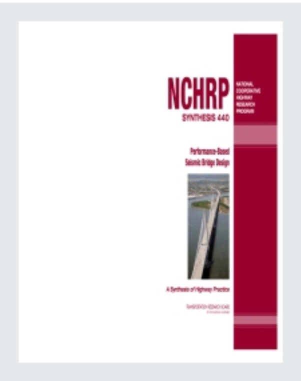

Performance-Based Seismic Bridge Design

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP SYNTHESIS 440

Performance-Based Seismic Bridge Design

A Synthesis of Highway Practice

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FOREWORD

Highway administrators, engineers, and researchers often face problems for which information already exists, either in documented form or as undocumented experience and practice. This information may be fragmented, scattered, and unevaluated. As a consequence, full knowledge of what has been learned about a problem may not be brought to bear on its solution. Costly research findings may go unused, valuable experience may be overlooked, and due consideration may not be given to recommended practices for solving or alleviating the problem.

There is information on nearly every subject of concern to highway administrators and engineers. Much of it derives from research or from the work of practitioners faced with problems in their day-to-day work. To provide a systematic means for assembling and evaluating such useful information and to make it available to the entire highway community, the American Association of State Highway and Transportation Officials—through the mechanism of the National Cooperative Highway Research Program—authorized the Transportation Research Board to undertake a continuing study. This study, NCHRP Project 20-5, "Synthesis of Information Related to Highway Problems," searches out and synthesizes useful knowledge from all available sources and prepares concise, documented reports on specific topics. Reports from this endeavor constitute an NCHRP report series, *Synthesis of Highway Practice*.

This synthesis series reports on current knowledge and practice, in a compact format, without the detailed directions usually found in handbooks or design manuals. Each report in the series provides a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems.

PREFACE

By Jon M. Williams
Program Director
Transportation Research Board

Performance-based seismic design (PBSD) for bridges is a design process that links decision making for facility design rationally and scientifically with seismic input, facility response, and potential facility damage. The goal of PBSD is to provide decision makers and stakeholders with data that will enable them to allocate resources for construction based on levels of desired seismic performance. PBSD is an advance over current prescriptive bridge design methodologies. This report summarizes the current state of knowledge and practice for PBSD.

Information for this study was acquired through literature review and a survey of state departments of transportation.

M. Lee Marsh and Stuart J. Stringer, Berger/ABAM, Federal Way, Washington State, collected and synthesized the information and wrote the report. The members of the topic panel are acknowledged on the preceding page. This synthesis is an immediately useful document that records the practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As progress in research and practice continues, new knowledge will be added to that now at hand.

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Note: Many of the photographs, figures, and tables in this report have been converted from color to grayscale for printing. The electronic version of the report (posted on the web at www.trb.org) retains the color versions.

PERFORMANCE-BASED SEISMIC BRIDGE DESIGN

SUMMARY

Currently, bridge seismic design specifications in the United States are based on prescriptive design methodologies that only marginally relate important design parameters to the performance of a bridge during an earthquake. With the current specifications, the designer does not directly control the seismic performance of bridges. This methodology has served the bridge community reasonably well, but techniques are being refined that would permit the designer, with appropriate owner input, to select and instill desired seismic performance into new bridges, and to some extent into retrofitted bridges. This new methodology is called performance-based seismic design (PBSD), and although it includes some features of the current design approaches it extends those features to a level at which designers and owners can make informed decisions about seismic performance. Such features include the ability to consider different earthquake inputs, or seismic hazard levels, and different operational classifications, such as bridges that have designated functions required after an earthquake. These functions could include postearthquake access for emergency responders or immediate availability to all traffic in order not to disrupt the regional economy. For these reasons, PBSD shows substantial promise in helping designers and owners build bridges whose performance in earthquakes is better understood and better quantified.

PBSD has been developed to such a level that it has been deployed on a limited number of large projects, and some departments of transportation have even developed approaches to apply PBSD to ordinary bridges. As this technology is promulgated, a clear and consistent approach will be crucial. This means that easy-to-use tools should be developed for relating typical engineering demand parameters (EDPs), such as displacements, force, and strains, to potential damage and to the risks arising from such damage. Damage might be in the form of concrete spalling, steel fractures, or permanent displacements. Damage can then be related to the direct risks of loss of use, loss of life, substantial repair costs, and downtime, in addition to the indirect risk of economic loss to the region.

As a profession, bridge engineers can relate earthquake loading to structural parameters, such as EDPs, through well-defined seismic hazard and structural analyses. The correlation of structural behavior to damage and then loss is less well understood, although various industries—bridges, buildings, and waterfront/marine—are working to develop loss calculation tools. A significant amount of relevant research and performance-based specifications from other practice areas was reviewed in this synthesis, and, although much work is incomplete, the profession is definitely moving toward fully probabilistic PBSD. Ultimately, the PBSD method may be able to address uncertainties in loading and resistance and to relate damage likelihoods, which can then be related to losses, including the randomness of the processes and uncertainties in our knowledge. This goal may take considerable time and effort to achieve.

In the near term, however, PBSD can be implemented on a deterministic basis with a design that includes multiple seismic hazard levels and targets specific performance levels. A guide specification or other nonmandatory guideline document could provide a consis-

tent basis for engineers to use for projects where PBSD makes sense. Typically, these would be large and important projects.

A survey of all 50 states was undertaken as part of this synthesis project, with 41 states responding (82%). Of the states that have regions in the higher seismic zones (34 states), 31 states responded (91%). It is clear that some states are using elements of PBSD, and, in reviewing various project-specific documents, there are some variations in EDP limits and expected damage between agencies. It is also clear that tools are needed to help frame the questions and produce the answers that policymakers will need when deciding seismic performance for future projects. Thus, developing a document that clearly defines terms and helps users to consistently apply performance-based concepts would be a beneficial first step in implementing PBSD.

In the longer term, research and feedback from initial implementations of PBSD will likely fill in much of the data needed to implement fully probabilistic PBSD. In the building industry, significant effort is underway to accomplish this point, but years will be needed to achieve the current goals. The bridge industry has done less work in this area, but the work will nonetheless be required. However, PBSD can help owners decide what performance they want and what modern seismic design can achieve.

CHAPTER ONE

BACKGROUND, OBJECTIVES, AND RESEARCH APPROACH

STATEMENT OF PROBLEM

Immediately following the 1971 San Fernando earthquake in southern California—a particularly damaging earthquake for bridges—the Applied Technology Council (ATC) began to develop a rational seismic design procedure for bridges in the United States. Simultaneously, the California Department of Transportation (Caltrans) developed an improved method for designing bridges, which augmented the existing AASHTO design specifications. The two efforts culminated in a prescriptive force-based design method that was eventually adopted nationally following the 1989 Loma Prieta earthquake in northern California. Although this force-based method is relatively simple to apply and reasonably effective for new design, it became evident that more quantitative methods of ensuring adequate performance of bridges were needed. Caltrans subsequently adopted a displacement-based procedure for seismic design, which has as its basis a more rational assessment of displacement demand relative to displacement capacity. This method linked detailing (e.g., reinforcement layout, section configuration) more directly with deformation capacity than the force-based method, which is based on prescriptive measures that provide assumed, but not directly checked, adequate behavior.

Taking the displacement-based procedure to the next logical level, expected performance may be quantified by linking component actions and deformations (e.g., rotations, strains) first to damage states and then to the likely postearthquake functionality of a bridge. This linking of engineering parameters to postearthquake performance has opened the door to more meaningful evaluations of a facility's potential use following an earthquake. Simultaneously, owners have begun to demand more knowledge of infrastructure risk, at least from the perspective of understanding likely seismic performance. Not long ago, it was an accepted maxim that it was not economical to design bridges to remain undamaged in large earthquakes. Today we are beginning to ask, "What exactly does it cost to design a bridge to deliver a limited amount of damage in a given earthquake?" The profession is beginning to be able to link performance, cost, and engineering metrics into a meaningful whole.

It is therefore widely believed that the next logical step in the development of seismic design of bridges is to adopt performance-based seismic design (PBSD) in lieu of the current force-based prescriptive design procedures. The vision is that an owner, advised by a knowledgeable designer, would be able to establish desired performance outcomes for bridges that are subject to earthquake loading. This would involve establishing performance levels in specific design earthquakes, which combined would comprise performance objectives. For instance, minor cracking, no bearing damage, no superstructure damage, and small permanent displacements in the substructure resulting from an earthquake with a 1,000-year return period might comprise a performance objective of "operational" for such an earthquake.

PBSD is becoming more prominent for two reasons:

1. The engineering design and research communities have developed new knowledge and tools related to seismic performance, opening the door to improved design.

2. Public expectations of bridge seismic performance may not be in line with target goals of the seismic design specifications, thus providing an opportunity for PBSD to improve the relation between expectations and target goals.

This synthesis reviews the current state of practice and knowledge for bridge PBSD.

DEFINITION OF PERFORMANCE-BASED SEISMIC DESIGN

What Is Performance-Based Seismic Design?

PBSD, or performance-based earthquake engineering as some have named it (Krawinkler and Miranda 2004; Moehle and Deierlein 2004), is a design process that attempts to link decision making for facility design rationally and scientifically with seismic input, facility response, and potential facility damage. The goal of PBSD is to provide decision makers and stakeholders with data that will enable them to allocate resources for construction based on levels of desired seismic performance. In such a system, performance is expressed in terms of facility loss or facility availability following an earthquake.

The Federal Emergency Management Agency's *FEMA 445 Next-Generation Performance-Based Seismic Design Guidelines: Program Plan for New and Existing Buildings* (2006) describes the PBSD process as follows.

> Performance-based seismic design explicitly evaluates how a building is likely to perform, given the potential hazard it is likely to experience, considering uncertainties inherent in the quantification of potential hazard and uncertainties in assessment of the actual building response. It permits design of new buildings or upgrade of existing buildings with a realistic understanding of the risk of casualties, occupancy interruption, and economic loss that may occur as a result of future earthquakes.
>
> It also establishes a vocabulary that facilitates meaningful discussion between stakeholders and design professionals on the development and selection of design options. It provides a framework for determining what level of safety and what level of property protection, at what cost, are acceptable to building owners, tenants, lenders, insurers, regulators and other decision makers based upon the specific needs of a project.

PBSD, when implemented at the highest level, should be comprehensive in consideration of outcomes and uncertainties from seismic loading and thus would be probabilistically based, providing holistic tools for designers and decision makers. However, at a practical level, the process may be significantly truncated in order to accomplish limited goals with the currently limited data and analytical tools. This synthesis attempts to summarize the current state of practice of bridge PBSD and to lay out a preliminary road map to a comprehensive process that may someday provide the rational and scientific tools the profession is currently seeking.

Figure 1 presents a visualization of the PBSD process, which is adapted to bridges from a figure that Moehle and Deierlein (2004) present in their description of a framework for performance-based earthquake engineering of buildings and that the authors credit to William T. Holmes of Rutherford and Chekene. The figure illustrates a simple pushover curve (base shear versus displacement) for a bridge. The primary feature of the figure shown here is the juxtaposition of several elements:

- Conceptual bridge damage states in the sketches above the curve
- Performance levels (as further described in chapter six of this synthesis)
  - Fully Operational
  - Operational

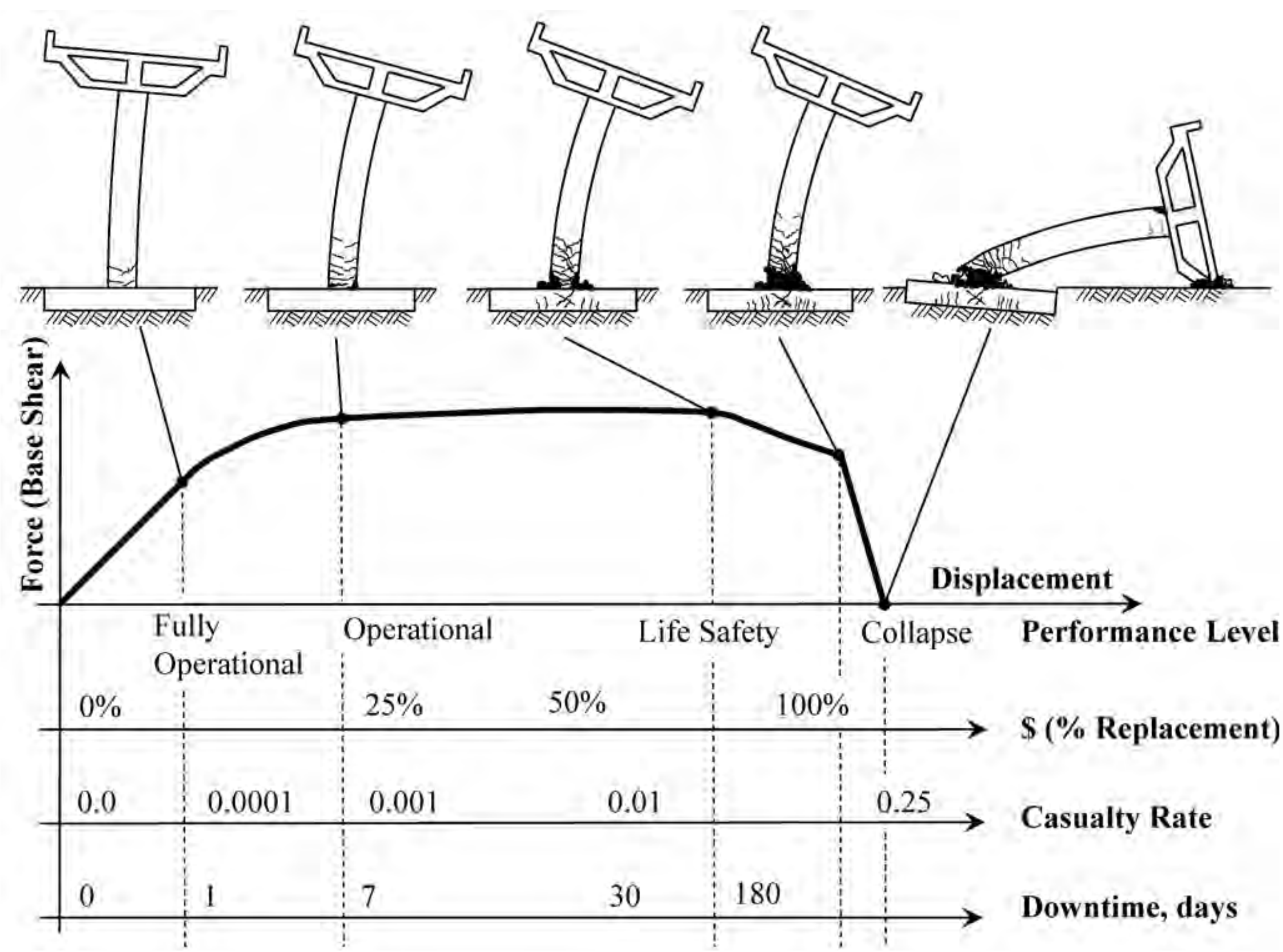


FIGURE 1 Visualization of PBSD (after Moehle and Deierlein 2004).

  - Life Safety
  - Collapse
- List of damage repair costs related to replacement cost
- Potential casualty rate for the bridge
- Estimate of loss of use of the bridge

This visualization is powerful because it represents the capacity side of seismic structural response in structural performance and potential outcome terms that decision makers could use to evaluate the success of the design when loaded to various levels along the pushover curve. This simple graphic summarizes much of what PBSD attempts to provide. If this graphic were combined with seismic input for the site, then the entire PBSD method would be illustrated in one figure. (Note that the replacement costs, casualty rates, and downtime values in Figure 1 are provided solely as examples and do not represent actual figures.)

Indeed, the PBSD process may be broken into four steps, three of which are included in Figure 1. These steps were conceived to guide the work of the Pacific Earthquake Engineering Research Center (PEER), as outlined by Moehle and Deierlein:

1. Seismic hazard analysis that quantifies the seismic input at the site in terms of intensity measures (IM), such as spectral acceleration (S_A).

2. Structural analysis that relates the seismic input to structural response that is related by engineering demand parameters (EDPs), such as strains, rotations, displacements, drifts, or internal forces.

3. Damage analysis that relates the structural response to damage measures (DMs), which describe the condition of the structure, such as the occupancy or use definitions: Fully Operational, Operational, Life Safety, and Collapse.

4. Loss analysis that relates damage to some type of decision variable (DV), such as the repair costs, casualty rate, or downtime, as shown in the figure. Of course, availability, in lieu of loss, could be used for loss analyses.

When these four steps are considered in the context of current design practice, it is evident that the first two steps are routinely performed. The third step is usually not considered directly, although it is inherent in the design specifications because preservation of life safety is the underlying principle of the codes. If the design code requirements are followed, then life safety will be preserved. This was the primary reason for the original development of design codes, whether driven by safety in the face of fire or safety from collapse. It is at this third step that our current design methodologies begin to wane with respect to PBSD, and it is important to recognize that the designer does not make choices about performance. Instead, he or she simply complies with the code requirements and, therefore, tacitly assumes that life safety will be ensured. In such cases, the code, not the designer or the owner, controls the performance.

PBSD therefore seeks to go beyond the current level of rigor required by the design codes by having the designer and owner decide what performance is targeted from the structure under earthquake loading. Here single- or multilevel seismic input may be considered, depending on the desired performance that is sought at various levels or intensities of strong ground shaking. However, fundamental to all designs is that life safety must be preserved in some preselected level of earthquake shaking. Beyond that minimum, the design may be enhanced to ensure the range of structural performance desired. Such enhancements would be selected based on data provided to decision makers who determine resource allocation based on the facility's postearthquake functional requirements.

For bridges, the long-held notion of preservation of life safety for a predetermined earthquake input has served communities fairly well. However, bridges can be important lifelines for communities where life safety of people who are not physically on the bridge at the time of an earthquake may be at stake. Thus, the bridge may have a postearthquake role in serving the community by providing emergency vehicle access. Such a role would suggest a higher performance objective than the basic levels included in the design codes.

It is important to recognize that the design codes, beginning in the wake of the 1971 San Fernando earthquake, began to introduce Importance Factors, which sought to provide enhancements above basic life safety for people on the structure at the time of the earthquake by increasing the earthquake design forces. However, the force-based design methodologies used for both buildings and bridges were unable to rationally deliver the desired performance that is now sought with PBSD. With the emergence of displacement-based design procedures, true PBSD is more likely to be achievable.

To deliver on the promise of PBSD, all four steps of the approach must be completed. This most likely will occur through development of design methodologies that permit the process to first be completed in deterministic fashion—without full consideration of the uncertainties that exist at each step of the process. Deterministic analysis is currently in use where specific strain or displacement limits are adhered to, as with the current AASHTO *Guide Specifications for LRFD Seismic Bridge Design* (AASHTO SGS). However, at some future point it should be possible to employ a probabilistic approach through all four PBSD steps.

The PBSD process has been conceptualized in full probabilistic form by PEER for both buildings and bridges. Many

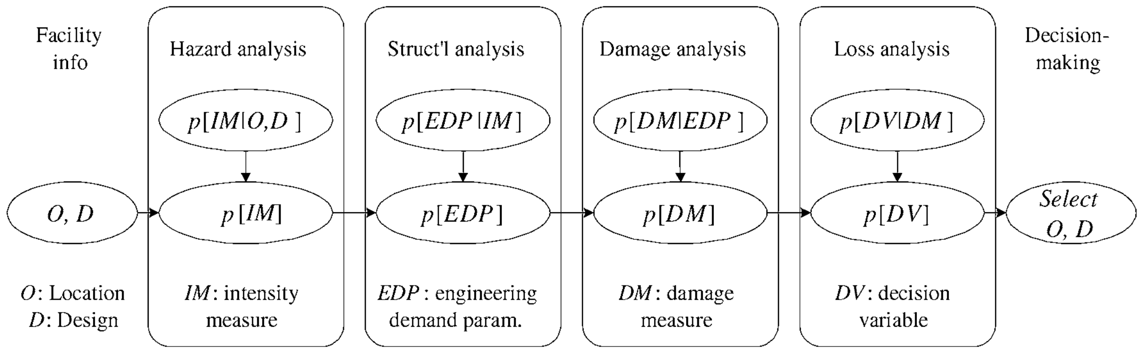


FIGURE 2 Underlying probabilistic framework of PBSD (Moehle and Deierlein 2004).

missing pieces of knowledge and data must be addressed before such a design process is ready for deployment into practice; however, many researchers are striving to fill in those gaps. Their efforts are discussed throughout this report.

In its full probabilistic form, the four-step design process may be summarized as in Figure 2. The figure clearly delineates the four steps, and the important measures and variables are as defined earlier for the overall concept. The measures are given as probabilities: p[IM], p[EDP], and so on. These probabilities depend on conditional probabilities, for example p[EDP|IM], which is read as “the probability of reaching an EDP given a value of IM.” Thus, to find the probability of annual exceedance of an EDP, p[EDP], one must combine the conditional probability of the EDP, given an IM with the annual probability of exceedance of the IM. Accordingly, this process is built up by successive combination considering the site location (O) and structure design features (D) to yield a DV that can be used to evaluate the adequacy of the site design. Subsequent chapters will address each major component: hazard analysis in chapter four, structural analysis and design in chapter five, damage analysis in chapter six, and loss analysis in chapter seven.

It should be apparent to users of the current design provisions of either buildings or bridges that the profession is able to relate the seismic input probabilistically, but not the remaining three steps. For example, we currently use spectral accelerations that have a preset percent chance of exceedance in a given window of time (e.g., accelerations that have 7% chance of exceedance in 75 years for bridge design). Completing all four steps in a fully probabilistic fashion will take more effort and many refinements and additions to the current design methodologies.

STUDY OBJECTIVES

This synthesis project gathers data from a number of different but related areas. The current status of bridge seismic design is briefly summarized and includes methodologies for smaller bridges and those that have been used for larger, more important structures designed with enhanced performance objectives. The state of knowledge of large-scale laboratory performance, as well as actual bridge performance in earthquakes, is also summarized. Then, the links between measurable behavior in the lab or field and inferred performance are explored, including a limited review of analytical techniques. From this review a status of the profession today, with respect to the technical challenges of PBSD, has been developed. The intent is that this document will feed the next challenge—deciding how to employ PBSD.

It is recognized that challenges beyond the technical face the implementation of PBSD. Tools for decision makers need to be developed such that engineers can provide alternatives and costs to allow informed transportation administrators to make decisions regarding the use of enhanced performance. An obvious use of PBSD would be to support the design of corridors or specific bridges that have distinct postearthquake operability requirements. This implies that only some bridges might be designed using PBSD, particularly in the near future. PBSD could be used to augment the current life-safety minimum standard and provide enhanced performance only in selected cases. In fact, several agencies have used this approach in the past, and part of the goal of this synthesis project is to document such project-specific criteria for ready access by other agencies considering enhanced seismic performance for bridges.

Key to this documentation are the decision-making elements that feed the ultimate selection of PBSD for use on a project. Such decisions are best made with informed data of the risk posed to a facility by seismic activity. Therefore, a logical basis for evaluating risk is a probabilistic one. Work is being performed in this area, and eventually PBSD may be probabilistically based. Currently, however, a simpler deterministic basis may be the first logical step. Because PBSD departs from the traditional approach of using standards based on the minimum threshold of life safety, an optional transition to PBSD design may be the way forward, particularly until experience with both design and construction costs is developed.

