition of the AASHTO *LRFD Bridge Design Specifications*, with the second, third, and fourth editions published in 1998, 2004, and 2007, respectively. Similar to the previous 1992 and 1996 AASHTO standard specifications, the LRFD specifications account for column ductility

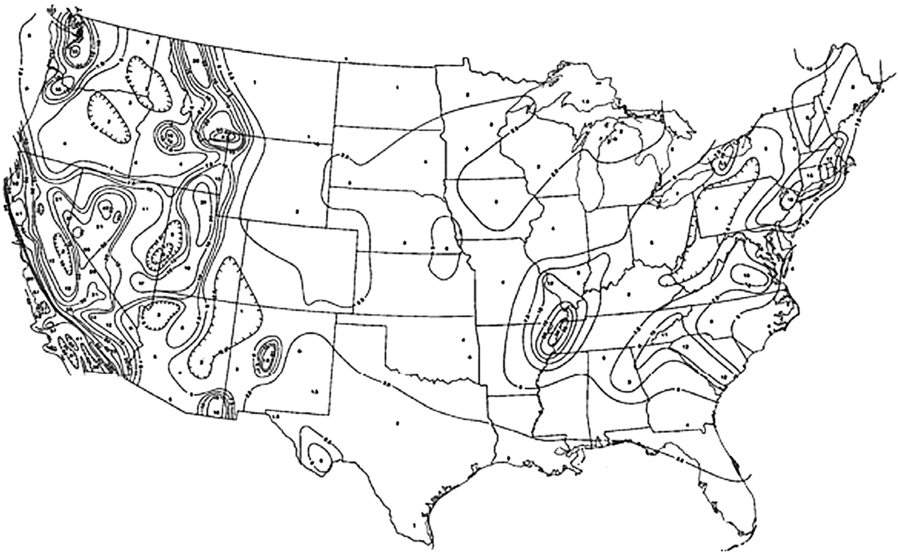

FIGURE 1.3 PGA acceleration coefficient A. (From American Association of State Highway Transportation Officials (AASHTO), *Standard Specifications for Highway Bridges*, 16th edn., Washington, DC, 1996.)

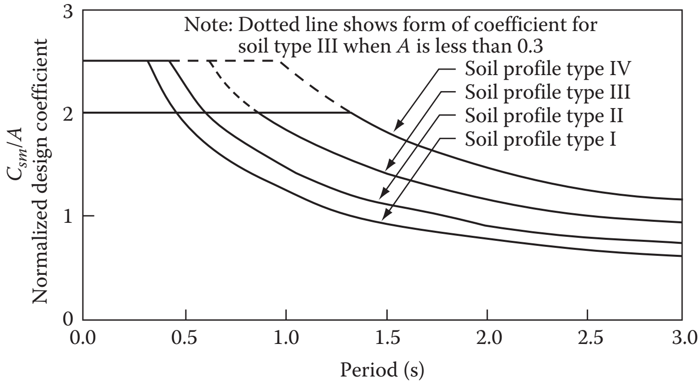

FIGURE 1.4 Normalized seismic response spectra for various soil types. (From American Association of State Highway Transportation Officials (AASHTO), *Standard Specifications for Highway Bridges*, 16th edn., Washington, DC, 1996; American Association of State Highway Transportation Officials (AASHTO), *LRFD Bridge Design Specifications*, 4th edn., Washington, DC, 2007.)

TABLE 1.3
Seismic Performance Category

Acceleration Coefficient (g)	IC	
A	I	II
$A \leq 0.09$	A	A
$0.09 < A \leq 0.19$	B	B
$0.19 < A \leq 0.29$	C	C
$0.29 < A$	D	C

using response modification *R* factors. The *R* factors in the LRFD specifications are shown in Table 1.4. The number of levels of bridge importance was increased from two levels ("essential" and "other") to three levels ("critical," "essential," and "other"). Critical bridges are those that must remain open to all traffic after the design earthquake. Essential bridges are those that should be open to emergency vehicles and for security/defense purposes immediately after the design earthquake.

Instead of using SPCs, the LRFD requires each bridge to be assigned to one of the four seismic zones in accordance with Table 1.5. Similar to the AASHTO Standard Specifications, the seismic zone reflects the different requirements for methods of analysis and bridge design details.