The information gathered for this project includes the following.

- Potential benefits that an owner might realize by using a performance-based seismic design to achieve enhanced performance over that available with the current design procedures. In other words, why transition to PBSD and where does it make the most sense? How can PBSD improve the profession's delivery of infrastructure?
- Definitions of performance. Data linking engineering demand parameters (e.g., displacements, rotations, strains) with bridge damage and, thus, with bridge system performance are required. The data linking structural behavior to performance must also consider nonstructural and operational characteristics. For instance, displacements must be considered when designing utilities supported on bridges, and these utilities must have performance goals in addition to those defined for the structure. Additionally, permanent displacements of the structure, which may or may not be repairable, will play into performance because such displacements can affect the postearthquake operation of the facility.
- Status of PBSD research. Does enough information exist to transition to PBSD, or if not, what essential elements are currently missing? Is the information consistent for all types of bridges, including data on different superstructure, substructure, foundation, and abutment types?
- Earthquake hazard level. How does earthquake hazard level (expressed as either chance of exceedance in a specific number of years or as return period) play into decision making for PBSD? It is well known that the earthquake input (acceleration or displacement) changes at different rates in different parts of the country. The manner in which this input varies will be considered relative to expected performance in different earthquakes. A single minimum level of earthquake hazard generally will not provide equal protection in all areas of the country. For instance, using a single hazard level (e.g., 1,000-year return period) may not provide the same level of protection and performance in the more seismically active western states as it does in the East because more frequent earthquakes could produce more damage in the West.
- Performance in smaller earthquakes. Can we improve our designs with PBSD such that damage in smaller earthquakes is correctly anticipated and controlled during design?
- A survey of the developments in PBSD from buildings, waterfront/marine (piers and wharves), and bridge perspectives to determine the overall direction in which the earthquake engineering community is moving. This survey will provide an overview and long-range perspective on PBSD. A second objective is to determine how PBSD is and can be used in the nearer term for bridge design. The current design procedures stop short of true PBSD, but it is becoming possible for designers to make choices that affect the likely performance of bridges during earthquakes. This trend is seen in major projects that have been accomplished in the past 15 or so years.
- Criteria used in previous projects. Project-specific criteria for projects where PBSD has been used will be reviewed. Although the scope of the synthesis is new bridge design, criteria for retrofit design will also be considered. The FHWA *Retrofit Manual* is based more nearly on PBSD than are the current new design provisions of AASHTO. Also, the methodology used in building seismic rehabilitation, which is covered in the ASCE Standard 41-06, is reviewed. Both these documents provide a relatively complete methodology for applying performance-based principles.

This synthesis primarily deals with the effects of strong ground motion shaking. Secondary effects, such as tsunami/seiche, ground failure (surface rupture, liquefaction, slope failure, etc.), fire, and flood, are outside the scope of this document. Regardless, their impact on bridges may be substantial and investigation into their effects is undoubtedly important.

RESEARCH APPROACH

The research approach was to conduct an extensive literature and practice review on PBSD. This has been an active area of research for the past 20 years or so, and there have been numerous efforts to implement PBSD on unique, special, and/or important projects.

The literature review canvassed various practice areas, with a focus on bridge, marine, and building design because much work has been accomplished in these areas. Results from one area often help "cross-pollinate" ideas and thinking in the other practice areas. Because the approach to seismic design—permit, but control damage—is the same for buildings, marine structures, and bridges, reviewing these areas can bear fruit that a limited review of only bridges might not. The building practice area has also published design standards that address PBSD, particularly for rehabilitation or seismic retrofit projects. The review covers primarily U.S.-based work, but international research and specification-development efforts were also reviewed.

Bridge practice review was accomplished through a survey of all 50 states, with a particular focus on states that have higher seismic hazard. A sampling of organization-specific and project-specific design requirements was collected and reviewed.

From the literature and practice review, an overview of the current status of PBSD engineering details and deployment has been assembled. A general direction of the development of the practice is evident. From these, a road map forward for the bridge engineering community, including near-term research needs, has been achieved.

REPORT ORGANIZATION

The report is divided into 13 chapters, including this first chapter, which covers background, statement of the problem, definition of PBSD, objectives of the study, and the research approach.

Chapter two reviews public and engineering expectations of seismic design and discusses the regulatory framework and associated issues and challenges.

Chapter three contains the resulting findings of a literature review, including reviews of the bridge, building, and marine structures (piers and wharves) practice areas with emphasis on their respective design specifications. This includes an overview of current bridge seismic design practice as specified by the AASHTO LRFD *Bridge Design Specifications*, the AASHTO *Guide Specification*, and the FHWA *Retrofit Manual*.

Chapters four through seven contain the detailed findings from literature review for the four primary areas of PBSD: seismic hazard analysis (chapter four), structural analysis and design techniques (chapter five), damage analysis (chapter six), and loss analysis (chapter seven).

Chapters eight through ten review the individual organization (chapter eight) and project-specific bridge practices (chapter nine), which are compared and summarized in chapter ten. The review includes descriptions of specific practices of various departments of transportation (DOTs) in the seismic design area, as well as brief descriptions of selected project-specific data that were contributed by various DOTs. Additional information was generated by a survey questionnaire that was sent to all 50 U.S. DOTs, the results of which are summarized in chapter eleven.

Chapter twelve identifies knowledge gaps and explores the information that is needed with respect to prediction of response and damage, as well as the information or methodologies that decision makers need. Finally, chapter thirteen provides conclusions and suggested research, along with suggested short- and long-term implementation efforts.

CHAPTER TWO

PUBLIC AND ENGINEERING EXPECTATIONS OF SEISMIC DESIGN AND THE ASSOCIATED REGULATORY FRAMEWORK

The public expects that structures, including bridges, are designed to resist earthquakes. Beyond that simple statement, it is not abundantly clear what the public really expects, as few surveys of the general public have been conducted and published. The University of Delaware Disaster Research Center used mail surveys and focus groups of Alameda County, California, residents to determine "perceptions of acceptable levels of performance of different elements in the built environment in the event of a major earthquake" (Argothy 2003). Within the portion of the survey and focus group discussions on transportation systems, strong and varied views were expressed, with one respondent stating "there's just not a perfect world, but you don't expect the bridge to fall down when you're driving across it" (Argothy 2003). This clearly reinforces the impression that the public is expecting at least life-safety or no-collapse performance under even the most severe earthquakes. Additionally, some respondents said that closures of important bridges, such as the 1-month closure of the east portion of the San Francisco–Oakland Bay Bridge after the 1989 Loma Prieta earthquake, were unacceptable. This response shows that the general public expects enhanced performance objectives along essential corridors and for signature structures.

Although not specifically addressed in the University of Delaware study, some of the public will undoubtedly surmise that if a new structure is designed for earthquake loading or an existing structure is retrofitted for the same loading, then the structure is "earthquake-proof." Engineers know better, but sometimes only marginally so. We know from experience and from the design codes' general language that life safety must be assured and that damage may be significant for the majority of structures. If a structure is designed with better performance in mind, then we may expect more from the structure, bordering on the earthquake-proof designation. In all, there is a wide range of expectations and a general lack of public consensus as to what to expect following an earthquake. The engineering community does not help the matter because we generally are not adept at articulating how structures will behave under earthquake loading. This situation must be changed if PBSD is ever to take hold because owners, and often the public, must have input into the project's performance objectives.

Additionally, the perception of acceptable performance in both the engineering community's mind and likely in the public's mind may be a function of previous experience. Kawashima (2004) cites survey data of engineers following the 1995 Kobe earthquake, where those who had firsthand experience with that event preferred higher performance objectives than those who had not experienced it. His survey systematically indicated similar trends for repair time, for which those who had experienced the Kobe earthquake gave higher and more realistic estimates. Thus, experience with actual earthquakes is an important parameter in setting realistic goals and expectations, yet many stakeholders in seismic regions across the United States do not have such experience.

For the most part, informed decision making has taken a back seat to "risk and safety as by-products of design" (May 2001). May argues that safety and risk must be treated as explicit considerations and not the products of other choices. The other choices he refers to are the engineering decisions that are made largely out of the public view and that are related to satisfying and choosing among prescriptive requirements of design specifications. The public is not equipped to participate in decision-making discussions that focus on prescriptive engineering requirements.

The decisions made in structural seismic design are almost all based on satisfying requirements related to resistance and structural behavior. May makes the case, and PBSD indeed requires, that more choices be considered in the process, and these choices must be put into terms that are meaningful to the public. For instance, choices about seismic design might be considered relative to such alternatives as purchasing insurance to mitigate the risk of loss as the result of an earthquake or use of alternative facilities. "As with any client, the engineering profession should seek to inform, rather than make, collective decisions about minimum standards of performance for different situations or classes of facilities" (May 2001). However, with our current prescriptive design requirements, we are operating more on the "making decisions" level than on the "seeking to inform" level.

In some sense, the public relies on the regulatory community to consider alternatives and set appropriate requirements. In so doing, the public does not consider or understand the choices that its building officials are making for it. This may have elements of representative democracy, but it does not always bring the public into the decision-making process as fully as may be warranted. Such situations, where public involvement is limited, are much more likely to exist for conventional or ordinary structures or bridges, whereas the opposite is generally true for larger, important projects where the public involvement process is much more overt and developed.

May (2001) again argues that in the project development phases, deliberations must—

- "expose the consequences of choices and their trade-offs with respect to safety/risks, benefits and costs," which is often done for larger projects, but insufficient data typically exist to produce meaningful conversation for ordinary projects.
- "expose distributional aspects of choices," which means that the implications of choices across geographic and economic sectors must be understood.
- "express consequences for different levels of decision making," which simply means that different jurisdictions consider or perceive the consequences of decisions differently because the consequences are indeed different for each—for example, the monetary contributions that are made by each jurisdiction may be different.
- "inspire confidence in the approach and conclusions," which as May observes "may seem obvious, but it is an important lesson that has been lost in past debates over nuclear safety and high-level nuclear waste," for example.

Today, we likely do not consider the effects of each project equally and in the manner described earlier. In some cases, this is because the engineering community simply does not have the resources to make the comparisons and frame the questions that are necessary.

To move beyond the existing situation and begin to more completely apply PBSD, the engineering community will need to develop a "greater societal awareness of earthquake risks and their consequences, but also transform the way that owners, financial entities and the design community think about seismic safety" (May 2007). One of the key aspects of considering such consequences is that there are trade-offs to be considered regarding resources dedicated to mitigating seismic hazards and the risks (i.e., losses) that could come from those hazards.

The Christchurch earthquake sequence triggered by the September 4, 2010, Darfield earthquake is a recent example of the difficulties in allocating resources for earthquake hazard mitigation, where a previously unknown fault unleashed a series of shallow damaging earthquakes and aftershocks over a period of more than 18 months. To a large extent, life safety has been achieved, in that only a few buildings have collapsed. However, many structures are no longer safe to be occupied and, thus, a large fraction of the city's buildings were deemed unusable and need to be demolished and replaced. This tragedy has demonstrated that when a damaging earthquake directly strikes even a modern and well-prepared community, the sheer amount of short-term losses can cause serious disruption to the community and its economic viability after the event (I. G. Buckle, personal communication, 2012).

Other examples are cases where the resources dedicated to hazard mitigation might be somewhat out of line with the perceived benefits that might be achieved. This is an area where accelerated bridge construction (ABC) techniques may become more valuable. Implementation of ABC could lead to permitting more seismic damage than normally would be case, provided life safety is still achieved, because bridge replacement might be quick, thereby reducing delay costs compared with conventional construction. ABC concepts will likely change the dynamic of the decision-making process. Further information on ABC concepts for higher seismic regions can be found in Marsh et al. (2011). Currently, the engineering community is not adept at making such decisions, or at framing the appropriate questions for decision makers. The engineering community will need to do a better job, but this will take time.

With respect to systems where more than one facility is included in a linear network that delivers a service (e.g., highway system), the choices and trade-offs must be considered in the context of the system performance rather than just the individual facility or structure performance. Such tools as Risks from Earthquake Damage to Roadway Systems (REDARS), discussed in chapter seven, help provide a more complete picture of the questions so that decision makers can allocate resources in a manner that is most beneficial to system performance.

With respect to the potential benefits and costs of implementing PBSD, there may not be solely positive aspects of implementing PBSD. Some situations may lead to increased first cost relative to long-term risks. There may be costs of educating the engineering, construction, and regulatory community in terms of using, implementing, and administering PBSD. There may be potential legal risks if target performance goals are not met. And there may be costs associated with inconsistency relating to ambiguous interpretation of performance levels when criteria are unclear (May 2007). Considering the way public work is contracted in this country, unintentional (or less safe) interpretations of performance levels relating to criteria could occur, leading to future problems with facility service.

An advantage of the current prescriptive-based seismic design procedures is that they are somewhat easy to enforce. With prescriptive methods, implementation of design details is binary—a detail either was or was not included. This advantage plays to our method of controlling the construction process, and is evidenced in our special inspection and construction observation procedures. We prefer, and our legal system encourages, that acceptance procedures be enforceable at the time of the construction when the contractor is still on the project and before all payments are made. Prescriptive measures lend themselves to enforcement during the period of the contract; performance-based measures may not and instead rely on the use of warranties in case of future substandard performance. Thus, performance-based methodologies are not yet on the same easy-to-enforce, binary (yes/no) basis as conventional prescriptive seismic design.

This lack of clear guidance leads to a political and legal challenge whereby the enforcement of seismic safety requirements becomes less structured. This particularly becomes challenging when the public is involved in the decision-making process. As May (2007) articulates,

> on the one hand, determining levels of acceptable risk is fundamentally a value judgment that presumably requires some form of collective decision-making. On the other hand, knowledge of relevant risk considerations, technical details, and costs and benefits are important for establishing minimum standards. The first consideration argues for public processes for establishing safety goals. The second argues for deference to technical experts. Finding the appropriate middle ground is a serious challenge.

May and Koski (2004) illustrate this challenge in *Performance-Based Regulations and Regulatory Regimes.* They first observe that the move toward performance-based design is related to the general modern political movement to relax regulation to foster innovation and remove barriers to economic growth. However, their treatment of the regulatory environment points out through four different case studies how the challenge of open decision making and performance-based approaches to regulation may not achieve the desired outcomes.

For instance, they review the introduction of performance-based code provisions for home construction in New Zealand in 1991, which coincided with popular preferences for stucco or adobe finishes on home exteriors. Problems with moisture, leakage, and so on began to emerge, and by the early 2000s a crisis was at hand. There were many inadequacies in the regulatory system (code provisions) and with the construction industry's delivery of homes. Reforms were enacted to swing the pendulum back, to increase government oversight, to more clearly define the performance standards, and develop mechanisms to monitor products and provide warnings about defective ones. In all, "a general tightening of the regulatory regime with emphasis on greater specification of performance standards and stronger monitoring of building inspection practices" was enacted. This case history illustrates the need for balance between performance-based objectives and oversight of the industry.

An observation of modern design specifications is that many, if not most, of the provisions and prescriptive requirements were included in the specifications to prevent some type of failure or poor service performance that actually occurred at some time and at some location. When prescriptive requirements are incorporated into design specifications, the associated details of the poor performance the provisions are intended to prevent are often lost. This loss accelerates proportionally with time from the originating failure or research. The preservation of such behavioral information is one of the primary reasons for including a commentary to a design specification.

Good engineering involves anticipating and preventing modes of failure, and if previous lessons learned regarding past failures are lost in a morass of prescriptive design provisions, then innovation is stifled and the engineering community is likely destined to relive the past. Therefore, the well-crafted performance-based design specification would likely control specific modes of failure by a combination of performance requirements and a fallback to prescriptive requirements when performance objectives are unclear or ambiguous. This process might be thought of as a hybrid approach to performance-based design, and, given our struggle with balancing the challenges of regulation and the desire to innovate, such a hybrid design specification may be the most logical way forward.

CHAPTER THREE

REVIEW OF INDUSTRY PRACTICE

A literature review of the current seismic design standard of practice for the bridge, building, and waterfront/marine industries was conducted. This chapter provides the results.

BRIDGE INDUSTRY PRACTICE

Current AASHTO Practice

Two seismic design methods are codified as minimum standards and permitted by AASHTO. One is a force-based method that is embedded into the AASHTO *LRFD Bridge Design Specifications* (AASHTO 2012), referred to herein as the AASHTO LRFD, and the other is a displacement-based method that is the basis of the AASHTO *Guide Specifications for Seismic Bridge Design* (AASHTO 2011), referred to as the AASHTO SGS. The LRFD gives these two methods equal weight, thus permitting the displacement-based method of the SGS to be used in lieu of the force-based method, even though the displacement-based method is outlined only in a guide specification. Both methods use a single-level earthquake input, a 1,000-year return period ground motion.

The force-based method has its roots in the improved design procedures that followed the 1971 San Fernando earthquake in southern California. Caltrans and AASHTO quickly updated their design procedures, and in 1981 the Applied Technology Council (ATC) published ATC-6, *Seismic Design Guidelines for Highway Bridges* (ATC 1981). AASHTO adopted this document as a guide specification in 1983, and it was formally adopted into the *Standard Specifications for Highway Bridges* in 1991 following the Loma Prieta earthquake. These design provisions became the basis for the seismic provisions included in the AASHTO LRFD. These force-based provisions were modified over the years as improvements were identified; however, the provisions remain largely as they were formatted in the ATC-6 document.

The AASHTO seismic design provisions seek to produce a structure that can resist more common smaller earthquakes without significant damage and to resist larger, rare earthquakes without collapse. However, in the larger event the damage may be severe enough that repair of the structure is not feasible; the objective is simply to prevent loss of life. While the design approach generally seeks to deliver these performance objectives, there is no direct quantitative check of multilevel earthquake loading, nor is there a direct linkage between the design parameters checked and actual damage states.

From the perspective of performance objectives, the two specifications differ in that only the AASHTO LRFD addresses design of more important structures. The AASHTO SGS has its origins in part in the Caltrans *Seismic Design Criteria* (SDC) (2006a) for ordinary standard or conventional bridges. In the case of a bridge with a higher importance being designed with the AASHTO SGS, project-specific criteria would need to be developed, which is the approach that Caltrans uses for such bridges (Caltrans 2010b). The AASHTO LRFD defines three operational classifications of bridges: Other, Essential, and Critical. The AASHTO LRFD commentary describes Essential bridges as those that should be open to emergency vehicles immediately after a 1,000-year event. Critical bridges must remain open to all traffic after the design event and be open to emergency vehicles after a 2,500-year event. However, such performance is not directly assessed.

The force-based method, as implemented in the AASHTO LRFD, is built around the capacity design process that has its origins in New Zealand in the late 1960s. The process was credited to John Hollings by Robert Park in his interview with Reitherman (2006). John Hollings (Park and Paulay 1975) summarized the process thus:

> In the capacity design of earthquake-resistant structures, energy-dissipating elements of mechanisms are chosen and suitably detailed, and other structural elements are provided with sufficient reserve strength capacity, to ensure that the chosen energy-dissipating mechanisms are maintained at near full strength throughout the deformations that may occur.

In the force-based procedure an elastic analysis of the bridge is performed under the requisite earthquake loading and internal forces are determined. Forces in elements that are those chosen for energy dissipation—typically columns—are reduced by a response modification factor, *R*, and then combined with concurrent nonseismic forces to generate the design forces. These forces would typically be in the form of column moments at selected plastic hinging locations. The reinforcement for these locations is chosen to match the required design moments; then these locations are prescriptively detailed to be adequately ductile. The remainder of the bridge, including foundations, superstructure, bearings, abutments, and the nonyielding portion of the columns, is designed to be able to withstand the maximum possible forces—known as overstrength forces—that the plastic mechanism would ever be capable of generating. This process essentially satisfies the capacity design objective, although a direct check of the actual expected response, inclusive of yielding effects and demand displacements, is never made, and a direct check of ductility capacity is, likewise, never made. This process was developed to be expedient for design using elastic analysis tools, and is further discussed in chapter five.

The aforementioned *R*-factors are in effect a measure of the ductility capacity of the structural system: Large *R*-factors imply that the system has a high displacement ductility capacity and small *R*-factors imply low displacement ductility capacity. One difficulty of this method is that a single *R*-factor cannot provide a reliable method of damage or performance control under certain structural configurations. For example, two reinforced concrete columns that differ only in their height will have two different displacement ductility capacities (and therefore *R*-factors); the longer column will have a lower ductility capacity owing to the increased influence of elastic deformations to the overall displacement (i.e., the ratio of plastic hinge length to the overall column length reduces with an increase in column length). Changes in behavior of this kind are best captured using displacement-based methodologies, such as those adopted into the AASHTO SGS.

The displacement-based method in the AASHTO SGS focuses the designer's attention on checking the system deformation capacity rather than selecting the precise resistance of the yielding or energy dissipating elements. This method is based heavily on the Caltrans practice for conventional bridges (Caltrans 2006a). The design process then becomes one of checking a trial design, rather than a linear progression of steps to calculate required internal forces in the structure. The process still follows the capacity design overall methodology, in that locations for damage are selected; these locations then are detailed to deliver adequate displacement or ductility capacity, and that capacity is directly checked. In the displacement-design process, the effect of confinement steel, for example, is directly included in the calculation of displacement capacity. Thus, the designer has some direct control over the amount of ductility or deformation capacity that will be provided versus the amount of ductility that is required.

The elements that are not part of the energy-dissipating mechanism are subsequently designed to be adequately strong under the maximum expected actions of the plastic mechanism. In principle, this step is identical to the one for force design. In application, the process differs primarily by the material strength factors that are used.

In the AASHTO SGS displacement-based design process for the high seismic areas—Seismic Design Category D (SDC D)—the deformation capacity is controlled by limiting the maximum amount of tensile strain in the reinforcement steel and the maximum concrete compressive strain. The design method links element strains to member curvature, then to member rotations, and finally to member and systems displacements.

Figure 3 shows the relationship between global and local deformations and damage, where a cantilever reinforced concrete column is subjected to an inertial lateral force (*F*) at the

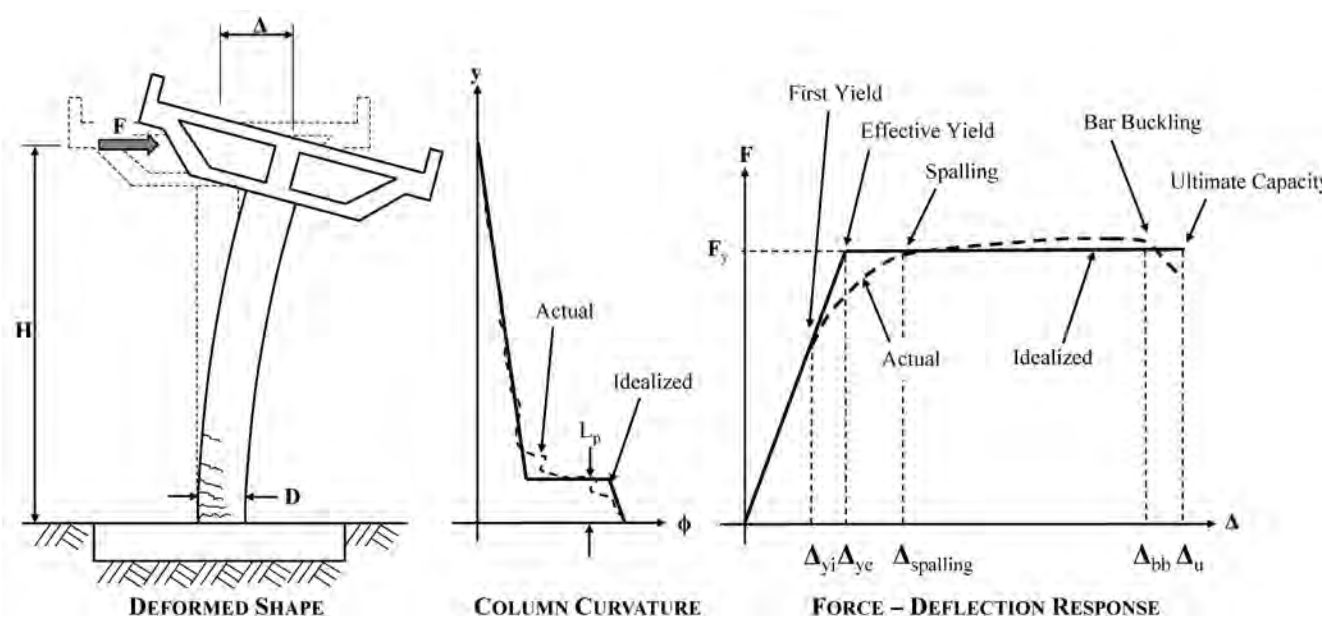


FIGURE 3 Cantilever column deformations and limit states.

center of gravity of the substructure, resulting in the deformed shape shown. Local deformations (strains) are related to the global displacement (Δ) through the curvature distribution along the height of the column (often idealized as shown, taking advantage of the plastic hinge length, L_p). Finally, using the strain limits, displacement limits can be determined as indicated on the force-deflection (pushover) response.

This calculation is not made in the force-based method. Because of this key difference, it is logical that the displacement-based process of the AASHTO SGS is the appropriate method into which to incorporate performance-based design. Although the AASHTO LRFD force-based method attempts to differentiate ordinary, critical, and essential bridges, giving the impression of accommodating different performance objectives, the method is ill suited for a performance-based process specifically because deformation adequacy is not directly checked at the earthquake demand level. That said, at present the AASHTO SGS displacement-based method addresses only ordinary bridges and does not provide criteria or guidance for more important structures. This is a key gap that easily could be closed in the near future.

It is also important to recognize that performance-based design procedures are possible because a capacity-design process is used. The selection of damage-tolerant elements and their subsequent design to accommodate earthquake demands permits these elements to be designed to respond with more or less damage, depending on the performance objectives desired. Such a process is predicated on the ability to relate engineering demand parameters—strain, rotations, and so on—to damage states and then operational performance. Thus, the capacity design method is a key component of the PBSD process. Although both AASHTO design methods are based on capacity design, the AASHTO SGS displacement-based method is better suited for extension into performance-based design. The AASHTO SGS method could be converted to a nominally performance-based approach by using concrete and reinforcing steel strain limits that are correlated to specific damage states—for instance, spalling or bar buckling.

The AASHTO SGS method uses implicit formulae to calculate the displacement capacity of reinforced concrete columns for the intermediate Seismic Design Categories (SDCs) B and C. The formulae were derived using data from Berry and Eberhard's (2003) database of column damage, whereby statistics for experimental tests of columns were developed using the two damage states, spalling of the cover concrete and buckling of reinforcement (discussed in further detail in chapter six). The experimental spalling data are averaged with the AASHTO analytical spalling limit state to calculate the displacement capacity for SDC B, and the experimental spalling data are averaged with the analytical data corresponding to attainment of a displacement ductility of four for SDC C. The data used for the implicit formulae are shown in Figure 4 for SDC B and Figure 5 for SDC C. The averaging process used is evident in the figures by the SDC line's position relative to the neighboring lines. An estimate of the displacement capacity of the column is calculated from the column aspect ratio. The graphs represent cantilever columns with fixity at one end only. Other configurations are handled by decomposing the columns into equivalent cantilevers joined at the inflection points.

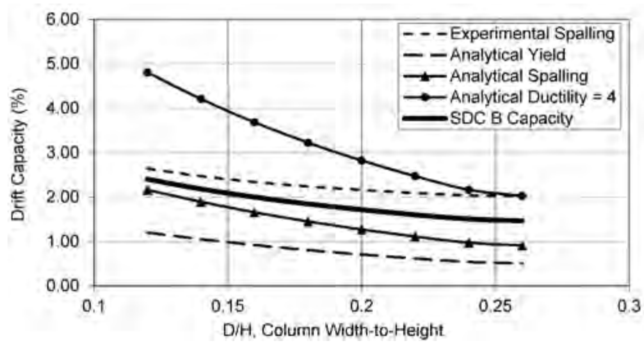


FIGURE 4 SGS Seismic Design Category B displacement capacity (after Imbsen 2006).

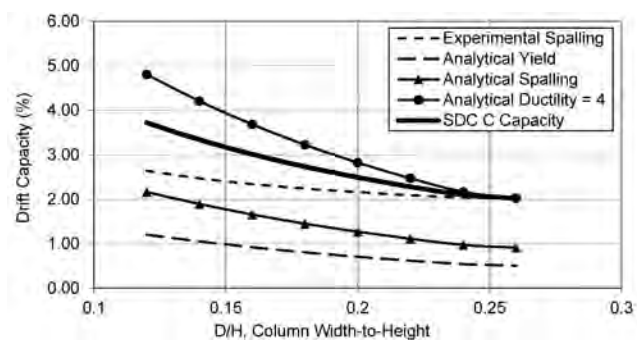


FIGURE 5 SGS Seismic Design Category C displacement capacity (after Imbsen 2006).

For SDC B, this method produces a displacement capacity that is a conservative estimate of the spalling limit state. It is conservative because the implicit capacity is less than the average spalling limit state owing to the slightly conservative nature of the analytical spalling data. Likewise, for SDC C the displacement capacity lies between spalling and the attainment of a displacement ductility of four. These estimates are meant to be easy to calculate and are conservative as shown. The limits are also linked to the minimum confinement reinforcement required in columns in SDCs B and C. These categories each have lower required transverse steel contents, leading to lower expected ductility capacity. Therefore, the maximum permitted ductility demand must also be kept low. The idea behind this approach was to ease congestion of reinforcement, provided that the lower capacities were adequate for the anticipated demands. If designers wish to expend more effort or if the implicit capacities are too low, they may use the more rigorous displacement capacity calculation method for SDC D and, by doing so, calculate a larger displacement capacity; however, more confinement steel will typically be required.

This is a simple example of how data that relate engineering parameters to damage are currently used. A true PBSD methodology will require more data and correlation of this nature. In the case of probabilistically based PBSD, the statistical dispersion of the data around a central tendency will also be required. This will be discussed further later in this synthesis.

Much of what is discussed in this synthesis is related to reinforced concrete (RC) construction following the Type 1 (ductile substructure with essentially elastic superstructure) design strategy as defined in the AASHTO SGS. This type of construction is the most common, and the bulk of laboratory testing and design methodologies apply to RC construction. Ultimately, the PBSD techniques developed for RC will have to also be developed for other types of construction, including Type 1 structures with steel columns or concrete-filled-tube columns. AASHTO Type 2 structures, those with an essentially elastic substructure and ductile steel superstructures (e.g., ductile cross frames or diaphragms) have seen relatively little research attention in the context of PBSD, certainly constituting an area of research need. AASHTO Type 3 structures, those with an elastic superstructure and substructure with a fusing mechanism (seismic isolation and supplemental damping devices) between the two, are well researched, with many publications specifically devoted to their analysis and design. Chapter six briefly discusses Type 2 and 3 structures in terms of their ability to reduce structural damage.

The two AASHTO seismic design specifications, LRFD and SGS, are becoming increasingly difficult to maintain because it is challenging to maintain parity of treatment between them as new information is added. It is likely that a choice will need to be made in the not-to-distant future regarding keeping both design methodologies—force-based and displacement-based—or dropping the force-based procedures. These maintenance challenges will likely increase if PBSD elements, whether mandatory or optional, are adopted into the specifications.

FHWA Retrofitting Manual

The FHWA *Seismic Retrofitting Manual for Highway Structures: Part 1—Bridges* (2006) is essentially a performance-based guideline, which uses a multiple-level approach to performance criteria. It defines two seismic hazard levels, 100-year and 1,000-year return periods, and uses anticipated service life (ASL), along with importance, to categorize suggested performance levels (PLs). The ASL values are as follows:

- ASL 1: 0–15 years
- ASL 2: 16–50 years
- ASL 3: >50 years.

The PLs range from PL0 to PL3, and the corresponding expected postdesign earthquake damage levels are as follows:

- **PL0: No minimum**—No minimum level of performance is specified.
- **PL1: Life safety**—Significant damage is sustained, service is significantly disrupted, but life safety is preserved. The bridge may need to be replaced after a larger earthquake.
- **PL2: Operational**—Damage sustained is minimal and service for emergency vehicles should be available after inspection and clearance of debris. Bridges should be repairable with or without traffic flow restrictions.
- **PL3: Fully operational**—Damage sustained is negligible and full service is available for all vehicles after inspection and debris clearance. Damage is repairable without interrupting traffic.

Importance is set as either standard or essential, where essential is defined as a bridge that (1) provides for secondary life safety, such as emergency response vehicle use, (2) would create a major economic impact, (3) is formally defined in an emergency response plan as critical, or (4) is a critical link in security and/or defense road network.

Table 1 presents the minimum performance levels, determined by combining seismic hazard level, importance, and ASL.

TABLE 1
MINIMUM PERFORMANCE LEVELS FOR RETROFITTED BRIDGES

Earthquake Ground Motion	Bridge Importance and Service Life Category					
	Standard			Essential		
	ASL 1	ASL 2	ASL 3	ASL 1	ASL 2	ASL 3
Lower-Level Ground Motion 50%/75 years (approx. 100 years)	PL0	PL3	PL3	PL0	PL3	PL3
Lower Upper Ground Motion 7%/75 years (approx. 1,000 years)	PL0	PL1	PL1	PL0	PL1	PL2

Source: FHWA (2006).

These performance criteria are then combined with appropriate assessment techniques to determine whether retrofit is required. From that point, a retrofit strategy is selected (if required), then approaches to satisfy that strategy are developed and retrofit measures are defined to provide the selected approach. The strategy is the overall plan for retrofitting the bridge and may include several approaches made up of different measures. Example strategies may include do nothing; partial retrofit of the superstructure; or full retrofit of the superstructure, substructure, and foundations. Approaches may include such things as strengthening, force limitation, or response modification. Measures are physical modifications to the bridge, such as column jacketing.

While the *Retrofitting Manual* includes performance-based objectives for different levels of importance, service life, and ground motions, it does not address in detail the linkage between the performance levels, PL0 to PL3, and damage limit states. For reinforced concrete, for example, the following limit states are quantified in terms of curvature:

- Unconfined concrete compression failure
- Confined concrete compression failure
- Buckling of longitudinal bars
- Fracture of longitudinal reinforcement
- Low-cycle fatigue of longitudinal reinforcement
- Lap-splice failure
- Shear failure
- Joint failure.

Using these damage limit states and the descriptions of the required performance or service levels following an earthquake, an engineer can establish criteria that would deliver the required performance. For example, to meet the PL3 performance level of fully operational, the limiting performance levels would be to prevent—

- Unconfined concrete compression failure
- Shear degradation
- Yielding of the longitudinal bars
- Joint failure
- Lap-splice failure.

The more serious longitudinal bar damage states, such as buckling, fracture, and low-cycle fatigue, would not be an issue if yield of the bars were prevented.

The relative lack of detailed procedures to link performance, damage states, and finally operation reflects the state of practice in 2006, when the manual was published. However, the damage states are overtly stated in terms of physical damage, not simply strain limits or curvature limits in a table. These physical damage states may be compared with those of Berry and Eberhard's database, discussed in the previous section, which were used for the AASHTO implicit displacement equations. However, in the *Retrofitting Manual*, material properties and curvature limits reflect the performance and behavior of older materials, and as such the limiting values may be considerably lower than those associated with new construction. These damage states also correspond to the Caltrans visual damage guidelines described in chapter six. Linking these observations together, one may conclude that the way forward into PBSD would be to develop specifications that combine these physical damage states with analytical methods of structural analysis on the one hand, and performance and loss estimates on the other hand.

An alternative rating method called the Seismic Rating Method Using Expected Damage is briefly outlined in the retrofit manual. The method provides a concise overview of the process of using the National Bridge Inventory database, standard bridge fragility functions, (the concept of fragility functions is defined in the Damage Prediction section of chapter six. Briefly, a fragility function relates the likelihood or probability of attaining a specified damage state to an EDP such as drift, where, for example, first yield of a column can be related probabilistically to drift.), and repair cost data to calculate a ranking, *R*. These data are combined with estimates of indirect losses, network redundancy, nonseismic deficiencies, remaining useful life, and other issues to determine an overall priority for retrofit. This method follows the four basic steps of probabilistic PBSD to determine the retrofit priority.

Although the expected damage method provides an overview of the process, significant data and methodology remain to be developed before the method can be applied with the same level of precision as the more conventional deterministic techniques, which are outlined in detail in the *Retrofitting Manual*. However, those conventional techniques are well developed only through the first two steps of PBSD, seismic hazard analysis and structural analysis. The remaining two steps, damage prediction and loss prediction, require more development before they mature, and this development will likely be a focus area in the coming years in earthquake engineering research.

BUILDINGS INDUSTRY PRACTICE

Overview

Buildings seismic design practice evolved similarly to bridge design practice in that life safety has been the primary minimum goal of both design methodologies. Like the AASHTO bridge seismic design procedure, the building codes sought indirectly to provide structures that could resist smaller, more frequent earthquakes with little or no damage and larger, rare earthquakes with significant damage, but without loss of life—hence the name, life safety. For example, The *International Building Code* (2009) defines as its purpose "to establish the minimum requirements to safeguard the public health, safety and general welfare.…" However, there is no direct check that the life safety performance objective is met for a code-compliant design. Instead, the design must comply with given design parameters and prescriptive detailing requirements. If these requirements are met, adequate seismic performance is implied in much the same way as in the AASHTO LRFD. The implication is that if the structure meets the code, then life safety is reasonably assured. However, this approach, while simple, can result in some structures performing better than others under earthquake loading, even though the structures were designed to the same code.

The need to seismically rehabilitate existing buildings has led to the development of "first-generation guidelines" that have PBSD as their core objective. In recognition that existing buildings contained elements that did not conform to new building design requirements, a need existed for alternative means of setting criteria for rehabilitation that departed from criteria for new buildings. Often, existing building elements cannot be made to perform to the strength, stiffness, and ductility levels expected of new buildings, so different criteria were needed to guide such rehabilitation. The Structural Engineers Association of California (SEAOC), with FEMA funding, produced its Vision 2000 report, *Performance Based Seismic Engineering of Buildings,* in 1995 with the goal of defining both rehabilitation and new building seismic design criteria. It was the first such document to define multiple discrete levels of earthquake design. These levels are shown in Figure 6. Multiple performance levels were defined, along with multiple earthquake design levels. Then, performance objectives were defined as groups of combined earthquake and performance levels. For example, the Safety Critical Performance Objective was a combination of fully operational performance in the rare earthquake and operational performance in the rare event.

At about the same time, FEMA funded the development of national guidelines for the rehabilitation of buildings, which led to the publication of FEMA 273, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (1996). FEMA 273, along with its commentary, FEMA 274, used a similar multiple design level approach. For more important structures, the criteria became more rigorous. The FEMA documents used slightly different earthquake hazard levels, but the concept was the same as that first presented by SEAOC. In 2000, FEMA 273 and FEMA 274 were revised into FEMA 356, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings* (2000f), which represented the second generation of PBSD guidelines for existing buildings. FEMA 356 was subsequently adopted in 2006 as ASCE 41-06, *Seismic Rehabilitation of Existing Buildings*, which is still the standard for developing rehabilitation designs for buildings.

In the ASCE 41-06 approach, several earthquake levels are assessed, depending on the overall performance objective that is selected (see Table 2 for the performance levels and Table 3 for the definitions of the performance levels and their associated damage states). The acceptance criteria are also provided on an element-by-element basis, depending on the desired performance level that is being checked. Thus, more restrictive limits are provided for each element type and for each performance level. The ASCE 41 methodology permits both force- and displacement-based assessments to be made. The force-based approach uses m-factors, which are essentially element-based R-factors. The displacement-based approach uses deformation limits, such as element rotations for moment frames and shear walls and element displacements for bracing elements. While both force- and displacement-based approaches are used, the displacement-based method is preferred and is required in some cases, depending on structural regularity, desired performance objectives, and other parameters.

Structural actions are checked at the element level, and each primary lateral force resisting element is classified as either force-controlled or deformation-controlled. When checking force-controlled (brittle) elements, the nominal resistance is used to form a design resistance similar to

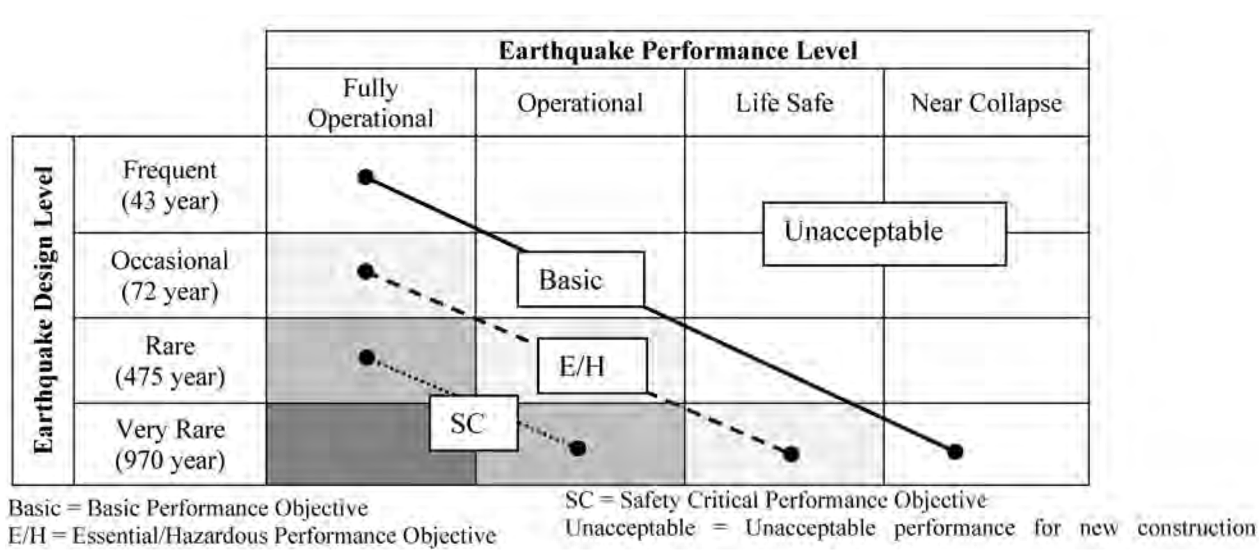


FIGURE 6 Performance objectives for buildings (SEAOC Vision 2000).

TABLE 2
REHABILITATION OBJECTIVES (ASCE 41-06)

		Target Building Performance Levels			
		Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
Earthquake Hazard Level	50%/50 year	a	b	c	d
	20%/50 year	e	f	g	h
	BSE-1 (≈10%/50 year)	i	j	k	l
	BSE-2 (≈2%/50 year)	m	n	o	p

Notes:
1. Each cell in the above matrix represents a discrete rehabilitation objective.
2. The rehabilitation objectives may be used to represent one of three specific rehabilitation objectives, as follows:

Basic safety objective (BSO)	k and p
	k and m, n, or o
	p and i or j
	k and p and a, b, e, or f
	m, n, or o alone
Limited objectives	k alone
	p alone
	c, d, g, h, or l alone

TABLE 3
DAMAGE CONTROL AND BUILDING PERFORMANCE LEVELS (EXCERPTED FROM ASCE 41-06)

	Target Building Performance Levels for Seismic Rehabilitation			
	Collapse Prevention Level (5-E)	Life Safety Level (3-C)	Immediate Occupancy Level (1-B)	Operational Level (1-A)
Overall Damage	Severe	Moderate	Light	Very Light
Structural	Little residual stiffness and strength, but load-bearing columns function. Large permanent drifts. Building is near collapse. Some exits blocked.	Some residual strength and stiffness left in all stories. Gravity load-bearing elements function. Building may be beyond economical repair.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades and partitions. Elevators can be restarted. Fire protection operable.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades and partitions. All systems' important to normal functions are operable.
Comparison with Performance Intended for New Buildings	Significantly more damage and greater risk	Somewhat more damage and slightly higher risk	Less damage and lower risk	Much less damage and lower risk

Note that the number/letter designator reflects structural damage by the number and nonstructural damage by the letter. Numbers range from 1 to 5, letters from A to E. Lower numbers and earlier letters in the alphabet are better. The range of structural and nonstructural damage states reflects the range of performance permitted for seismic rehabilitation.

way column shear is checked in AASHTO seismic design methods. When checking deformation-controlled elements, maximum inelastic deformations for each performance level are specified.

For example, a reinforced concrete beam with symmetric top and bottom reinforcement, transverse reinforcement that conforms to minimum confinement details, and a low shear demand would have permissible plastic rotation angles of 0.01, 0.02, and 0.025 rad for immediate occupancy, life safety, and collapse prevention, respectively. These checks are similar to those made using the AASHTO SGS, although they are made at the rotation level rather than strain level.

ASCE 41-06 also permits two methods of inelastic displacement estimation, the coefficient method and the capacity spectrum method. Chapter five describes these methods.

It has been observed that the FEMA documents and the follow-on standard, ASCE 41-06, have several significant shortcomings:

> First the procedures do not directly address control of economic losses, one of the most significant decision maker concerns. Also, the procedures are focused on assessing the performance of the individual structural and nonstructural components that comprise a building, as opposed to the building as a whole. Perhaps most significantly, the reliability of the procedures in delivering the design performance has not been characterized (ATC 2003).

In fact, many engineers who have worked with and applied the documents believe that they are too conservative and restrictive, and lead to inappropriate engineering analysis and strengthening of structures (Searer et al. 2008). One of the main complaints by Searer and colleagues is that performance-based earthquake engineering is not a subject that is suitable for "standardization" or "cookbook-ization." Their point is that performance-based earthquake engineering, particularly for rehabilitation of existing structures, must rely on first principles, be done on the merits or lack thereof of each structure, and be performed by appropriately trained and qualified engineers. These issues must be taken into account in the development of any new document purporting to guide the performance-based engineering process.