In LRFD design, load combinations are based on the following equation:

$$Q = \sum \eta_i \gamma_i Q_i \tag{1.9}$$

TABLE 1.4
Response Modification Factors

Substructure	IC Critical	IC Essential	IC Other
Wall-type piers, larger dimension	1.5	1.5	2.0
Reinforced concrete pile bents			
1. Vertical piles only	1.5	2.0	3.0
2. With batter piles	1.5	1.5	2.0
Single columns	1.5	2.0	3.0
Steel or composite steel and concrete pile bents			
Vertical piles only	1.5	3.5	5.0
With batter piles	1.5	2.0	3.0
Multiple column bents	1.5	3.5	5.0

Source: American Association of State Highway Transportation Officials (AASHTO), *LRFD Bridge Design Specifications*, 4th edn., Washington, DC, 2007.

TABLE 1.5
Seismic Zones

Acceleration Coefficient (g)	Seismic Zone
$A \leq 0.09$	1
$0.09 < A \leq 0.19$	2
$0.19 < A \leq 0.29$	3
$0.29 < A$	4

where

Q_i is the force effect from loading type i

γ_i is the load factor for load Q_i

η_i is the load modifier relating to ductility, redundancy, and operational importance for load Q_i

In most cases, the value of each η_i is between 0.95 and 1.05, though normally, a constant η is used for all force effects, Q_i. The load combination including earthquake load is considered as the "EXTREME EVENT I" limit state in the code, given by

$$Q = \eta[\gamma_{DC} DC + \gamma_{DW} DW + \gamma_{EQ} LL + WA + FR + EQM] \quad (1.10)$$

where

- DC is the dead load of structural components
- DW is the dead load of wearing surfaces and utilities
- LL is the vehicular live load
- WA is the water load
- FR is the friction load
- EQM is the elastic seismic member force, EQ, modified by the appropriate R-factor given in Table 1.4

The elastic seismic member force, EQ, is calculated via response spectrum analysis. The design spectrum value, C_{sm}, corresponding to the mth mode shape is expressed by Equation 1.8. Essentially, the same design spectrum shown in Figure 1.4 was used in the 1994–2007 LRFD specifications.

The 2008 AASHTO LRFD interim bridge design specifications use the same R factors shown in Table 1.4. However, they incorporate some major changes to the calculation of the elastic force demand, including (1) three 1000-year USGS seismic maps (PGA, 0.2 and 1.0 s) are provided in the interim specifications (Frankel et al., 1996) and (2) more realistic site effects are incorporated into the design acceleration spectrum. The revised site effects are the result of studies carried out following the 1989 Loma Prieta earthquake in California, which culminated in recommendations that have also been adopted by the Uniform Building Code (ICBO, 1997), NEHRP Building Provisions (BSSC, 1998), and the International Building Code (ICC, 2000).

The design response spectrum in the 2008 interim specifications as shown in Figure 1.5 is constructed using accelerations taken from three seismic maps mentioned above. The design earthquake response spectral acceleration coefficients, A_S, S_{DS} (the short period 0.2 s), and S_{D1} (the 1 s period acceleration coefficient) are determined using Equations 1.11 through 1.13, respectively:

$$A_S = F_{pga} PGA \tag{1.11}$$

$$S_{DS} = F_a S_S \tag{1.12}$$

$$S_{D1} = F_v S_1 \tag{1.13}$$

where

- PGA is the peak horizontal ground acceleration coefficient from the PGA seismic map
- F_{pga} is the site factor corresponding to the PGA coefficient
- S_S = 0.2 s period spectral acceleration coefficient from 0.2 s seismic map
- F_a is the site factor for S_S
- S_1 = 1.0 s period spectral acceleration coefficient from 1.0 s seismic map
- F_v is the site factor for S_1

The value of S_{D1} is used to determine the seismic zone level, as shown in Table 1.6.