Following the 1994 Northridge earthquake, FEMA sponsored the SAC *Program to Reduce the Earthquake Hazards of Steel Moment-Frame Structures* because an unexpectedly large number of moment frame buildings were damaged in the earthquake. SAC was a joint venture formed by SEAOC, ATC, and the Consortium of Universities for Research in Earthquake Engineering (CUREE. This effort led to the FEMA 350 *Recommended Seismic Criteria for New Steel Moment-Frame Buildings* (2000b) report and the three associated reports (FEMA 351, 352, and 353; 2000 c, d, e). The importance of this effort with respect to PBSD is that—

> these recommended criteria specifically quantified performance in terms of global behavior of buildings, as well as the behavior of individual components, and also incorporated a formal structural reliability framework to characterize the confidence associated with meeting intended performance goals (ATC 2003).

The consideration of global or system behavior, plus the addition of the reliability framework, represent a substantial leap forward with respect to PBSD, which directly corresponds to the probabilistic framework that was described conceptually in chapter one. Although the framework proposed was complex and not ready for adoption into formal design specifications, it did provide a launching point for the next effort, ATC-58, *Seismic Performance Assessment of Buildings.*

The incremental improvements seen from the Vision 2000 and FEMA 273 approaches onward are relevant for the bridge seismic design community because they are indicative of the amount of work that must be expended to develop PBSD procedures and methodologies.

Recent/Current Efforts

In the wake of the 1989 Loma Prieta and 1994 Northridge earthquakes, where little loss of life was incurred, some relatively new structures were damaged to the point that they could not be reoccupied, and the ensuing direct and indirect economic losses were surprisingly high [$7 billion in Loma Prieta and $30 billion in Northridge (ATC 2003)]; so high that conversations began about what it meant to meet code. It was clear that the structural and nonstructural damage was not consistent with public expectations of acceptable damage in a code-compliant design (FEMA 2006).

The development of first- and second-generation guidelines, as described previously, was undertaken to address the disparity between actual structure performance and code-inferred performance. However, it was also recognized that new design should be kept simple by the use of advanced assessment techniques. The road to PBSD of buildings was going to be longer than some had perhaps first envisioned, and it would take a great deal of effort. To address the way forward, both the Earthquake Engineering Research Center (EERC), which is now part of PEER, and the Earthquake Engineering Research Institute (EERI) drafted action plans for development of PBSD. These plans were published as FEMA 283, *Performance-Based Seismic Design of Buildings—an Action Plan* (1996), and FEMA 349, *Action Plan for Performance Based Seismic Design* (2000a), and were authored by EERC and EERI, respectively.

In 2001, FEMA awarded a contract to ATC to conduct a long-term project to prepare the next generation of PBSD guidelines, a multiyear, multiphase effort known as the ATC-58 Project. The project was estimated at $21 million for two phases in 2004 dollars, but funding was not available for the full scope, and the project was finally funded at about 50% of the original estimated level. The project, which is still in progress, issued a 75% draft in 2011, 10 years after beginning, reflecting that the project is not typical in terms of effort or time to complete. Several features of the ATC-58 project that are relevant to this synthesis were the subject of a workshop that is discussed here, along with the current plan for the ATC-58 project and the project's technical direction.

In 2002, a workshop was held with primary stakeholders—building owners, tenants, lending institutions, building regulators, and others who had no formal training in probabilistic risk assessment concepts—to develop methods to communicate earthquake risk (ATC 2002). The "participants confirmed that life losses, direct losses, and indirect economic losses are the primary aspects of earthquake concern." Some participants were keenly interested in being able to quantify the length of time a facility might be out of service and to quantify associated economic losses. Interestingly, life safety was not a primary topic in the workshop, and it is believed that was because most participants felt that assurance of life safety was a given in a code-compliant structure. Also of interest was the participants' preference to consider earthquake effects and losses using a scenario event rather than probabilistic considerations of earthquake losses. Annualized losses were the least favored way to compare data. All participants understood the uncertainties in the prediction of losses from seismic events, and even though there was not a preference for probabilistic treatment, the use of confidence (as in 10% chance of exceedance) was favorably received.

The current plan for the project is to develop a methodology that will provide a framework for identifying probable consequences in terms of human losses, direct economic losses, and indirect economic losses. The framework is outlined in detail in *Seismic Performance Assessment of Buildings Volume 1—Methodology* of the 75% draft of ATC 58-1 (2011), and "the companion Volume 2 provides guidance on implementing the technology, including instructions on how to use an electronic Performance Assessment Calculation Tool (PACT) that has been developed to enable practical implementation of the methodology."

The technical direction of the ATC-58 project has the goal of replacing performance levels that are currently used, such as fully functional, immediate occupancy, life safety, and collapse prevention, with probable future earthquake impacts as measures of performance. The following impacts are considered:

- Casualties—the number of deaths and injuries of a severity requiring hospitalization
- Repair cost—including the cost of repairing or replacing damaged buildings and their contents
- Repair time—the period of time necessary to conduct repairs or replace damaged contents, building components, or entire buildings
- Unsafe placards—the probability that a building will be deemed unsafe for postearthquake occupancy.

Clearly, these impacts would be relevant to decision makers, such as owners of buildings, but the approach also represents a leap beyond the current method of attempting to quantify building performance based on a limited set of element deformation or force levels.

To address the necessary methodology and data requirements, significant effort has gone into developing tools that knowledgeable designers might eventually use to produce consistent and technically sound performance-based designs. The 75% draft of *Volume 2—Implementation Guide* outlines this process, provides the PACT calculation package, and describes the probabilistic damage state (or fragility, which is defined in the Damage Prediction section of chapter six) database that supports the process. The National Institute of Standards and Technology Interagency Report (NISTIR) 6389, *UNIFORMAT II Elemental Classification for Building Specifications* (NIST 1999) specifies unique identification codes for common structural and nonstructural systems and components. The PACT database ties system and component fragilities to those identification codes, thus permitting PACT users to simply input a code that will then invoke the appropriate fragility function. Fragility databases are clearly a useful method to ensure consistent application of fragilities across the design community. To fully deploy PBSD, the bridge community will need to develop a similar methodology.

Significant progress has been made in the identification of knowledge gaps and the research required for the full implementation of PBSD in buildings, including limitations of the ATC-58 project. This was published in the NIST Grant/Contract Report (GCR) 09-917-2 report (NIBS 2009), which outlines in detail 37 research topics that are critical for PBSD implementation. Included are tasks related to the determination of data to define fragility relationships for components found in old and new buildings, and to determine the performance of buildings designed according to prescriptive codes and standards in order to improve building codes and ensure a smooth transition to the widespread use of PBSD in the next decade.

As a general observation, the range and complexity of construction types is probably greater for buildings than for bridges. Thus, the PBSD development effort for bridges should be less than that for buildings.

ASCE 7-10

The ASCE 7 Standard, *Minimum Design Loads for Buildings and Other Structures*, is the primary structural reference governing the seismic design for the *International Building Code* (IBC). The 2010 version of the ASCE 7 (ASCE 2010) has taken a significant departure from previous editions. The mapped values for seismic ground motions are probabilistic ground motions that are based on uniform risk rather than uniform hazard. This means that a notional, standard, or generic probability of collapse has been used to translate the seismic hazard (a property of a structure's site and geographic location) to a seismic risk of structural collapse (a property of site, location, structure type, and assumed damage state). An overview of the process and rationale for the change is given in FEMA P-749 (FEMA 2010) and in Luco et al. (2007). FEMA P-749 also provides an overview of the ongoing development and regulatory process of the U.S. building codes with respect to seismic design.

The previously mapped values were for uniform seismic hazard with a 2% chance of exceedance in 50 years. This hazard was known as the maximum considered earthquake (MCE) and was given as spectral accelerations with uniform probabilities of exceedance as a function of period. The new mapped values are for 1% chance of exceedance in 50 years, risk-adjusted maximum considered earthquake (MCER) spectral accelerations. Stated differently, these mapped values represent ground motions that result in a 1% chance that the structure could collapse in 50 years.

The new risk-based maps result in slight reductions in the 1-second spectral accelerations in many areas of the country, including the eastern United States. These reductions range from 0% to 20%. In the more seismically active areas of the country—California, Alaska, and Hawaii—the 1-second spectral accelerations increase slightly, on the order of 0% to 20%. ASCE 7-10 provides maps of these adjustment factors.

The reasons that the acceleration levels change are related to (1) how the accelerations change with increasing return period throughout the country, and (2) the consideration of structural fragility or probability of collapse that has a statistical distribution. For example, in San Francisco, where the 100-year return period accelerations are almost as high as the 2,500-year (MCE) accelerations, the risk of collapse, when considering the full distribution of capacity (i.e., fragility), is larger than in Memphis, where the 100-year accelerations are significantly smaller than the 2,500-year accelerations (Luco et al. 2007). Although reasons for the changes in spectral acceleration levels are apparent in Luco et al., it is not clear whether similar changes to the basic hazard level would be present for the 1,000-year event, which AASHTO uses.

The return period for ground motions producing a uniform risk will not correspond to a single value across the country. At sites where earthquakes occur relatively frequently, such as California, the return period generally will be less than at sites where earthquakes occur infrequently. The ASCE 7-10 MCER shaking approach produces shaking values that are approximately, but not exactly, equal to 2,500 years.

The change from uniform hazard to uniform risk represents a shift away from simple quantification of earthquake damage in terms of EDP and code-implied performance toward more complete performance-based design that calculates chances of DMs occurring. This approach attempts, at a notional level, to combine the first three of the four steps of PBSD: seismic hazard, structural response, and damage prediction.

The simplification of using a generic probability of collapse may provide a somewhat inaccurate indicator of the actual risk of collapse compared with the risk that would be calculated by a rigorous analysis based on system-specific fragilities. However, the method does pick up general trends related to regional ground motion differences. The ASCE 7-10 design methodology is still predicated on a force-based methodology, and thus, there is no direct rationalization of the methodology with respect to DM prediction for actual structures designed using the specifications. However, ASCE 43-05, *Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities* (2005a), also uses a risk-targeted seismic hazard, and that standard does include displacement-based methodologies, although they are not outlined in detail. Nonetheless, it is clear that the building profession is incrementally migrating toward probabilistic PBSD.

International Code Council Performance Code

The *International Building Code* (IBC), and its adjunct ASCE 7-10, are force-based and prescriptive and, as such, do not contain performance-based provisions. However, the International Code Council (ICC) publishes a performance-based document, *International Code Council Performance Code* (ICC-PC). The ICC-PC allows the user to achieve various design solutions and is intended to envelop the single solution obtained using the basic IBC. To that extent, the IBC is considered to provide an acceptable solution that will comply with the ICC-PC, thus making the ICC-PC the higher-level document. This is a model code that must be adopted or otherwise permitted by a jurisdiction before it may be used on a project. The ICC-PC addresses seismic loading in addition to considering performance-based design for other loadings and performance types (i.e., nonstructural).

The ICC-PC uses four performance groups (PGs) and four design events, as shown in Table 4. The performance groups are:

- PG I—agricultural, temporary and minor storage facilities
- PG II—facilities other than I, III, or IV
- PG III—buildings representing substantial hazard to human life (assembly of more than 300 people in one area, schools, health care, power facilities, occupancy of more than 5,000 total, etc.)
- PG IV—essential facilities (hospitals, fire, rescue, police, emergency shelters, air traffic control towers, buildings with critical national defense functions, etc.).

TABLE 4
ICC-PC MODEL CODE

Increasing Magnitude of Event / MAGNITUDE OF DESIGN EVENT	PERFORMANCE GROUPS (Increasing Level of Performance →): PG I	PG II	PG III	PG IV
Very Large (Very rare – 2,475-year return period)	Severe	Severe	High	Moderate
Large (Rare – 475-year return period)	Severe	High	Moderate	Mild
Medium (Less frequent – 72-year return period)	High	Moderate	Mild	Mild
Small (Frequent – 25-year return period)	Moderate	Mild	Mild	Mild

These performance groups are essentially the same as those in the IBC and ASCE 7. The performance group, combined with four design events, is then used to define the maximum level of damage that can be tolerated, as shown in Table 4. Four damage levels are defined in terms of structural damage, nonstructural system damage, occupant hazards, overall extent of damage, and hazardous materials:

- Mild—No structural damage and the building is safe to occupy.
- Moderate—Moderate structural damage, which is repairable, and some delay to reoccupancy can be expected.
- High—Significant damage to structural elements, but no large falling debris occurs; repair is possible, but significant delays in reoccupancy can be expected.
- Severe—Substantial structural damage, but all significant components continue to carry gravity load demands; repair may not be technically possible; the building or facility is not safe to reoccupy, as reoccupancy may cause collapse.

The owner and the principal design professional (PDP) have the responsibility to develop performance criteria, have them peer reviewed, and gain approval by the code official. The design reports must document required elements such as the goals and objectives of the project, performance criteria, bounding conditions (restrictions on use to ensure the desired performance is achieved) and critical design assumptions, system design and operation requirements, and operational and maintenance requirements. It is the responsibility of the owner and PDP to develop a plan that will meet the required performance requirements. Prescriptive limits are not provided in the ICC-PC, and this approach puts significant responsibility on the PDP to satisfy the performance objectives over the life of the facility. This may constitute a significant, practical shortcoming of the performance code approach, as illustrated by the New Zealand "leaky home" case study described in chapter two.

MARINE INDUSTRY PRACTICE

Deterministic PBSD, using the first two steps of PBSD, has been used extensively in the marine industry for well over a decade, primarily because of the development of the Marine Oil Terminal Engineering and Maintenance Standards (MOTEMS 2011) and the Port of Los Angeles/Port of Long Beach (POLA/POLB 2009) seismic design guidelines. In general, marine PBSD uses a multiperformance-level approach that limits damage and downtime in small to moderate earthquakes, and prevents structural collapse and loss of life in large seismic events. The overall design philosophy and many of the provisions and recommendations have been adopted from the bridge industry, with a heavy influence from Priestley et al. (1996). Both the MOTEMS and POLA/POLB guidelines are summarized.

MOTEMS

California has developed guidelines for design and maintenance for marine oil terminals (MOTs). Included in these provisions (adopted as Chapter 31F of the California Building Code) are multilevel performance-based seismic design requirements. These provisions have become standard practice for many waterfront structures other than MOTs, including piers and wharves. Seismic performance is characterized using two performance levels.

The return period for a Level 1 or Level 2 seismic hazard is defined based on the quantity of oil the terminal processes, the number of transfers per year per berthing system, and the maximum vessel size. However, all new MOTs are classified as high risk, with return periods of 72 years and 475 years, respectively, for Levels 1 and 2.

- Performance for Level 1:
  - Minor or no structural damage
  - Temporary or no interruption of operation.
- Performance for Level 2:
  - Controlled inelastic structural behavior with reparable damage
  - Prevention of structural collapse
  - Temporary loss of operations, restorable within months
  - Prevention of a major oil spill.

Performance is quantified using material strain limits and nonlinear static capacity (pushover) curves. Strain limits are material and location specific (i.e., strain limits associated with pile in-ground plastic hinging are more restrictive than strain limits in the pile-to-deck connection). Demand is determined through a two-dimensional (2-D) nonlinear static demand procedure (simplified or refined) based on recommendations by Priestley et al. (1996) or the FEMA 440 modifications of the ATC-40 Capacity Spectrum Method, which is described in chapter five of this synthesis. Additionally, if the structural configuration is irregular, three dimensional (3-D) linear modal procedures are required. Nonlinear dynamic analyses are optional, and capacity protection is achieved through methods similar to those used in bridge design.

Port of Los Angeles/Port of Long Beach

The Port of Los Angeles and the Port of Long Beach each have developed their own seismic design guidelines for waterfront structures, with a primary emphasis on marginal wharves (parallel to shore). Both guidelines are similar to each other and to the MOTEMS approach. Seismic performance is characterized using performance-based procedures with three performance levels, an operating level earthquake (OLE) with a 72-year return period, a contingency level earthquake (CLE) with a 475-year return period, and a design earthquake (DE), which represents two-thirds of the maximum considered earthquake (MCE). The DE ground motion is associated with meeting the minimum requirements of the 2007 California Building Code, which invokes ASCE 7-05 (ASCE 2005a). Thus, the DE is meant to be a direct check of life safety consistent with ASCE 7-05 and was added to demonstrate minimum compliance with ASCE 7-05 to the building official. Seismic performance for each performance level is defined by operability, reparability, and safety concerns, as follows:

- Performance for the OLE
  - No interruption of operations
  - Forces and deformations (including permanent embankment deformations) shall not result in structural damage
  - All damage shall be cosmetic in nature and located where visually observable and accessible
  - Repair shall not interfere with wharf operations.
- Performance for the CLE
  - Temporary loss of operations of less than 2 months is acceptable
  - Forces and deformations (including permanent embankment deformations) may result in controlled inelastic behavior and limited permanent deformations
  - All damage shall be repairable and shall be located where visually observable and accessible for repairs.
- Performance for the DE
  - Forces and deformations (including permanent embankment deformations) shall not result in structural collapse of the wharf, and the wharf shall be able to support the dead load of the structure including cranes
  - Life safety shall be maintained.

Seismic performance is primarily quantified using strain limits for each level of ground motion. Strain limits are material and location dependent. Structural capacity is generated using nonlinear static analyses. Demand is determined primarily using the substitute structure method of a 2-D segment of the structure. However, if the wharf is considered irregular, 3-D modal response spectra or linear response history analysis may be employed, and nonlinear response history analyses may be used to verify seismic displacement demands. Capacity protection is enforced for all elements except the piles, where a strong deck-weak-pile philosophy is used. As with other first-generation performance-based standards, seismic hazard analysis and structural analysis are well defined, but damage analysis and loss analysis are not.

CHAPTER FOUR

SEISMIC HAZARD ANALYSIS

As indicated in chapter one, PBSD requires an estimate of the seismic hazard for the site in question. This hazard includes effects of both the regional tectonics and the local site characteristics. From either a deterministic or probabilistic viewpoint, this design step is perhaps the best developed. Deterministic seismic hazard analysis was the first methodology developed; it comprises four steps (Reiter 1990):

1. Identification of the seismic sources that affect a site
2. Selection of a source-to-site distance parameter, often taken as the closest approach to the site and
3. either hypocentral or epicentral distance, along with an attenuation relationship that defines hazard as a function of distance to the site from the source
4. Selection of the controlling earthquake—the earthquake that could produce the largest shaking
5. Generation of the hazard at the site (often ground acceleration or spectral acceleration) using the source-to-site attenuation relationship.

The deterministic form allows one to assess the shaking at a site as a function of the controlling earthquake that can occur on all the identified faults or sources.

The methodology was extended to a probabilistic basis by Cornell (1968) and first mapped for the country by Algermisson et al. (1982). This approach, with subsequent refinements, still forms the basis for the seismic maps used with the AASHTO specifications and the various building codes.

With probabilistic seismic hazard analysis, the individual steps listed previously are each put on a probabilistic basis that takes into account uncertainties in the occurrence of earthquakes on any sources affecting a site, in the attenuation relationships for a local region, and in the local seismic settings. These relationships are then combined to form the probability that a given acceleration may be exceeded during a given window of time (e.g., a 7% chance of exceedance in 75 years). Alternatively, the probability may be related in terms of an average return period for a given value of seismic hazard (e.g., ground acceleration or spectral acceleration).

Today seismic hazard is mapped by the U.S. Geological Survey (USGS) and is provided on a gridded basis for the entire country and territories. The hazard data that USGS provides are for firm soil or rock. The site seismic hazard is then built considering site effects. This process may use the so-called National Earthquake Hazards Reduction Program (NEHRP) site classification factors that form the basis of the AASHTO methods or may use a site-specific study to develop the accelerations at the ground surface. USGS is continually improving the source, uncertainty, and attenuation data for seismic hazard and issues updated geographic acceleration data on a several-year cycle. The process of developing site hazard in this manner is clearly defined in the AASHTO specifications and their associated commentary.

The USGS data that AASHTO currently uses in its tools to assist engineers in determining site hazard are normally in the form of uniform hazard spectra. These then relate structural response (spectral acceleration) in a form that has a uniform chance of being exceeded in some window of time. For AASHTO, this window is nominally 1,000 years (actually 975 years), or approximately a 7% chance of exceedance in 75 years.

Seismic hazard is generally included in design using three parameters: site-adjusted peak ground acceleration, spectral acceleration at 0.2 seconds, and spectral acceleration at 1.0 second. These values are normally used to determine the shape of a design response spectrum for the site, or they could be used with site-specific response spectra to define a unique seismic input for each site. The former is more common, especially with conventional bridges.

When considering PBSD, the concept of using more than one return period of seismic hazard is often included. For instance, this might result in use of both a 100-year return period and 1,000-year return period for the design checks of a bridge. The performance in each event might be different, with operational performance for the more frequent event and avoidance of collapse for the larger event. Such an approach is rational in terms of providing higher protection and less risk in the more frequent event. Then, significant damage and potential loss of service are considered for the larger event. However, it is important to recognize that the relationship between the acceleration levels for two events is not the same across the country. This is illustrated in Figure 7, excerpted from the Multidisciplinary Center for Earthquake Engineering Research MCEER/ATC-49 reports (2003). In the figure, the ratios of the 1-second spectral acceleration for return periods higher than 475 years, up to 2,475 years, are shown for the central and eastern United States (CEUS), the western United States (WUS), and California. Ratios are provided for some 24 locations across the United States, including Alaska and Hawaii. In the CEUS, the ratio is over 5 for Memphis, Tennessee, and Charleston, South Carolina, at 2,475 years. This value is over 2 at the 1,000-year return period level. By contrast, the ratio never exceeds about 1.6 in California, even for long return periods. Other areas in the WUS have intermediate values from 1.5 to 3.5 at 2,475 years and 1.5 to 2.2 for 1,000 years.

The trends are similar for the short-period spectral accelerations (0.2 seconds), and for the data more recently developed by USGS. Clearly, the hazard of more frequent earthquakes relative to rare earthquakes is different across the country, and this difference may affect the approach used if multiple-level earthquakes are used for PBSD of bridges.

In recognition that the seismic hazard varies significantly between frequent and rare events across the United

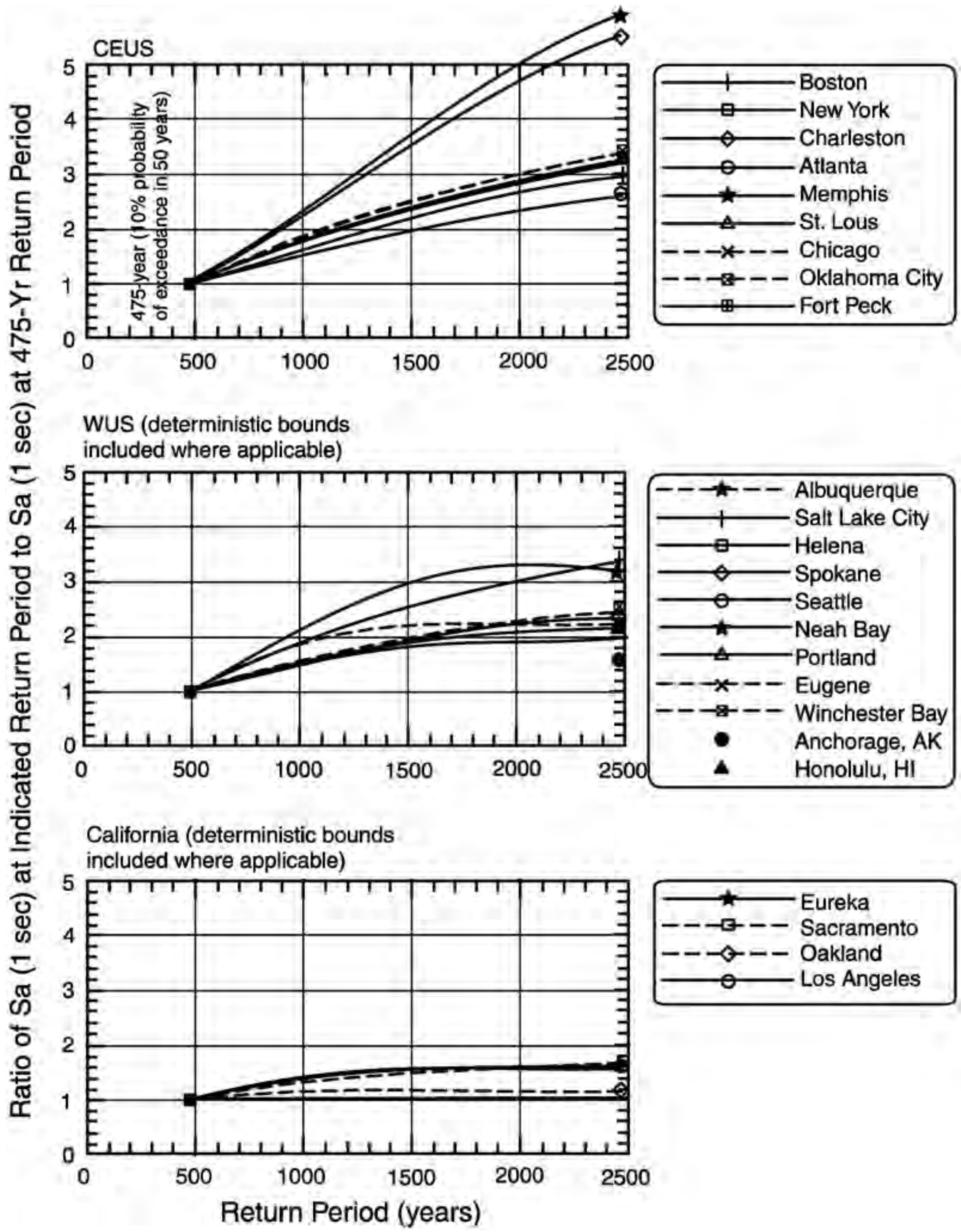


FIGURE 7 Ratios of long-period spectral acceleration at various return periods to the long-period spectral acceleration at 475 years (after FHWA 2006 and MCEER/ATC 2003).

States, and that the risk of collapse or attainment of other damage levels is not uniform across the county, the ASCE 43-05 *Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities* standard (2005b) presents a method to adjust design spectra to account for the uneven hazard. The commentary of the ASCE 43-05 standard has one of the better descriptions of the approach. This methodology has also been picked up in a slightly modified form in the ASCE 7-10 (2010) standard, as described previously.

Typically, as PBSD is taken into the full probabilistic format, median predictors are desired for all parameters that are used. In these median predictors, factors that provide the dispersion around the median value are used to bring in the probabilistic nature of the parameters. With uniform hazard spectra, often no single earthquake can produce the shaking indicated by the uniform hazard design spectra. To address the statistical likelihood that accelerations all along the spectrum cannot be created by a single earthquake, the conditional mean spectrum (CMS) has been developed. In the CMS, a single period of vibration is selected; a statistical spectrum that would be associated with the control period is then developed. These spectra are typically lower than uniform hazard spectra, and the CMS is often used in conjunction with the fully probabilistic PBSD process. This is different than the seismic hazard process that AASHTO is currently using.

CHAPTER FIVE

STRUCTURAL ANALYSIS AND DESIGN

During the PBSD process, it is critical to accurately predict the structural response to earthquake ground motions. Depending on the geometry of the system and the extent of inelastic behavior, various methods of increasing complexity and refinement have been developed. In general, structural analysis for earthquake forces can be broken into four categories, as shown in Table 5. Fundamental to three of the analysis methods is some form of simplification from the most general and powerful, but time-consuming and complex methodology—full nonlinear dynamic response history analysis.

TABLE 5
TYPES OF STRUCTURAL ANALYSIS

	Linear	Nonlinear
Static	• Equivalent lateral force procedure	• Pushover analyses (e.g., coefficient method, equivalent linearization)
Dynamic	• Modal response spectrum • Linear response history	• Inelastic response spectrum • Nonlinear response history analysis

The differences between methods are contingent upon the explicit treatment of inelasticity and dynamic behavior. Many of the current PBSD procedures use a combination of techniques from more than one analytical category. For example, the AASHTO SGS uses a linear dynamic analysis adjusted by scalar multipliers to determine the estimated displacement demand that an earthquake might place upon the bridge, while a nonlinear static pushover analysis is used to determine the displacement capacity of either the individual piers or the bridge as a whole. Many analysis and design techniques exist in various stages of development and implementation, and detailed explanation of all possible options is beyond the scope of this synthesis. However, this chapter presents several of the more common techniques.

There are excellent resources explaining in detail the dynamic response of structures to seismic excitation (Clough and Penzien 1975; Chopra 2007; Villaverde 2009), which provide the theoretical basis for all of the analytical categories.

LINEAR STATIC PROCEDURE

The most basic analytical procedure uses equivalent static lateral forces imposed on the structure in a predefined load pattern that represents the distribution of inertial forces imposed by earthquake shaking. This method is well suited to simple, regular structures that are dominated by their first mode response. The method predicts linear elastic response, and inelastic effects must be handled separately. For example, in design the performance and allowed inelasticity may be treated implicitly using R factors, which reduce the design forces as a function of the bridge importance and the structural systems ability to withstand plastic deformations. This method has been adopted into the AASHTO LRFD, but is limited in its treatment of inelasticity and structural complexity. A brief discussion of R-factors can be found in the Current AASHTO Practice section of chapter three. In general, the equivalent lateral force procedure loses accuracy for structures where higher mode effects are significant, as in long-period structures, and where geometric irregularities or sharp discontinuities or asymmetry are present, as these breach the basic assumptions on which the method is founded. This analysis method is poorly suited for PBSD as it does not explicitly quantify bridge performance.

LINEAR DYNAMIC PROCEDURES

If a structure displays dynamically complex behavior that cannot be captured by the equivalent lateral force procedure, then modal response spectrum analysis (RSA) can be used. In this linear elastic procedure, a modal analysis of the structure is conducted to determine the deformed shape and natural frequency of all pertinent modes of vibration, typically including a sufficient number of modes to capture at least 90% mass participation in each orthogonal direction of displacement. A design response (acceleration) spectrum is then used to determine the magnitude of each modal response (e.g., displacements, shears, moments) based on the participation factor, response, and damping for each mode.

The maximum modal responses are then determined using a modal combination rule such as the square root of the sum of the squares (SRSS) or the complete quadratic combination (CQC) rules, depending on the spacing of the natural frequencies. These combination methods are intended to provide estimates of response appropriately built up from individual elastic mode response. Modal RSAs are described extensively in structural dynamics texts and will not be further explained here.

This method provides an accurate prediction of the elastic dynamic response of simple and complex structures and has the added benefit of being relatively easy and quick to perform with modern software. Additionally, the method uses the same modeling assumptions that are typically employed for nonseismic loading, with which structural engineers are familiar. On their own, modal RSAs cannot predict inelastic displacement demands, plastic deformation capacity of the system, or accurate internal force fields if yielding is expected to develop. This method is therefore suitable for PBSD only if the structure is expected to remain nearly elastic under the ground motion of interest, or if the method is only used to determine displacement demands (as with the coefficient method described subsequently), or for analytical methods used to assess or check the inelastic response of a structure. This is how linear dynamic procedures are used in the AASHTO SGS.

To further refine the accuracy of linear elastic analysis, step-by-step time integration or linear response history analysis methods can be used. These methods solve for the response of the structural system in the time domain by analytically subjecting the structure to an earthquake acceleration record. This solution is accomplished by numerically using a series of small time intervals and integrating the incremental equations of motion for each time step based on the loading function for that time step and the state of the system from the previous time step. With small enough time steps, the solution will converge on the exact solution of the equation of motion. Because the response is solved explicitly, the assumption that the system mode shapes are orthogonal to the damping matrix, and the need for approximate modal combination rules to determine maximum displacements, are unnecessary (Villaverde 2009). Despite these advantages, linear response history analyses are rarely performed in design practice and are only accurate if the structure remains essentially elastic under the selected ground motion.

NONLINEAR STATIC PROCEDURES

Whenever inelastic behavior is expected, linear static and linear dynamic analyses cannot reliably be used to precisely quantify structural performance; therefore, nonlinear methods must be employed. The simplest methods are the nonlinear static procedures, which use what are known colloquially as "pushover" analyses, to evaluate the nonlinear deformation capacity of the system and explicitly quantify the redistribution of internal forces as a result of yielding elements. In this technique, an assumed force or displacement field (typically corresponding to the first mode deformed shape) is applied to the structure. The magnitude of the applied force or displacement field is monotonically increased until failure or another applicable limit state is observed. A trace of the deformation versus base shear is recorded in force–displacement space. This response is known as the pushover or capacity curve.

The maximum global displacement demands resulting from the seismic hazard are then predicted using simplified, approximate procedures that relate the inelastic response of a single degree of freedom (SDOF) oscillator to the elastic response of an infinitely strong elastic system. These procedures are ideally calibrated using nonlinear response history analyses. The global displacement demand is then compared with the global displacement capacity, as determined from the pushover curve, to determine the adequacy of the design. From the global displacement, member drifts, forces, and component actions can be determined, leading to the final design of the structure. Figure 8 is a schematic representation of nonlinear static procedures.

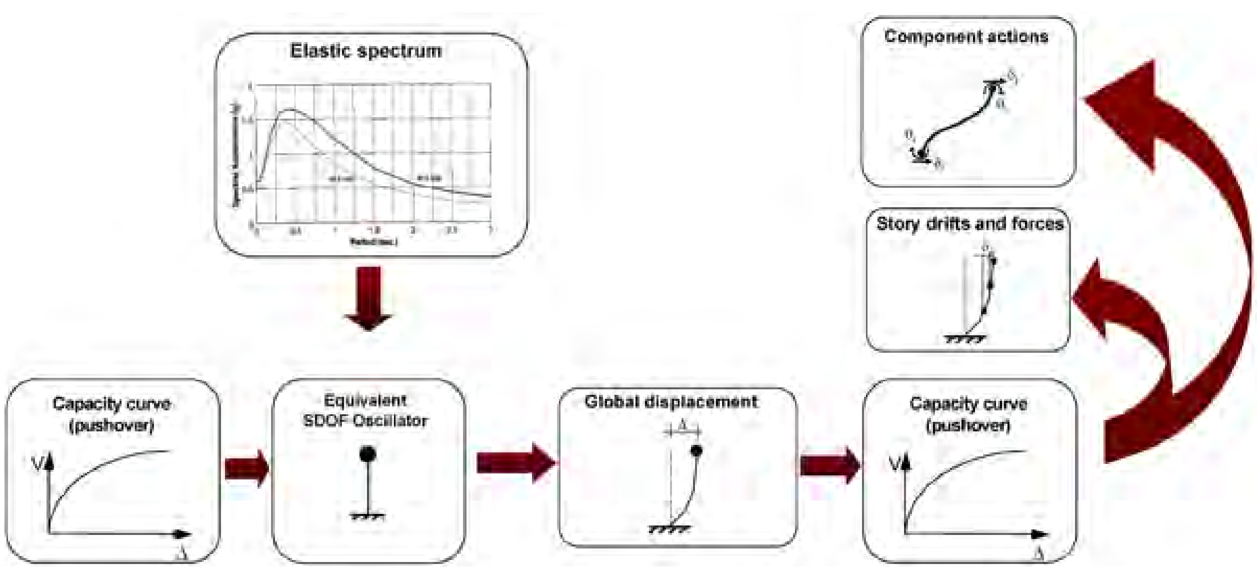


FIGURE 8 Nonlinear static procedures (FEMA 440).

The benefits of this method are increased insight into inelastic behavior of a structure, including the location and formation sequence of plastic hinges, relative simplicity, and accuracy for simple structures, especially those that can be well represented as an SDOF system. This makes nonlinear static methods attractive for use in PBSD methodologies. The basic limitations of these procedures are that elastic modal properties are used to compute the inelastic system parameters, and it is assumed that the response of the structure is controlled by a single mode (usually the first mode). Therefore, the contribution of the other modes (usually higher modes) may not be explicitly considered.

In general, two primary manifestations of the nonlinear static procedure are commonly used in design practice: the coefficient method and the equivalent linearization method. If they are well calibrated against nonlinear response history analyses, these methods can provide reasonably accurate predictions of seismic behavior and are well suited to performance-based design of ordinary structures. A comparative evaluation of the two methods is provided in Miranda and Ruiz-Garcia (2002).

Coefficient Method

The coefficient, or displacement modification method, builds on the linear modal response spectrum procedure described previously. Because the modal response spectrum analysis is a linear-elastic method, inelastic behavior and damage cannot be captured explicitly. However, the inelastic displacement demand on a structure can be approximated if the so-called "equal displacement assumption" is imposed. This states that the maximum lateral displacement of a nonlinear structural system is approximately equal to the maximum displacement of the same system behaving elastically with unlimited strength, as shown in Figure 9. In other words, the yielding of the system does not affect the maximum displacement experienced during a ground motion. This assumption is only valid for medium- to long-period structures with minimal strength and stiffness degradation, and insignificant P-Δ effects. If these requirements are not met, scalar coefficients have been developed and calibrated to modify the predicted displacement. The coefficients are derived empirically from nonlinear response history analyses of SDOF oscillators with varying periods, strengths, and hysteretic shapes.

The displacement capacity of the inelastic structural system is determined by the application of strain, rotation, drift, or displacement limits to the pushover response. The location of this displacement demand relative to the displacement capacity indicates whether the performance objective has been met. Simply stated, the displacement demand is determined using a modified response spectrum analysis, while the displacement capacity is determined using a pushover analysis. This displacement is shown schematically in Figure 10. This method has been developed by Newmark and Hall (1982), Miranda (2000), and Chenouda and Ayooub (2008), among others, and has been implemented by the AASHTO SGS, FEMA 440, ASCE 7, ASCE 41, and FEMA 356.

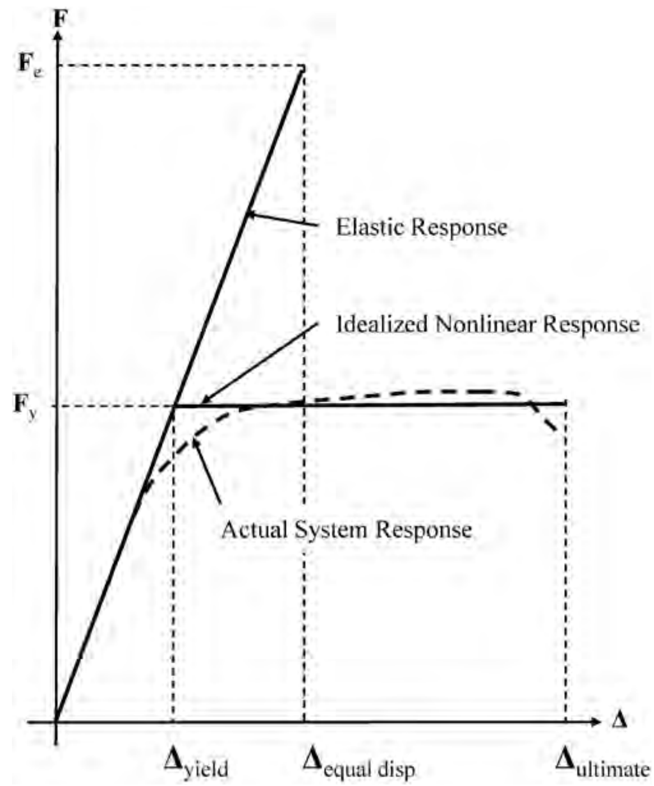


FIGURE 9 Linear and nonlinear approximations of structural response.

Equivalent Linearization Method

Another common nonlinear static analysis method is the equivalent linearization method, also known as the secant stiffness or substitute structure method. In this procedure, the nonlinear system is replaced with an equivalent linear system with an effective period defined by the secant stiffness of the nonlinear system at the displacement demand. Hysteretic energy dissipation is accounted for by equivalent viscous damping determined from the ductility and hysteretic response of the nonlinear system. The global inelastic displacement of the system is calculated as the maximum displacement of the equivalent linear SDOF oscillator. This method has been implemented by Rosenblueth and Herrera (1964), Gulkan and Sozen (1974), Shibata and Sozen (1976), Kowalsky (1994), and Iwan (1980), among others.

A convenient graphical representation of equivalent linearization is the capacity spectrum method of ATC-40, which has been further refined by FEMA 440 to incorporate various hysteretic properties, such as strength and stiffness degradation. In this method, the elastic response spectrum and the pushover capacity curve are converted into spectral ordinates (spectral displacement versus spectral accelera-

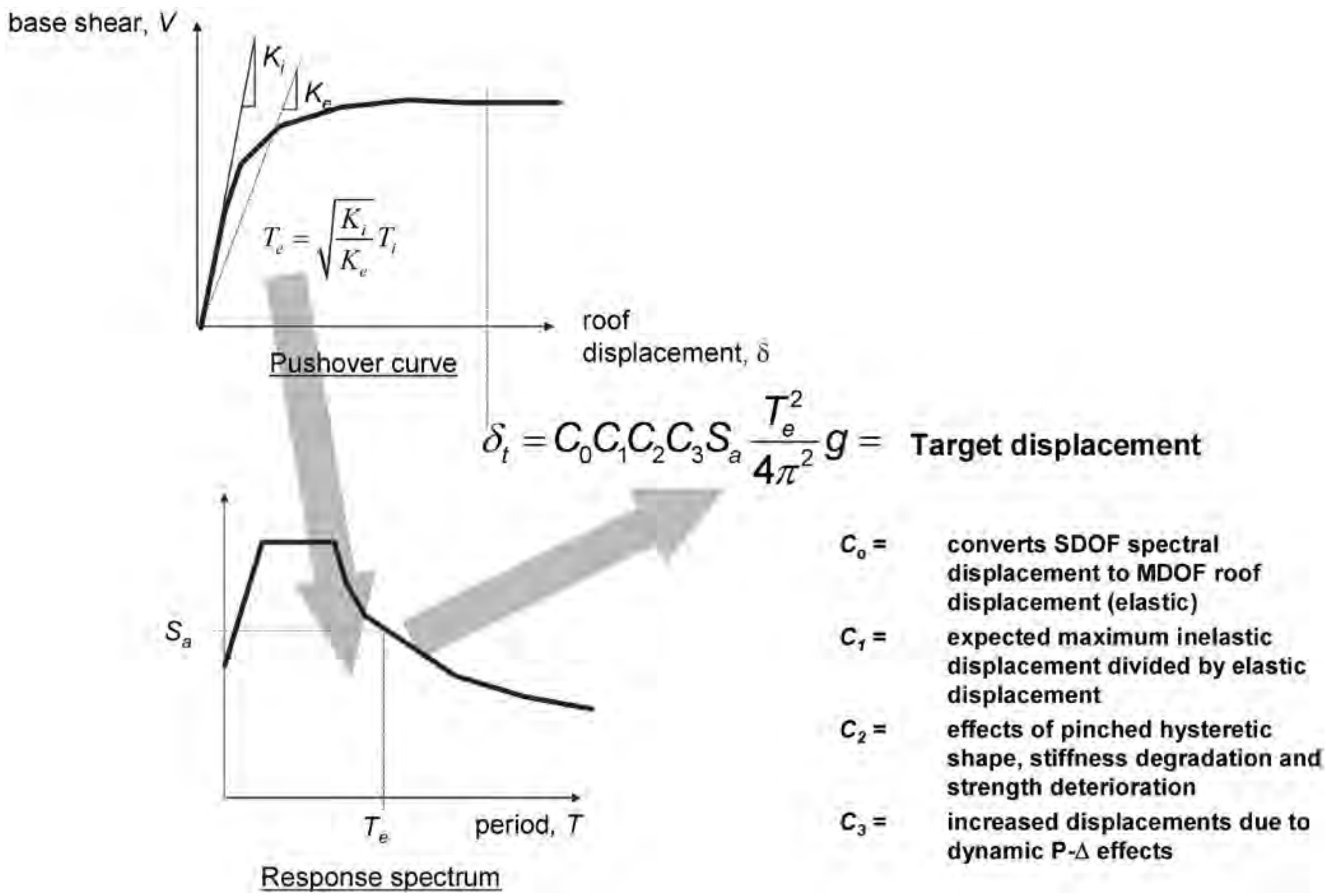


FIGURE 10 FEMA 356 Coefficient Method (FEMA 440).

tion). The demand spectrum is modified to account for the equivalent viscous damping generated by hysteretic energy dissipation. The intersection between the demand and capacity spectra when their equivalent viscous damping terms are equal represents the "performance point" or the displacement demand, which is shown in Figure 11. Equivalent linearization methods are permitted in the *AASHTO Guide Specifications for Isolation Design* (2010a), the AASHTO SGS, and the provisions of ASCE 7 for seismically isolated structures and structures with damping systems.

Another common use of equivalent linearization is the direct displacement based design (DDBD) procedure developed and advocated by Priestley et al. (1996, 2007). This method uses the same fundamental equivalent linearization concepts but employs an elastic displacement spectrum, which is modified by the equivalent viscous damping to account for hysteretic energy dissipation. Specific equivalent viscous damping formulas have been calibrated for various hysteretic shapes (e.g., bilinear, Takeda, flag shaped), as well as various soil types for soil-foundation interaction of piles and drilled shafts (Priestley et al. 2007).

As with the coefficient method, the system performance is determined based on the location of the displacement demand in relation to the displacement capacity. The system displacement capacity is determined using strain, rotation, drift, or displacement limits defined according to the required performance criteria.

MULTIMODAL NONLINEAR STATIC PROCEDURES

In some structures, an SDOF representation is inadequate, as neglecting higher mode effects will produce an inaccurate and often unconservative prediction of response. Because of this, several multimodal pushover procedures have been developed, in particular for tall buildings, but the concepts are potentially applicable to bridge systems sensitive to higher mode response.