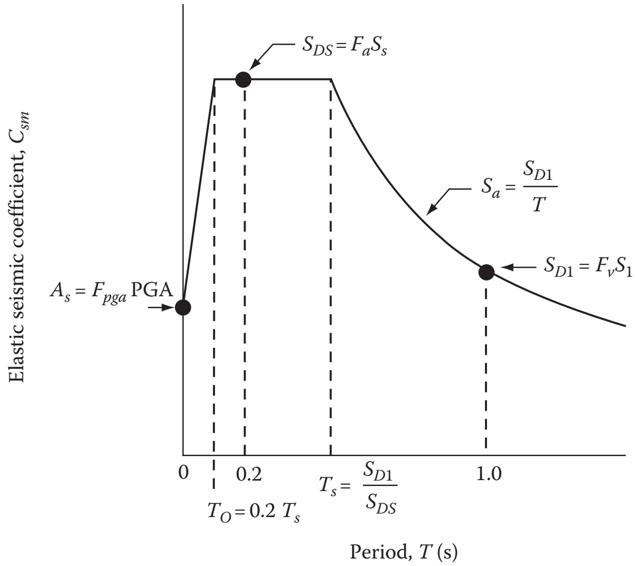

FIGURE 1.5 Design response spectrum.

TABLE 1.6
Seismic Zones

Acceleration Coefficient, $S_{D1} = F_v S_1$	Seismic Zone
$S_{D1} \leq 0.15$	1
$0.15 < S_{D1} \leq 0.30$	2
$0.30 < S_{D1} \leq 0.50$	3
$0.50 < S_{D1}$	4

The code recognizes that a well-designed structure should have enough ductility to be able to deform inelastically to the deformations imposed by the earthquake without loss of the post-yield strength. R-factors are used in the code to estimate the inelastic deformation demands on the resisting members when a bridge is subjected to the design earthquake.

The concept of R-factor is based on the equal-displacement approximation, as illustrated in Figure 1.6.

The equal-displacement approximation assumes that the maximum seismic displacement of an elastic system is the same as (or very close to) that of an inelastic system when subjected to the same design earthquake. Figure 1.6 shows two structures with the same lateral stiffness, K_e, but with different lateral yield strengths, F_{Y1} and F_{Y2}. Based on the equal-displacement approximation, the inelastic deformation,

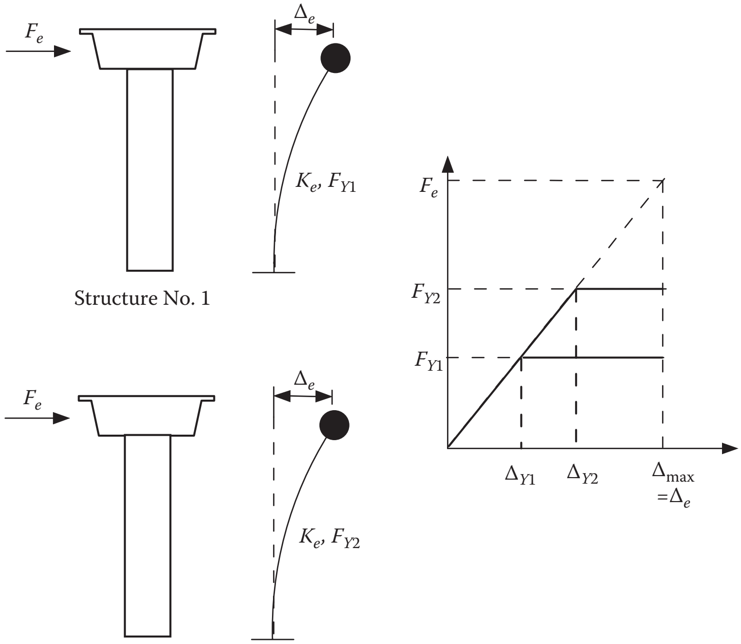

FIGURE 1.6 *R*-factor based on equal-displacement approximation.

Δ_{max}, is equal to the elastic deformation from the elastic lateral force, F_e. Therefore, the ductility demands of structures 1 and 2 can be expressed as follows:

$$\mu_1 = \frac{\Delta_{max}}{\Delta_{Y1}} = \frac{F_e}{F_{Y1}} = R_1 \tag{1.14}$$

and

$$\mu_2 = \frac{\Delta_{max}}{\Delta_{Y2}} = \frac{F_e}{F_{Y2}} = R_2 \tag{1.15}$$

From Equations 1.14 and 1.15, the force-reduction factor R represents the ratio of the elastic strength demand to the inelastic strength demand. Based on the equal-displacement approximation, the force-reduction factors R_1 and R_2 also represent the member ductility demands μ_1 and μ_2, respectively. Sound seismic design dictates that a structure should be designed for the ductility capacity greater than the seismic-induced ductility demand. However, the code-specified R-factor has its drawbacks, which will be discussed in the following section.