Modal Pushover Analysis

In this procedure, pushover analyses are conducted independently in each mode, using lateral force profiles that represent the response in each of the first several modes. This procedure is performed even though the response in each mode may be nonlinear, whereby the mode shapes and lateral force profiles are assumed to be invariant as the inelastic mode shapes are only weakly coupled. Response values are determined at the

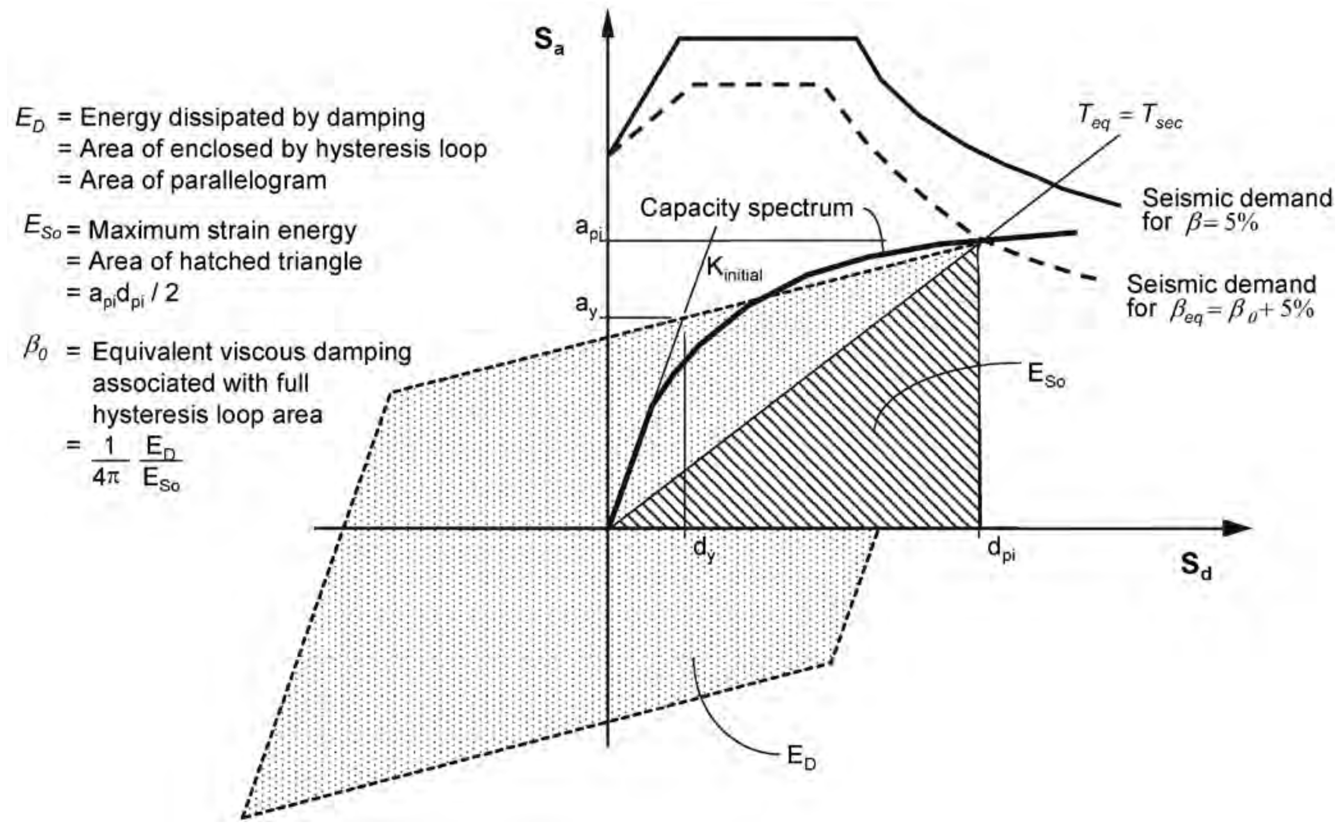


FIGURE 11 ATC-40 Capacity Spectrum Method (FEMA 440).

target displacement associated with each modal pushover analysis. The target displacement values may be computed by applying the coefficient method or equivalent linearization procedures to an elastic spectrum for an equivalent SDOF system representative of each mode being considered. Response quantities obtained from each modal pushover are normally combined using the SRSS method. Modal pushover analyses are detailed in Chopra (2007).

Limitations of this procedure are that (1) elastic modal properties are used to compute the inelastic system parameters and (2) the displacements are approximated from the maximum deformation of an SDOF system for all the modes.

Adaptive Modal Combination Procedure

> [This] methodology offers a direct multi-modal technique to estimate seismic demands and attempts to integrate concepts built into the capacity spectrum method recommended in ATC-40 (1996), the adaptive method originally proposed by Gupta and Kunnath (2000), and the modal pushover analysis advocated by Chopra and Goel (2002). The AMC [adaptive modal combination] procedure accounts for higher mode effects by combining the response of individual modal pushover analyses and incorporates the effects of varying dynamic characteristics during the inelastic response via its adaptive feature. The applied lateral forces used in the progressive pushover analysis are based on instantaneous inertial force distributions across the height of the building for each mode. A novel feature of the procedure is that the target displacement is estimated and updated dynamically during the analysis by incorporating energy-based modal capacity curves in conjunction with constant-ductility capacity spectra. Hence it eliminates the need to approximate the target displacement prior to commencing the pushover analysis (Naiem 2001).

The primary feature of adaptive schemes is the updating of the applied story forces with respect to progressive changes in the modal properties at each step. This allows progressive system degradation resulting from inelastic deformations to be represented more realistically in a static framework. The method is described in Kalkan and Kunnath (2006).

The use of these more elaborate modal analysis techniques may or may not be easily, or economically, adapted to bridge design, as the work in this area has been primarily directed toward building response.

NONLINEAR DYNAMIC PROCEDURES

Although the aforementioned nonlinear static methods aim to define inelastic displacement demands using a simplified framework, some situations warrant the use of nonlinear dynamic or nonlinear response history analytical procedures, often called the "time-history" method. This method is an extension of linear response history analysis, with nonlinear material and geometric behavior explicitly accounted for in the equations of motion. In this most general of all procedures,

> it is necessary to (a) write the equations of motion in an incremental form, (b) make the assumption that the properties of the structure remain unchanged during any given time interval [or iterate the interval], (c) solve the equations of motion for such a time interval considering that the structure behaves linearly, and (d) reformulate its properties on the basis of the obtained solution at the end of the interval to conform to the state of stresses and deformations at that time (Villaverde 2009).

The solution within each time step is iterated using various corrective solution algorithms, which reduce the error associated with (1) the assumption that system properties are constant during a time step and (2) the sudden change in system properties resulting from yielding or unloading during the time step. The details of nonlinear dynamic solution procedures, algorithms, and modeling guidelines are well documented elsewhere (Chopra 2006; Priestley et al. 2007; Villaverde 2009).

These analysis procedures have the fewest limitations, as both the transient dynamic and inelastic responses are solved for explicitly. However, these are also the most time-consuming and difficult analyses to perform, troubleshoot, and interpret, requiring skilled analysts. Additionally, to obtain usable results, multiple ground motions must be selected. Typically, design forces and deformations are taken as the maximum structural actions if only three ground motions are used or taken as the average if seven or more ground motions are used. In most cases, each ground motion will consist of the two orthogonal horizontal components of shaking, and often the vertical component of shaking is incorporated as well.

Some of the challenges associated with conducting nonlinear response history analyses are the selection and scaling of the input ground motions (NEHRP 2011), the calibration and validation of element hysteretic response, the treatment of elastic damping (Charney 2008), and the computational hurdles of convergence, run time, and postprocessing. In many cases, the use of nonlinear response history analyses is simply too costly and time prohibitive for all but high-profile or critical structures.

There is, however, a simplification of the nonlinear response history analyses, known as the constant ductility or inelastic response spectrum. This method is the nonlinear extension of an elastic response spectrum. Instead of recording the maximum response from an elastic SDOF oscillator to a suite of one or more ground motions, the constant ductility spectrum records the maximum response of a nonlinear SDOF oscillator at specific system ductilities. The nonlinear SDOF oscillator has an assumed hysteretic behavior that sufficiently represents the nonlinear cyclic force-deformation response of the system. This methodology is well documented (Newmark and Hall 1982; Krawinkler and Nassar 1992; Han et al. 1999; Chopra and Goel 1999; Fajfar 1999; Chopra 2007) and can provide a direct and efficient analytical method for structures that can be represented as SDOF systems. This method can also use a graphical solution procedure similar to the capacity spectrum method described earlier (see Chopra 2006).

MODELING OF NONLINEAR SYSTEMS

The techniques and assumptions required to accurately model a nonlinear structural system are dependent on the system characteristics and configuration, the intended analytical procedure, the computer software available, the level of accuracy needed, and the time available to perform the analysis. Therefore, detailed modeling recommendations are beyond the scope of this synthesis. Modeling guidelines are typically given in the literature describing particular analytical methodologies. It can be said, however, that for all structural modeling, the "analysis should be as simple as possible, but no simpler," according to Einstein's maxim. This requires the judicious use of modeling assumptions so unintended errors or gross oversimplifications are avoided. There are many excellent resources describing modeling for nonlinear structural analysis, including (Filippou and Issa 1988; Filippou et al. 1992; Priestley et al. 1996; Berry and Eberhard 2007; Priestley et al. 2007; Aviram et al. 2008; FIB 2008; Dierlein et al. 2010).

The importance of accurate modeling of soil-structure interaction, abutment restraint, and movement joint effects must be emphasized here, as these effects can greatly change the predicted response. The details of the modeling approaches depend on the type of foundation: typically either shallow-spread footings or deep foundations (extended shafts, pile columns, pile bents, or pile groups). In most cases, spread footings and pile groups are capacity protected and are modeled using linear and/or nonlinear springs to capture footing rotation and translation. Analytical methodologies are well documented (e.g., AASHTO SGS, FHWA *Retrofitting Manual*). However, in the case of extended shafts and pile bents/columns, inelastic action is often allowed to occur below ground. Including this inelastic response usually requires more sophisticated modeling and direct incorporation of the soil response into the solution. These methods are well defined in the literature (Budek 1995; Priestley et al. 1996; Budek 1997; Boulanger et al. 1999; Chai 2004; Song 2005; Suarez and Kowalsky 2006a and 2006b; Blandon 2007; Priestley et al. 2007; Goel 2010), with several closed-form approximations being developed.

As with foundations, abutment and movement joint type can also have a significant influence on the dynamic response of bridges. Modeling guidelines and recommendations can be found in ATC (1996), Priestley et al. (1996), FHWA (2006), Aviram et al. (2008), and Shamsabadi et al. (2010). Simplified models and analytical techniques for predicting movement joint response are difficult to define and have relatively high levels of inherent uncertainty. One such method that has gained considerable favor in the design community for its simplicity is to use a tension and a compression model of the structure to capture the response when the joint is fully open (the tension model) and fully closed (the compression model). The design actions are then taken as the envelope of member action from the tension and the compression model. While this appears to ensure adequate performance in strong ground motions movement, joint response is certainly an area warranting additional research.

UNCERTAINTY IN NONLINEAR ANALYSIS METHODS

Within the realm of nonlinear structural analysis for seismic design there is a constant struggle to balance simplicity and transparency of an analytical procedure with its ability to accurately predict often complex structural behavior. As with any simplification or approximation errors can be introduced. However, as long as the methods produce acceptable designs (i.e., not unconservative/unsafe or overly conservative/unnecessarily expensive), then errors or bias can be tolerated. Ideally, structural analysis could be simplified without introducing significant error or bias into the results/design; however, this is often not the case. Therefore, a considerable amount of research is currently being conducted to refine and simplify nonlinear analysis and design techniques. To be able to reliably apply PBSD in the full sense, the uncertainties in the structural analysis must be understood and quantified. Clearly delineating and considering such uncertainties is a significant challenge.

It is of interest to examine the analytical methods described previously in relation to the relative uncertainty that is introduced into the solution solely by the assumptions inherent in an analytical technique. An increase in relative uncertainty is represented in Table 6 as a darker shade of grey. It should be noted, however, that uncertainty classified here as "high" is still within acceptable limits for usage within PBSD, especially given the uncertainty inherent in the ground motion. The vertical columns represent the two primary ground motion input characterizations, elastic response spectra and individual ground motion records. The rows describe the refinement of the structural model; "Detailed" represents a complex 2-D or 3-D model with refined element boundary conditions and associated nonlinear behavior. This level of refinement is typically reserved for academic research or critical structures. The equivalent multiple degree of freedom (MDOF) model defines the structural system as simply as possible while maintaining all critical degrees of freedom and modes of deformation. Structural analysis according to AASHTO SGS would fall into this category. Finally, the equivalent SDOF lumps all structural behavior into an SDOF oscillator with a nonlinear force-deformation response defined by the system pushover curve.

TABLE 6
UNCERTAINTY IN NONLINEAR STRUCTURAL ANALYSIS METHODS (AFTER FEMA 440)

	Ground Motion Input	
Structural Model	Response spectra	Ground motion records
Detailed		Dynamic analysis
Equivalent MDOF	Multimode pushover analysis	Simplified MDOF dynamic analysis
Equivalent SDOF	Nonlinear static procedures	Simplified SDOF dynamic analysis

HIGH — LOW

Relative Uncertainty

SDOF = single degree of freedom.
MDOF = multi degree of freedom.

When the structural model or the ground motion input are simplified, the uncertainty increases. Because of this, the nonlinear static procedures typically have the largest inherent uncertainty owing to their modeling and analytical assumptions, and nonlinear response history analyses with detailed model definitions can often provide more accurate representations of actual structural behavior. However, increasing modeling refinement and complexity can introduce substantial uncertainty into the analytical solution by virtue of the difficulty in defining, verifying, and calibrating nonlinear structural response. Furthermore, the detailed model category in Table 6 is broad and ranges from relatively simple predefined hysteretic models (e.g. bilinear or Takeda) to fiber and continuum type models, each of which carries its level of uncertainty, along with limitations and challenges regarding its implementation.

Also, highly refined models and analysis may lull analysts into believing that the results have a higher accuracy, when in fact the seismic input often carries nontrivial uncertainty that may greatly outweigh the uncertainty of the structural model. Increased modeling complexity may then have a low benefit-to-cost ratio. In general, however, although uncertainty increases with analytical simplicity, time and cost decrease. There is typically a trade-off between cost and accuracy.

PROBABILISTIC TREATMENT OF NONLINEAR ANALYSES

One of the main goals of next-generation PBSD is the quantification of uncertainty throughout the analysis and design process. Therefore, how to characterize the amount of uncertainty in the predicted structural demand is of vital importance. Uncertainty can enter from two primary sources: (1) the input ground motion record-to-record variability and (2) the mathematical model used to predict structural response. As discussed in the previous section, simplified analytical procedures, such as nonlinear static analyses, increase the uncertainty in the estimation of structural demand. Therefore, there is an increasing trend to use nonlinear response history analyses in next-generation performance-based design procedures (ATC-58, PEER), as this provides a much more refined estimate of uncertainty and reduces the bias and uncertainty associated with simplified analytical techniques.

A rigorous evaluation of uncertainty is accomplished by using what is known as a probabilistic seismic demand model (PSDM), which relates a selected IM such as peak ground acceleration, peak ground velocity, or first mode spectral acceleration, to an EDP, such as drift ratio or plastic rotation. In their most rigorous form, PSDMs quantify demand uncertainty by applying a suite of scaled ground motions representative of the site-specific hazard (fault mechanism, distance to fault, event magnitude, and local site conditions) to the structural model using nonlinear dynamic analyses. The use of suites of ground motions is necessary because each ground motion will produce a different structural response, known as the record-to-record variability. ATC-58 (2011) recommends that at least 20 ground motions are necessary to truly represent the record-to-record variability in nonlinear structural response.

Perhaps the two most common rigorous treatments of record-to-record variability are the probabilistic seismic demand analysis (PSDA) and the incremental dynamic analysis (IDA).

> PSDA uses a bin approach, where a portfolio of ground motions is chosen to represent the seismicity of an urban region. The intensities of the ground motions in the portfolio cover a range of seismic hazard Intensity Measures (IM), such as first mode spectral acceleration. Nonlinear time-history dynamic analyses are performed for each motion using a model of the structure to compute extreme values of structure-specific Engineering Demand Parameters (EDP) (Mackie and Stojadinovic 2003).

PSDAs are explained in further detail in Mackie and Stojadinovic (2004) and Shome (1999).

The IDA or dynamic pushover is the continuous extension of a "single-point," nonlinear response history analysis in much the same way as the nonlinear static pushover analysis is the continuous extension of a "single-point" static analysis (Vamvatsikos and Cornell 2001).

> IDA is done by conducting a series of nonlinear time-history analyses. The intensity of the ground motion, measured using an IM, is incrementally increased in each analysis. An EDP, such as global drift ratio, is monitored during each analysis. The extreme values of an EDP are plotted against the corresponding value of the ground motion IM for each intensity level to produce a dynamic pushover curve for the structure and the chosen earthquake record (Mackie and Stojadinovic 2003).

Figure 12 shows example results from a PSDA and an IDA. Each demand model shows the structural response of a two-span ordinary California highway bridge. The only variation in the structural systems is the column diameter to superstructure depth ratio (Dc/Ds). In the case of PSDA, the relationship between first-mode spectral acceleration and the longitudinal drift ratio is linear in log-log space, intuitively with higher spectral accelerations producing higher drift ratios. Each data point denotes a unique ground motion that represents a particular seismic hazard at the site. However, for the IDA only four ground motions were selected, but each record was scaled incrementally until a specific IM was reached. The trace of EDP in relation to increasing IM is shown by the IDA curve, which again shows that an increase in IM results in a general increase in EDP. However, the increase in EDP is not always proportional to the increase in IM, with some cases

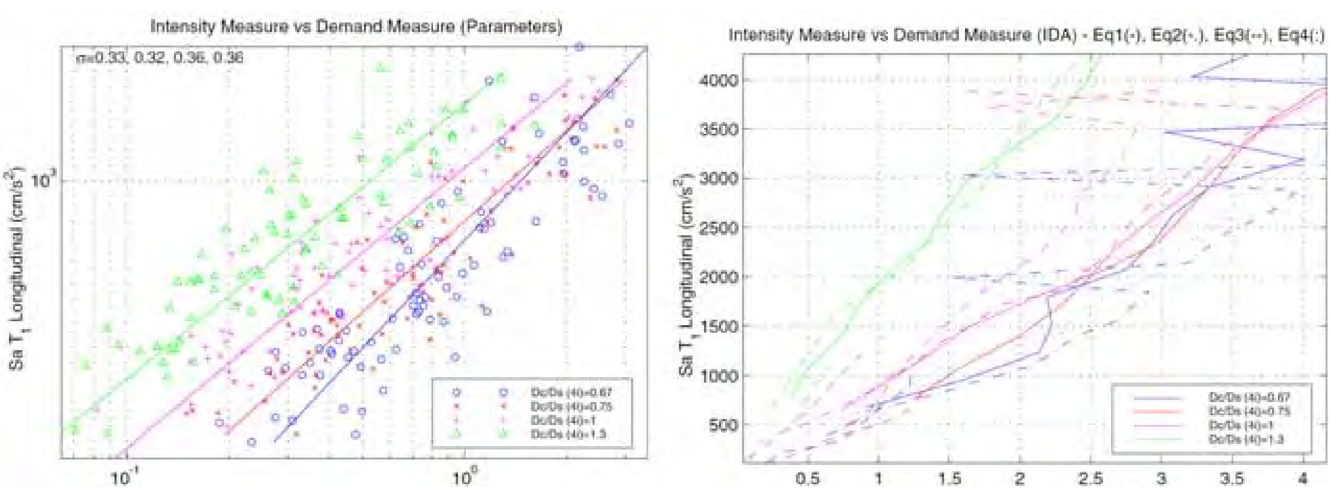


FIGURE 12 Example PSDA (left) and IDA (right) (Mackie and Stojadinovic 2003).

showing a higher IM generating a lower EDP. This is the result of the pattern and timing of response cycles, where "as the accelerogram is scaled up, weak response cycles in the early part of the response time-history become strong enough to inflict damage (yielding), thus altering the properties of the structure for the subsequent, stronger cycles" (Vamvatsikos and Cornell 2001). Characteristics of IDA response are further discussed in Vamvatsikos and Cornell (2001).

PSDA and IDA can be used interchangeably to determine the PSDM as long as a sufficient number of appropriate ground motions are chosen. Mackie and Stojadinovic (2003) observed that the IDA method is sensitive to the choice of ground motions, because a small suite of ground motions are used, whereas the PSDA generally provides sufficient ground motion variation as a result of the bin approach of ground motion selection. Mackie and Stojadinovic (2003) provide recommendations on the accuracy and equivalency of PSDA and IDA for generating PSDMs. Regardless of the method of generating the PSDM, the uncertainty within the prediction of EDPs is quantitatively determined.

As is obvious, the computational effort required to rigorously develop a PSDM is extremely high, and is likely only possible in academia and for critical or signature structures. To enable probabilistic treatment of demand uncertainty for noncritical structures, some assumptions must therefore be made. One such simplified method has been proposed by ATC-58 (2011). Although it still relies on nonlinear dynamic analyses, the number of permutations is greatly reduced by using best-estimate deterministic models and predetermined dispersion (β) factors for modeling uncertainty and ground motion record-to-record variability. The dispersion factors in this context represent the spread within the distribution of possible values; in other words it is the "fatness" or "width" of the bell-shaped curve describing the distribution of possible values. In many cases, the distribution within structural systems can be well represented using a lognormal probability density function, in which case the dispersion is the logarithmic standard deviation of the data set. The probability density function is then completely defined by the median response value and the dispersion.

This method requires that the nonlinear dynamic analyses represent the median structural response, using expected material properties and ground motions scaled so that the spectral geometric mean of the selected records matches the target spectrum within a predefined range of first mode periods. Typically, if there is strong correlation in spectral shape within the period range of interest, as few as three records can produce a reasonable prediction of the median response, whereas if there is high spectral fluctuation about the target spectrum, as many as 11 records may be needed. The selection and scaling of ground motion records is discussed further in NEHRP (2011).

ATC-58 (2011) provides ground motion record-to-record dispersion factors ($\beta_{a\Delta}$, $\beta_{aa,}\beta_{gm}$) based on the effective fundamental period, $\overline{T}$, and the strength ratio, S, of the structure. The strength ratio is effectively the ratio of the base shear demand on an infinitely strong elastic system to the base shear capacity of the yielding system, where $S_a(\overline{T})$ is the reactive weight, W is the first mode spectral acceleration, and V_{y1} is the first mode yield strength. A strength ratio of unity or less means the system behaves elastically. Table 7 shows example dispersion values for drift, $\beta_{a\Delta}$, and acceleration, β_{aa}. In general, the displacement and acceleration dispersion factors increase with fundamental period and a higher strength ratio (i.e., the yield strength is a smaller proportion of the elastic demand). These dispersion values are then used to determine the distribution of predicted structural response about the best-estimate median response. Additionally, ground motion spectral demand dispersion values for scenario-based assessments specifically (β_{gm}) are provided for western North America (WNA), CEUS, and the Pacific Northwest (PNW). The values for β_m (described subsequently) are default values of modeling uncertainty for typical new construction in the design development phase of the project.

TABLE 7

UNCERTAINTY DUE TO GROUND MOTION RECORD-TO-RECORD VARIATION (ATC 2011)

$\overline{T}, T_1$ (sec)	$S = \frac{S_a(T_1)W}{V_{y1}}$	$\beta_{a\Delta}$	β_{aa}	β_m	β_{gm} [1]		
					WNA	CEUS	PNW
0.20	≤ 1.00	0.05	0.10	0.25	0.60	0.53	0.80
	2	0.35	0.10	0.25			
	4	0.40	0.10	0.35			
	6	0.45	0.10	0.50			
	≥ 8	0.45	0.05	0.50			
0.35	≤ 1.00	0.10	0.15	0.25	0.60	0.55	0.80
	2	0.35	0.15	0.25			
	4	0.40	0.15	0.35			
	6	0.45	0.15	0.50			
	≥ 8	0.45	0.15	0.50			
0.5	≤ 1.00	0.10	0.20	0.25	0.61	0.55	0.80
	2	0.35	0.20	0.25			
	4	0.40	0.20	0.35			
	6	0.45	0.20	0.50			
	≥ 8	0.45	0.20	0.50			

Uncertainties will also develop from inaccuracies in component modeling (e.g., hysteretic behavior, material properties, imperfections in construction), damping, and mass assumptions. To account for this rigorously, mechanical properties can be treated as random variables with specified distributions during parametric studies. Therefore, a fully probabilistic treatment of modeling uncertainties requires extensive permutations and significant amounts of time and effort. In many cases, this is entirely cost prohibitive, and modeling uncertainty is treated using simplified methods such as those prescribed in ATC-58 (2011), which are described here.

The two primary sources of modeling uncertainty are (1) building definition and quality assurance, and (2) model quality and completeness. These are represented by dispersion values β_c and β_q respectively. β_c represents the variation in as-built structural member properties, such as in material strengths, geometry, and reinforcement location, with respect to properties assumed in analysis or specified in the design drawings, or both. β_q accounts for uncertainties in the modeling of actual structural behavior, such as the refinement and accuracy of the hysteretic model and the calibration to large-scale experimental test results. Tables 8 and 9 show example values. β_c and β_q are combined using the SRSS method to form the modeling uncertainty, β_m.

TABLE 8

UNCERTAINTY DUE TO BUILDING DEFINITION AND CONSTRUCTION QUALITY (ATC 2011)

Definition and Construction Quality Assurance	β_c
Building definition: The building is completely designed and well represented by drawings and specifications (i.e., construction documents are available). *Construction quality*: The building was or will be constructed using rigorous construction quality assurance measures, including special inspection, materials testing and structural observation	.10
Building definition: The building is completely designed or nearly completely designed but available drawings do not completely define all details of construction (i.e., design development documents or equivalent are available). *Construction quality*: For existing buildings, the structure is sited in a region and is of an age in which rigorous construction quality assurance measures were likely implemented. For new construction, the building will be constructed using reasonable construction quality assurance measures.	0.25
Building definition: The design is incomplete (i.e., schematic documents are available), or, for an existing building is known only on the basis of limited field observation and testing. *Construction quality*: For new or existing buildings, little information on the construction quality assurance program is available.	0.40

TABLE 9

UNCERTAINTY DUE TO MODEL QUALITY AND COMPLETENESS (ATC 2011)

Model Quality and Completeness	β_q
Model quality: The numerical model for each component is robust over the anticipated range of displacement or deformation response. Strength and stiffness deterioration and all likely failure modes are modeled explicitly. Model accuracy is established with data from large-scale component tests through failure. *Completeness*: The mathematical model includes all structural components and nonstructural components in the building that contribute strength and/or stiffness.	.10
Model quality: The numerical model for each component is robust over the anticipated range of displacement or deformation response. Strength and stiffness deterioration is fairly well represented, though some failure modes are simulated indirectly. Accuracy is established through a combination of judgment and large-scale component tests. *Completeness*: The mathematical model includes most structural components and nonstructural components in the building that contribute significant strength and/or stiffness.	0.25
Model quality: The numerical model for each component is based on cyclic envelope curve models of ASCE 41 or comparable guidelines, where strength and stiffness deterioration and failure modes are incorporated indirectly. *Completeness*: The mathematical model includes all structural components in the lateral-force-resisting system.	0.40

For typical well-defined new construction of ordinary structures, the building definition and construction quality would fall into the first or second category with a β_c value of 0.10–0.25, as the drawings are complete (or nearly complete) and the construction is monitored and of reasonable to high quality. However, the modeling uncertainty would be relatively high if a structural analysis is conducted with nonlinear models of similar form and refinement to the ASCE 41 envelope curves. These curves have not been explicitly calibrated to large-scale laboratory experiments. This results in a β_q value of 0.40 if significant inelastic action is expected. This value may be reduced if the structure behaves nearly elastically or with low levels of ductility. Higher modeling sophistication is currently reserved for signature or critical structures and academic studies.

Finally, the ground motion record-to-record variability is combined with the modeling uncertainty to generate the dispersion of the predicted structural response. This combination is again done using the SRSS method. At this point, the median structural response has been defined by the nonlinear response history structural analysis and the dispersion is quantified by the combination of the tabulated β factors.

As seen in Tables 7, 8, and 9, there is no clear trend as to the dominant source of uncertainty for all applications, as the dispersion varies depending on structural characteristics (stiffness and strength), the desired output of the assessment (drift or acceleration), the building definition, the construction quality control, and the modeling assumptions/calibration used.

Even with the ATC-58 simplifications, the uncertainty within seismic demand prediction still relies on complex, time-consuming, nonlinear dynamic analyses, which may prevent the complete methodology from being applied to ordinary or nonessential bridges. Continued emphasis should therefore be placed on the research and development of simple, robust, and accurate analytical procedures. It would appear that for ordinary and nonessential bridges, fully probabilistic analytical methods are simply too time and cost prohibitive. Even with the increase in computational power and efficiency, detailed nonlinear dynamic analyses may not become the norm for ordinary structures, in which case simplified analytical techniques should be further refined and strengthened. Steps in this direction have recently been made with documents such as FEMA 440 (2005). Uncertainty can be classified with dispersion factors for the median response predicted by simplified methods that are adopted into next-generation performance-based codes and guidelines. However, there is still a long way to go before such methodologies can be implemented in practice, and this is just one link of a chain of calculations that must be completed for PBSD to be implemented in a probabilistic fashion.

CHAPTER SIX

DAMAGE ANALYSIS

Fundamental to the PBSD methodology is the need to determine the type of damage and the likelihood that such damage will occur in particular components of the structural system. This determination is of vital importance, as the damage sustained by a structure (and its nonstructural components) is directly relatable to the use or loss of a system after an earthquake. Therefore, there is a need to reliably link structural and nonstructural response (internal forces, deformations, accelerations, and displacements) to damage. This is the realm of damage analyses, where damage is defined as discrete observable damage states (e.g., yield, spalling, longitudinal bar buckling, bar fracture). The primary focus of this chapter is on structural components, but similar considerations must be made for nonstructural components as well.

In this chapter, an initial discussion of types of structural damage observed during historic earthquakes and laboratory experiments prefaces the methods that have been developed to predict damage. This discussion is followed by structural details and concepts that can be used to reduce damage even in strong ground shaking. Finally, postevent inspection tools are reviewed.

REVIEW OF BRIDGE DAMAGE FROM EARTHQUAKES

Bridges have suffered various types of damage in past earthquakes, as evidenced by the San Fernando (1971), Loma Prieta (1989), and Northridge (1994) earthquakes in California; the 1995 Kobe earthquake in Japan; the 1999 Chi Chi earthquake in Taiwan; the 1999 Kocaeli and Duzce earthquakes in Turkey; the 2010 Darfield and 2011 Christchurch earthquakes in New Zealand; the 2010 Maule earthquake in Chile; and the 2011 Tohoku earthquake in Japan, where virtually every bridge component has experienced some sort of damage—in some cases leading to structural collapse. The following section includes a brief discussion, including photographs, of bridge damage observed in some of these earthquakes. Although there has been extensive laboratory testing on many components and subassemblages, real earthquakes have a powerful ability to reveal structural weaknesses or design deficiencies. Accordingly, extremely valuable information for the development of PBSD can be gathered from actual bridge performance in past earthquakes.

Some general observations, taken from Cooper et al. (1994) and Kiremidjian and Basöz (1997), can be made with regard to the observed behavior and performance of bridges in the 1989 Loma Prieta and 1994 Northridge, California, earthquakes: (1) Bridge skew angle, abutment type, pier type, and span continuity showed the highest correlations to damage for given levels of ground shaking, specific details of which can be found in Basöz and Kiremidjian (1998). (2) New bridges (post-1981) performed well, whereas designs of the 1972–1981 vintage had mixed performance. Pre-1971 designs generally performed quite poorly. This is a direct indicator that the improved seismic design criteria generated after the 1971 San Fernando, California, earthquake do achieve their performance goals, whereby inelastic action is controlled and life safety is maintained. The mixed response of 1972–1981 bridges is due primarily to the transition to the enhanced design criteria precipitated out of the San Fernando earthquake. (3) Retrofit measures, such as joint restrainers, column jacketing, and foundation strengthening, can dramatically improve the response of nonductile bridge designs. Although they are not a guarantee of perfect performance, seismic retrofits are effective at reducing damage and preventing collapse. (4) The preparedness of local jurisdictions and municipalities prior to a significant seismic event is critical to rapid emergency response, inspection, diversion of traffic to alternate routes, and repair/reconstruction efforts. (5) Lessons learned from past earthquakes and experimental research result in tangible improvements in performance. Although the seismic design community is still far from a complete understanding of structural behavior and performance in response to strong ground shaking, significant progress has been made, and research efforts continue.

A detailed review of bridge performance is beyond the scope of this synthesis, but there are many excellent resources. Overviews of bridge damage can be found in Pond (1972), Housner and Thiel (1990), Housner and Thiel (1994), Priestley et al. (1996), Chen and Duan (1999), Kawashima (2000, 2001), Yashinsky and Karshenas (2003), and Palermo et al. (2010), among many others. Perhaps one of the best surveys of bridge damage can be found in the EERI reconnaissance reports published as a part of the Earthquake Spectra series. EERI has conducted close to 300 international postearthquake investigations over the past 40 years as a part of its Learning from Earthquakes (LFE) program. Reconnaissance information can be found in the LFE Reconnaissance Archive (http://www.eeri.org/projects/learning-from-earthquakes-lfe/lfe-reconnaissance-archive/). Another resource provided by EERI is the Earthquake Clearinghouse (http://www.eqclearinghouse.org/), which provides a consolidated repository for brief observations from EERI reconnaissance teams shortly after a significant seismic event. Detailed observations and reports are often linked from the clearinghouse website.

The detailed analysis of damaged bridges has become an important component of postevent investigation, as it provides the opportunity for engineers to verify that current analytical methods are capable of predicting the failure modes observed in the field, as well as ensuring that design details and configurations perform as expected. Several detailed examples from recent earthquakes in California have shown that the observed damage could be explained and predicted by current nonlinear analytical methods as long as appropriate modeling assumptions are made (Housner and Thiel 1990, 1994; Broderick et al. 1994; Priestley et al. 1994; Basöz and Kiremidjian 1998). Such analyses of an entire bridge system helps further interpret the role of individual component damage in the overall damage state and performance of the bridge system. The data reviewed herein are largely concerned with component behavior, yet performance is an attribute of a system. Additionally, damage analysis of bridge systems following significant earthquakes can be used to direct further research and improve design criteria.

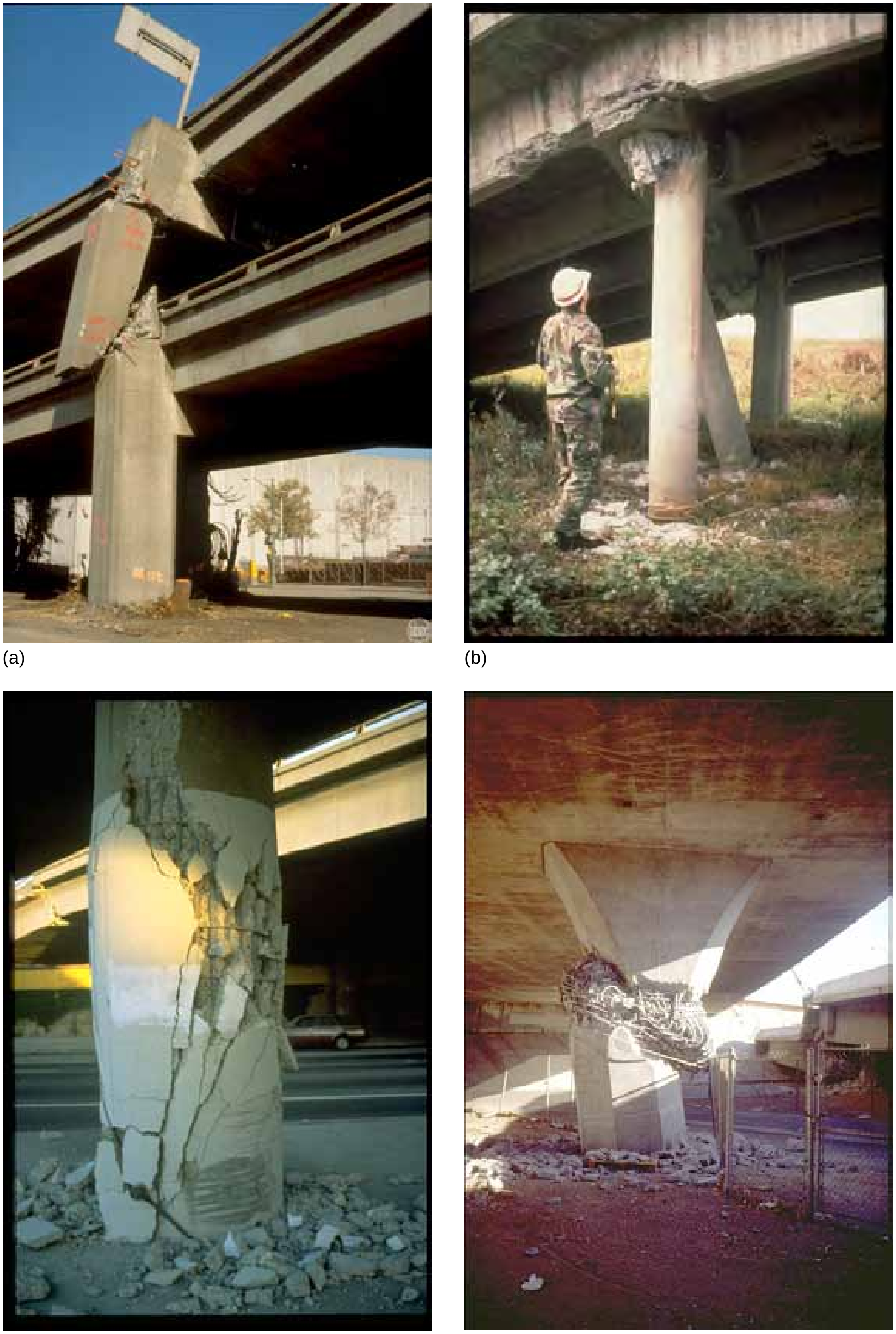
DAMAGE STATES

Past earthquakes have caused significant damage to some bridges, greatly affecting their functionality after ground shaking subsides. Structural damage has been observed in all major bridge components, exhibiting multiple failure modes. Figure 13 shows examples of bridge damage as seen in the 1989 Loma Prieta and the 1994 Northridge earthquakes. Perhaps the most well-known bridge collapse during the Loma Prieta earthquake was that of the Cypress Viaduct, which claimed 43 lives (Priestley et al. 1996). This collapse was largely caused by column joint shear failures, as shown Figure 13a, the result of the lack of a robust load path through the joint preventing force transfer from the columns to the crossbeams. Another significant collapse during Loma Prieta was the Struve Slough Bridge, shown in Figure 13b. In this case, the columns (pile extensions) had minimal transverse reinforcement at the connection to the skewed transverse cap beam. Under strong shaking the soil around the piles deformed significantly, leading to pile-to-cap plastic hinge failures, resulting in the collapse of the central segment of the bridge (Jablonski et al. 1992). The San Francisco–Oakland Bay Bridge was also damaged in this earthquake.

The Northridge earthquake was also particularly damaging to reinforced concrete bridges, with a number of short-column, brittle shear failures owing to the low transverse steel quantities within the columns, as seen in Figure 13c. This problem was exacerbated by geometric effects such as short, stiff columns attracting significant loading. This led Caltrans to develop criteria that holistically treat the bridge system whereby mass and stiffness was balanced throughout the bridge. Another design flaw that led in some cases to significant damage were flared columns similar to the one shown in shown in Figure 13d. These architectural column flairs were intended to break away under transverse loading allowing the column to develop a plastic hinge at the top of the column adjacent to the bridge superstructure soffit. However, in several columns the flairs remained intact forcing the plastic hinge to form lower in the column, increasing the rotational demands on the columns, which resulted in plastic hinge failures. Also evident is the fractured transverse reinforcement within the plastic hinge. This lack of core concrete confinement led to axial crushing of the plastic hinge. Damage from the Loma Prieta and Northridge earthquakes instigated much of the recent research on bridge components.

One of the purposes of conducting laboratory experiments on bridge components, subassemblages, and systems is to correlate definable deformation responses, such as strains, curvatures, rotations, and EDPs, with the initiation and progression of damage states, known as DMs. Finally, the DMs can be related to the functionality or loss of the bridge in terms of performance level. These correlations have been defined within the literature, but there is often a disparity in what is reported between researchers, institutions, and organizations. An example of such a correlation, which is based on the five-level performance evaluation approach developed and used extensively by the University of California, San Diego (Hose and Seible 1999) and by Caltrans in their Visual Catalog of Reinforced Concrete Bridge Damage (2006), is presented in modified form in Table 10.

Example correlations between damage levels (DLs), hysteresis, and EDPs are given in Figures 14 and 15 and Table 11. The hysteretic response of the specimen is characteristic of well-confined, reinforced concrete columns with a near elastic perfectly plastic response and stiffness degradation upon unloading, particularly at high ductility levels. The specimen was able to reach a displacement ductility of eight, which corresponds to a drift angle of nearly 9% before bar buckling and fracture occurred. This is well above the displacement ductility demand limits imposed in the AASHTO SGS and the Caltrans SDC. It is also evident that damage levels I through IV are repairable, as the transverse and longitudinal steel have not fractured or buckled, and the core concrete remains intact. Damage level V would require significant repair efforts or even complete column or bridge replacement.



(a) (b)

(c) (d)

FIGURE 13 Typical bridge damage: (*a*) Cypress Viaduct collapse (Loma Prieta, 1989); (*b*) Struve Slough Bridge (Loma Prieta, 1989); (*c*) I-10 at Venice Blvd. (Northridge, 1994); and (*d*) Mission Gothic Bridge (Northridge, 1994). (*Courtesy*: National Information Service for Earthquake Engineering, EERC, University of California, Berkeley.)

TABLE 10
BRIDGE DAMAGE AND PERFORMANCE ASSESSMENT

Damage Level	Damage Classification	Damage Description (Damage Measures)	Performance Level
I	No	• Onset of hairline cracks	Fully operational
II	Minor	• Crack widening • Theoretical first yield of longitudinal reinforcement	Operational
III	Moderate	• Initiation of inelastic deformation • Onset of cover concrete spalling • Development of diagonal cracks	Limited damage
IV	Major	• Formation of very wide cracks • Extended concrete spalling	Life safety
V	Local failure/ collapse	• Buckling of main reinforcement • Rupture of transverse reinforcement • Crushing of core concrete	Collapse

TABLE 11
BRIDGE PERFORMANCE/DESIGN PARAMETERS SRPH-1 (HOSE AND SEIBLE 1999)

Level	Description	Steel Strain	Concrete Strain	% Drift	Displacement Ductility
I	Fully operational	<0.005	<0.0032	<1.0	<1.0
II	Operational	0.005	0.0032	1.0	1.0
III	Life safety	0.019	0.01	3.0	2.0
IV	Near collapse	0.048	0.027	5.0	6.0
V	Collapse	0.063	0.036	8.7	8.0

Comparing the data in the Figure 14 and Table 11 with those in Table 10 reveals that the life safety designation applies at very low displacement or ductility levels in Hose and Seible's (1999) work, while the Caltrans and AASHTO design procedures would permit much higher displacements. This inconsistency in terminology and nomenclature should be resolved before PBSD can be uniformly applied. In Table 10, life safety has been adjusted upward to reflect the normal application of the term in bridge design practice.

The current relationships between damage and performance levels are somewhat subjective, based on the perceived risk and observed actual performance of components and structures in past earthquakes. Accordingly, there is some variation between correlations of damage and performance, depending on the agency or institution that produces the relationship. One hurdle that must be overcome to a reach a fully developed PBSD methodology is a consensus on what damage and performance levels should be incorporated and what definitions of damage and performance can be used.

Regardless of the performance levels chosen, all performance assessment metrics use visually or analytically determinable milestones of damage to quantify an EDP limit. The visual limit state assists postearthquake inspectors in the field, and the analytical limit states assist the designer. Example limit states for reinforced concrete and steel members will likely include some or all of the damage states listed in Table 12.

Although some authors have used as many as five separate damage levels of interest (Hose and Seible 1999), these could also be distilled down to two for reinforced concrete elements—the onset of cover concrete spalling and the initiation of longitudinal bar buckling—as they represent the flexural damage states that indicate the onset of operability limitations and onset of life safety concerns, respectively (Berry and Eberhard 2003). A nationwide consensus (among researchers, government, and practitioners) is needed on what damage states should be evaluated. This consensus would also assist in setting consistent damage state limits for bridges of high and low importance because of postevent operability requirements.

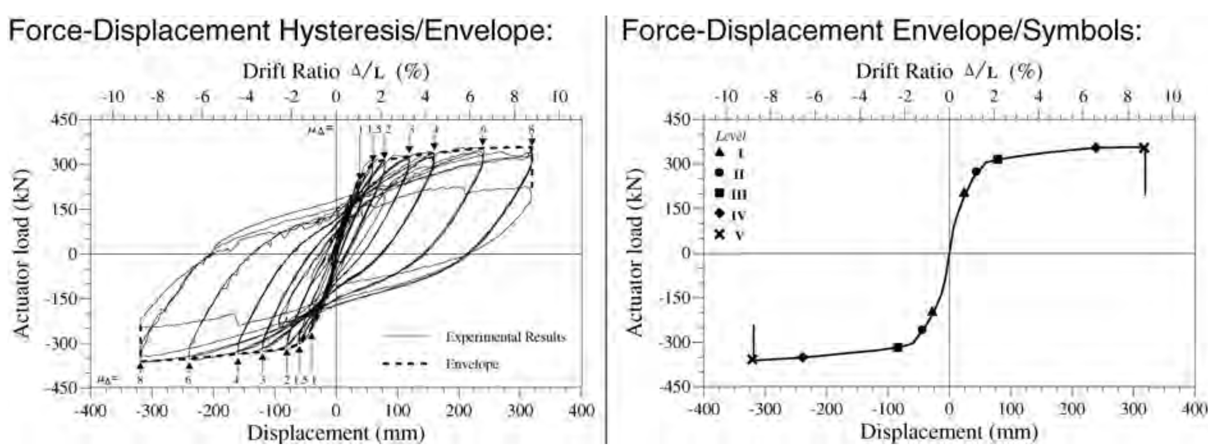


FIGURE 14 Typical bridge column performance curves SRPH-1 (Hose and Seible 1999).