1.2.3.1 Force-Reduction *R*-Factor

The problems with the force-reduction factor are described as follows:

1. *Period independence*: As described in the previous section, AASHTO force-based design specifications define constant *R*-factors for different substructure types, independent of the period of the structure. In fact, the *R*-factor is a function of the period of vibration, T, of the structure, the structural damping, the hysteretic behavior of the structure, soil conditions at the site, and the level of inelastic deformation (i.e., ductility demand). Figure 1.7 shows the mean force-reduction factor spectrum for a single-degree-of-freedom system, using a large number of ground acceleration time histories recorded on rock and on alluvium. The force-reduction factor spectrum represents the ratio of the elastic strength demand to the inelastic strength demand corresponding to a specific ductility demand for a range of periods of vibration. From Figure 1.7, it can be seen that the *R*-factor is period dependent. It demonstrates that soil conditions at the site can have a significant effect on the *R*-factor, particularly in very soft soil (Miranda and Bertero, 1994), and it also shows that the ductility demand is larger than the force-reduction factor for short-period structures, and the equal-displacement approximation is not appropriate. The method of developing the force-reduction factor spectrum is described in Appendix G.
2. *Constant member initial stiffness*: As shown in Figure 1.6, in the *R*-factor methodology, the ductility demand of a structural member is estimated by the equal-displacement assumption, which assumes a constant initial stiffness, K_e. Using this approach, it is assumed that the member's initial stiffness is independent of the member's strength, when, in reality, the opposite is the case. To demonstrate this, Figure 1.8 shows the moment–curvature relationship of a concrete column with cross section diameter of 48 in., subjected to different axial loads. INSTRUCT was used for the moment–curvature

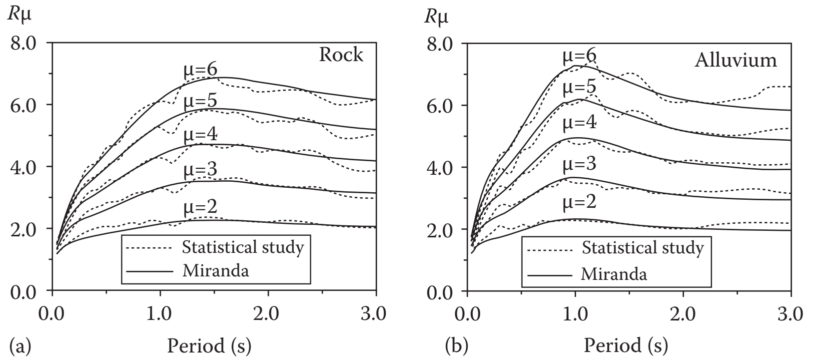

FIGURE 1.7 Mean force-reduction factors for (a) rock and (b) alluvium. (From Miranda, E. and Bertero, V., *Earthquake Spectra*, 10(2), 357, 1994.)

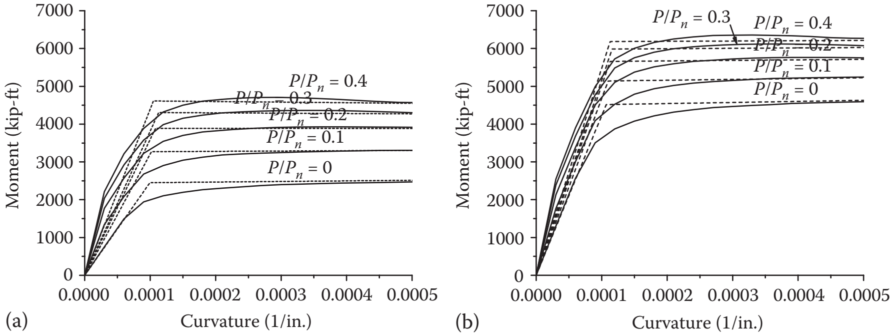

FIGURE 1.8 Moment–curvature curves of a 48″ circular column: (a) reinforcement ratio = 1.4% and (b) reinforcement ratio = 2.8%.

analysis. Two longitudinal reinforcement ratios of 1.4% and 2.8% are considered in the analysis The concrete compression strength, f_c', is 4 ksi; steel yield stress, f_y, is 60 ksi, concrete cover is 2.6″ transverse reinforcement is No. 5 spirals with 3.25″ pitch; and the steel post-yield stress–strain slope is 1% of the elastic modulus. For each longitudinal steel ratio, the axial load ratios, defined as the ratio of column axial load, P, to the column axial compression nominal strength, $P_n = f_c' A_g$, of 0, 0.1, 0.2, 0.3, and 0.4 are considered in the analysis. The simplified bilinear moment–curvature ($M-\phi$) curves are also plotted in the figure. The initial stiffness of the bilinear $M-\phi$ curve represents the cracked section flexural rigidity of the concrete member at which the first longitudinal steel reinforcement yield occurs. For bilinear $M-\phi$ curve, the point at which the line with initial stiffness intersects the line with post-yield stiffness defines the location of nominal moment M_n and nominal curvature ϕ_n. Figure 1.8 clearly indicates that the initial stiffness of the member is not a constant and is a function of the moment capacity. Figure 1.8 also shows that the nominal curvatures of the bilinear $M-\phi$ curves do not vary very much between the curves, where nominal curvature is about 0.0001 for this example. The moment capacity is strongly influenced by the axial load ratio and the amount of longitudinal reinforcement.

From the above discussion, Figure 1.9 compares the equal-displacement approximation with the more realistic condition of the reinforced concrete $M-\phi$ bilinear relationship (Priestley et al., 2007). It can be seen that the equal-displacement approximation correlates the strength poorly with the ductility demand (i.e., R-factor approach), due to the assumption that the nominal curvature will increase in proportion to the strength increase. In fact, the nominal curvature, ϕ_n, is independent of the strength (see Figure 1.9b) and is instead dependent on the column diameter and the yield strain, ε_y, of the longitudinal reinforcement. The column nominal curvature can be estimated by (Priestley et al., 1996)

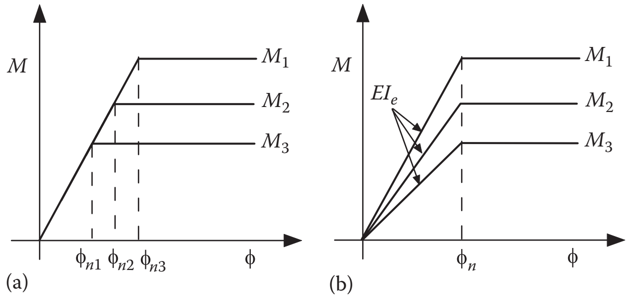

FIGURE 1.9 Moment–curvature relationship: (a) equal-displacement assumption and (b) realistic model.

$$\phi_n = \frac{2.45\varepsilon_y}{D} \quad \text{for circular concrete columns} \tag{1.16}$$

$$\phi_n = \frac{2.14\varepsilon_y}{h_c} \quad \text{for rectangular concrete columns} \tag{1.17}$$

where h_c = cross section depth. Figure 1.9b also shows that the initial bending stiffness, EI_e, increases as the strength increases.

3. *The use of elastic mode shapes to predict inelastic demand*: As mentioned previously, force-based design codes use the member stiffness at yield (i.e., cracked section stiffness for ductile members) in the elastic response spectrum analysis based on the code-provided design acceleration spectrum. However, this does not take into account the member inelastic stiffness distribution at the maximum inelastic response. For ductile structures, the inelastic mode shapes may be quite different from the elastic mode shapes used in the current design codes.
4. *Difficulty in predicting the bridge performance under strong ground motion:* As described above, the ductility demand of a ductile member cannot be accurately predicted, and, as such, the performance level of a bridge subjected to the design earthquake may not be achieved.

1.2.3.2 Capacity Design Concept

Normally, the strong beam–weak column design philosophy is used for bridge seismic design. In this strategy, plastic hinges are expected to occur in the columns but not in the beams or foundations. Whether or not a column can withstand a high ductility demand is dependent on the reinforcement details within and adjacent to the column plastic hinge zones. Columns with confined cores and sufficiently anchored reinforcement have been proven to have the necessary ductility capacity. Neither the AASHTO force-based standard specifications nor the LRFD design specifications provide detailed design criteria for estimating the ductility capacity of column subjected to the design earthquake. However, both specifications do require designers