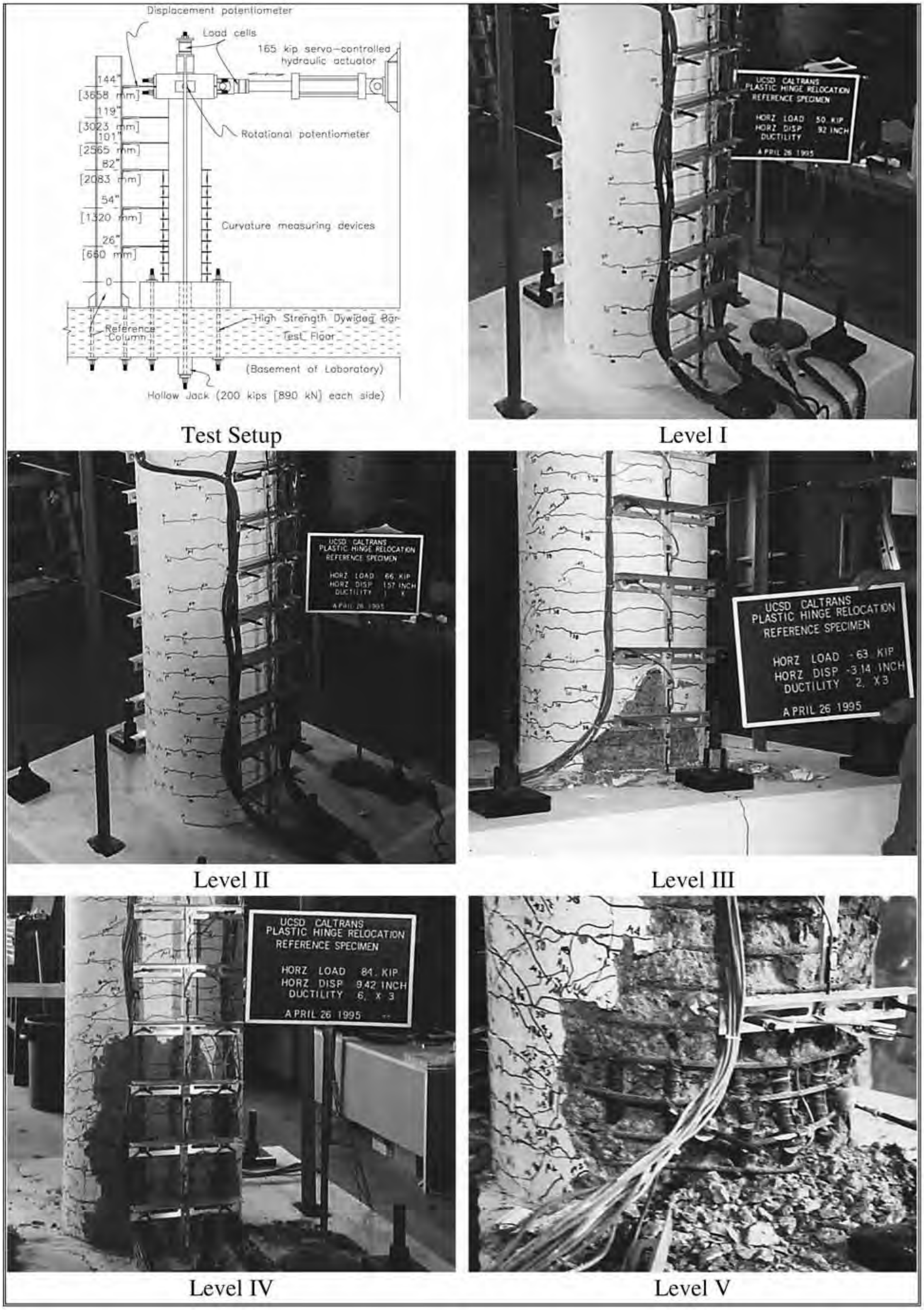


FIGURE 15 Bridge column damage at various damage levels SRPH-1 (Hose and Seible 1999).

TABLE 12
REINFORCED CONCRETE AND STRUCTURAL STEEL MEMBER DAMAGE STATES

Reinforced Concrete Damage States	Structural Steel Damage States
Concrete cracking	First yield
Cover concrete spalling	Local buckling (e.g., flange or web)
Core concrete crushing	Lateral torsional buckling
Yield of the longitudinal reinforcement	Brace buckling
Fracture of the transverse reinforcement	Fatigue cracking
Buckling of the longitudinal reinforcement	Connection fracture (e.g., bolt or weld)
Fracture of the longitudinal reinforcement	Gross or net section fracture or tearing
Residual deformations	Residual deformations

An observation or caveat regarding the element correlations between EDP and performance level discussed previously is that they are for individual elements or components, not systems. Thus, depending on the redundancy, configuration, boundary conditions, and articulation of the bridge system, such damage may not be indicative of the entire bridge and its performance. Often, individual components may experience damage that affects a local region, but the system as a whole may not be at the same damage level. In such situations, a local repair may return the bridge to full service in a short time, even though the component damage could have been severe, even near collapse. This situation produces a challenge in analytically predicting behavior, a priori, and making decisions regarding use before inspecting the bridge postearthquake.

A classic example is the collapse of two deck spans at Pier E9 on the East Bay Crossing portion of the San Francisco–Oakland Bay Bridge in the 1989 Loma Prieta earthquake (Housner and Thiel 1990), as shown in Figure 16. The spans were replaced and the bridge was returned to service relatively quickly. In no way was the entire bridge system a loss, even though several components reached a collapse damage state. A replacement for the bridge is under construction, but the original bridge has remained in service for many years following the earthquake.

DAMAGE PREDICTION

In first-generation PBSD, the onset of damage has typically been treated as discrete deformation limits based on strain

FIGURE 16 San Francisco–Oakland Bay Bridge Span E-9 Collapse, Loma Prieta Earthquake, 1989. (*Courtesy*: National Information Service for Earthquake Engineering, EERC, University of California, Berkeley.)

(e.g., AASHTO SGS) or rotation (e.g., ASCE 41), which essentially quantifies each damage state deterministically, where the likelihood of damage goes from 0% to 100% the instant a damage limit is reached. Unfortunately, the onset of damage is not a discrete deterministic quantity; there is a distribution of values. In effect, the damage prediction is a probabilistic problem, not a deterministic one. What is not often clear in the codes and some literature is whether the reported deformation limits represent lower bounds, mean, or some intermediate value for the onset of damage. This ambiguity results in a situation where the dispersion of the data and the exact location of the limiting value within the statistical spread are unknown.

This ambiguity is perhaps best illustrated through the cover concrete strain at the initiation of spalling in reinforced concrete columns. One issue that immediately arises is that the concrete strain at spalling is not directly measurable during an experimental test, and therefore, it has to be either back-calculated or determined using a numerical predictive analysis. To determine the strain at spalling using experimental data, the concrete strain must be calculated based on the experimentally measured curvature and steel strains using a plane-section assumption. While theoretically this approach is simple, such a calculation can introduce significant errors with respect to the curvature and the strain recordings due the effects of cracking and bond deterioration. The curvature measured during a test is an average curvature over a specific gauge length, and the steel strain recorded is subject to the proximity of the strain gauge to a crack within the concrete, along with thermal effects because of heat generated during yielding of the steel. These errors make the back calculation of the spalling strain based on experimental data difficult.

To circumvent these problems, the nominal strain at spalling can be calculated using a predictive analysis of the entire test specimen, which typically uses a moment-curvature analysis, the assumed plastic hinge length, and the second moment area theorem to determine deflections. However, when this is done, there is still a considerable spread in the predicted or calculated concrete strain at the onset of spalling, with values ranging from 0.002 in./in. to 0.018 in./in. for circular columns and 0.002 in./in. to 0.01 in./in. for rectangular columns (Lehman and Moehle 1998; Hose and Seible 1999; Berry and Eberhard 2003). Relationships between cover spalling and various parameters for spiral columns are shown in Figure 17, generated as a part of the research conducted by Berry and Eberhard (2003), where it is clear from the vertical distribution of points that there is a no obvious choice for the strain at spalling. The solid and dashed lines in the figure represent families of specimens that are nominally identical, other than the property defined as the abscissa. All columns included in the data set shown are classified as flexure-critical, have an aspect ratio (L/D) greater than 1.95, and the longitudinal reinforcement is not spliced.

The variables are defined as follows.

ε_{spall} = Extreme concrete strain at onset of spalling

D = Column diameter

P = Column axial load

L = Length of column cantilever

A_g = Gross concrete area

d_b = Longitudinal bar diameter

$$\rho_{eff} = \frac{\rho_s f_{ys}}{f'_c} \text{ = effective confinement ratio} \qquad \text{Eq. 1}$$

Where:

ρ_s = is the volumetric transverse ratio

f_{ys} = is the yield stress of the spiral reinforcement

f'_c = is the concrete compressive strength

$$\omega = \frac{\rho_l f_y}{f'_c} \qquad \text{Eq. 2}$$

Where:

ρ_l = is the volumetric longitudinal ratio

f_y = is the yield stress of the longitudinal reinforcement.

It is clear that the often-used values of 0.004 to 0.005 extreme compression fiber strain are simplifications of a highly variable quantity, and that there is poor correlation between spalling and the correlative secondary variables used in the figures. To further complicate the matter, damage initiation in steel and concrete appears to be dependent on the history of inelastic loading, known as the cumulative damage process (El-Bahy et al. 1999; Moyer and Kowalsky 2003; Tsuno and Park 2004). During seismic attack, the number and magnitude of inelastic excursions are a function of the ground motion and the structural characteristics (e.g., stiffness, strength, energy dissipation). These excursions can only be accounted for directly using a nonlinear response history analysis, where flexural damage models (low-cycle fatigue, bar buckling, and cumulative energy dissipation) are employed to predict if the ultimate deformation capacity of the component has been reached. Furthermore, the interaction between bar buckling, bar fracture (as result of low-cycle fatigue or prior buckling cycles), and transverse steel quantity is complex and is not entirely understood.

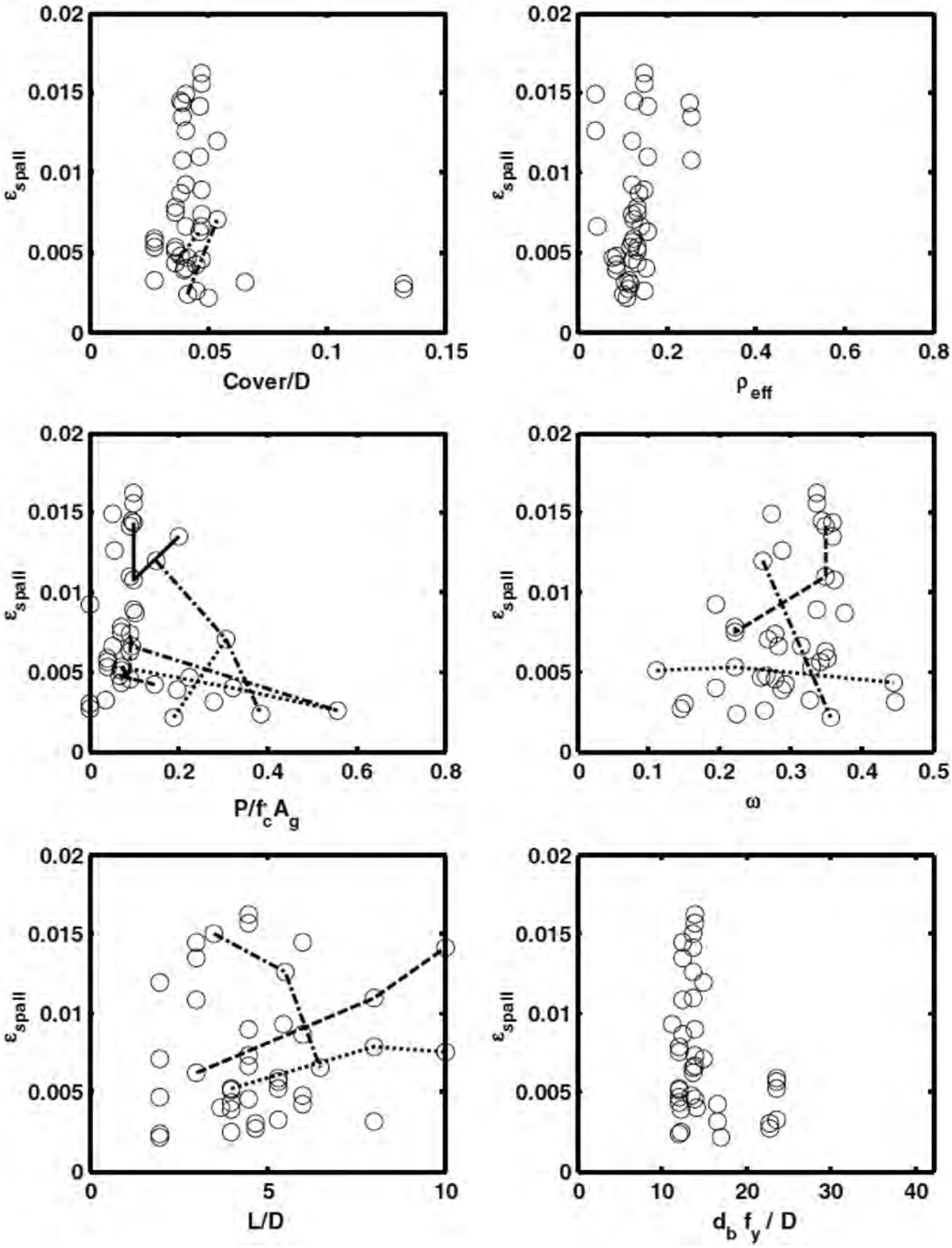


FIGURE 17 Trends in nominal compressive strain at cover spalling, circular columns (Berry and Eberhard 2003).

Although there has been considerable research investigating the effects of cumulative damage processes (Manson 1953; Coffin 1954; Manson and Hirshberg 1964; Mander et al. 1994; El-Bahy et al. 1999; Brown and Kunnath 2000; Moyer and Kowalsky 2003; Tsuno and Park 2004; Saisho 2009; Hawileh et al. 2010, among others), research is still ongoing at North Carolina State University by M. Kowalsky to evaluate current PBSD material strain limits and the relationship between strain and displacement considering the seismic load history and even temperature effects. This research is of critical importance as the preponderance of design practice uses monotonic sectional analyses and strain limits to define the performance (and damage) of a structure. The strain limits are currently selected without a consistent scientific justification or basis, and a given strain is nonunique to a specific displacement because of the uneven accumulation of tensile and compressive strain under cyclic loading.

Cumulative damage/load history effects without question contribute to the uncertainty in damage initiation, but further research is needed in this area to quantify their participation. Furthermore, load-history effects are significant in the determination of structural performance during strong aftershocks, as the initial conditions of the structure for the aftershock ground motion can be drastically different from those of the undamaged structure. These are potentially major hurdles that must be overcome before PBSD can be fully realized and implemented.

Because there is uncertainty or variability within the onset of damage, a conservative, lower-bound estimate is typically used as a discrete deterministic limit. This estimate adds conservatism to the solution and reduces the probability of damage occurring at lower deformation levels. Although this is certainly a valid methodology and has been used successfully in first-generation PBSD, improvements can be made. The prediction of damage can be further refined by employing a probabilistic description for the onset of damage, where deformation limits can be selected using a consistent basis and the inherent uncertainty in damage initiation can be defined. However, before the probabilistic treatment of damage is discussed, a brief aside will draw an analogy to the variation in strength of structural components using structural reliability theory.

It is well known in structural design that the strength of any element will fall within a certain distribution clustered about a central tendency (mean or median). The variability in strength is used in LRFD design, where both resistance and loading are treated as random variables described according to a certain distribution or probability density function (e.g., normal Gaussian, lognormal). Structural safety is achieved by selecting a lower-bound element resistance and an upper-bound load effect that will produce an acceptably low probability of failure.

Resistance factors (whose values are less than unity) are used to reduce the predicted element strength such that it lies toward the lower bound of the bell-shaped distribution of strength. The distribution of strength (resistance) is shown in Figure 18 as a Gaussian probability density function where m_R is the mean resistance, R_n is the nominal resistance, and ΦR_n is the design resistance.

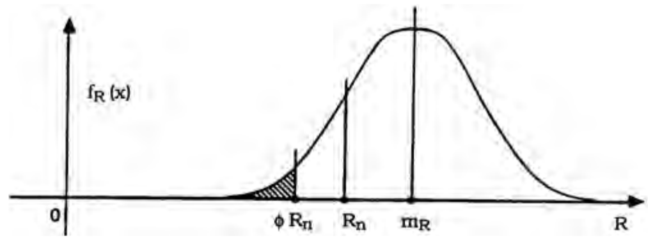


FIGURE 18 Distribution of resistance (Nowak 1999).

Associatively, load factors (whose values are greater than unity) are used to increase the service-level loads so they are representative of the upper bound of combined load effect on the element. The distribution of the loading is shown in Figure 19 as a Gaussian probability density function where m_X is the mean load, X_n is the service load, and γX_n is the factored load used in strength design checks.

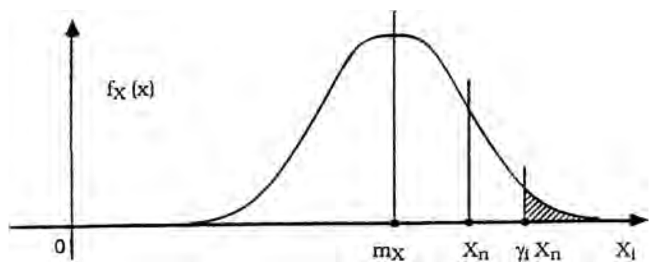


FIGURE 19 Distribution of load effect (Nowak 1999).

The load and resistance factors are calibrated based on the nature of the loading (e.g., duration, frequency, severity), the variability of component strength (i.e., the dispersion or uncertainty), the component failure mechanism (ductile or brittle), and the desired probability of failure. As long as the design resistance is greater than the factored load, the LRFD equation ($\Phi R_n \geq \gamma X_n$) is satisfied, and the design is considered safe.

The statistical variability of strength is also included in modern seismic design philosophy of capacity protection, where it is recognized that ductile elements experiencing plastic deformation have an associated distribution of strength with a maximum feasible strength, or "overstrength." Adjacent nonductile elements are then designed to elastically resist the overstrength forces, thereby capacity protecting them from damage, as higher seismically induced internal forces cannot be generated within the structure.

If the distribution of component strength is superimposed onto the nonlinear force-displacement response of a reinforced concrete column (as in the upper portion of Figure 20), significant strength levels can be identified. The expected (mean) strength of the component is represented by the solid line and is determined using expected material properties (1.3f'$_c$ and 1.1f$_y$). This is equal to m_R in Figure 18. The maximum feasible strength of the component is represented by the overstrength line and is typically 1.2 to 1.4 times greater the expected strength for reinforced concrete elements, in this case represented by 1.7f'$_c$ and 1.3f$_y$. Finally, the nominal strength is determined using nominal material properties. Note that this is the same strength represented by R_n in Figure 18.

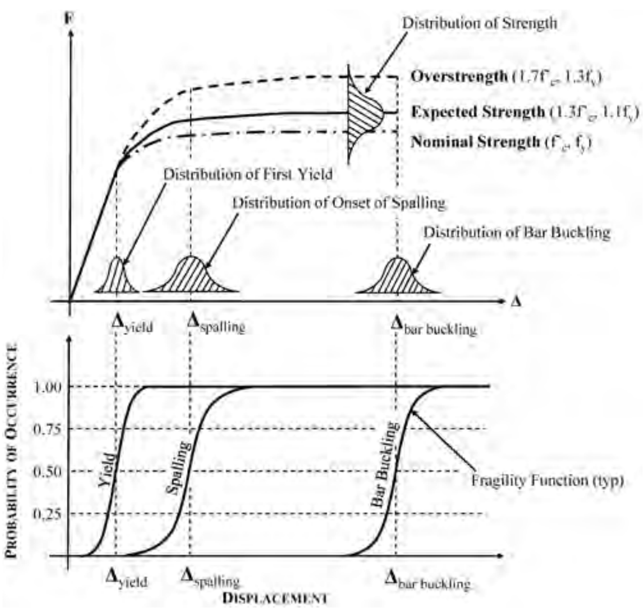


FIGURE 20 Sources and treatment of uncertainty in strength and damage.

It can now be seen that the probabilistic treatment of capacity and demand is nothing new to structural analysis, and it is therefore a simple extension to treat damage accordingly. In Figure 20, three damage states have been identified: first yield, spalling, and longitudinal bar buckling. Each damage state is also assigned an expected value, indicated by the vertical dashed line, and a distribution of possible values surrounding the expected value (the probability density function). The uncertainty of each damage state is unique, as indicated by the dispersion or width of the distribution. First yield is relatively well defined, as indicated by the narrow distribution. This means that the variable has little uncertainty and, if the dispersion is small enough, it can be treated deterministically. However, the distributions for spalling and bar buckling are much wider, showing that there is significant uncertainty in their values.

If the probability density function is integrated (summed) along its domain, the result is a cumulative distribution function (CDF). In the context of damage analysis, the CDF is known as a fragility function and represents the probability that a certain damage state (known here as DM) will occur given a certain value of an EDP, such as strain, rotation, drift, or displacement. Typically, a lognormal distribution is adopted in earthquake engineering for several key reasons: It fits a wide variety of structural and nonstructural components along with structural collapse, it has a strong precedent in seismic risk analysis, and it has an advantage theoretically because there is zero probability density at and below zero EDP values (Porter et al. 2007). However, fragility functions can also be defined using a normal Gaussian distribution. The general mathematical form of a lognormal fragility function is given in Equation 3.

$$P(DM \mid EDP) = \Phi\left[\frac{1}{\beta}\ln\left(\frac{EDP}{\theta}\right)\right] \qquad \text{Eq. 3}$$

Where $P(DM|EDP)$ is read as "the probability of DM occurring given EDP," Φ is the normal cumulative probability distribution defined as a function of the median EDP value of the distribution (θ) and the dispersion (β), which mathematically is the logarithmic standard deviation.

Fragility functions for the three column damage states are shown in Figure 20, which shows that the fragility function occupies the domain where its respective damage state can occur. High-variability damage have shallow fragility functions, while low-variability damage states have steep fragility functions. While the schematic representation of fragility functions are relatively simple, their derivation and modification from experimental or analytical data or expert opinion can be complex and are outside the scope of this synthesis. However, Kennedy et al. (1980), Krawinkler and Miranda (2004), Porter et al. (2007), and ATC (2011) are excellent references in fragility function theory and development.

Care must be taken in developing and applying fragility functions, as they are only as reliable as the data used to develop them. Accordingly, the selection and filtering of the input data are critical so that the fragility function relates only the true uncertainty of the actual component of interest. This uncertainty includes the variability of construction quality, actual material properties in relation to the properties assumed in design, and the testing procedure (e.g., test setup, loading protocol, boundary conditions). In effect, only the variability that cannot be directly accounted for using well-defined engineering principles should be included. Discretion is also required in selecting the EDP to define the fragility function of a component. Typically, the EDP with the lowest dispersion will be selected, as this will increase the accuracy of the damage prediction. For columns, this will be the drift ratio or the plastic hinge rotation.

The probabilistic treatment of damage can then be applied to PBSD in several different ways. First, the damage fragility functions can be used to generate deformation (strain, rotation, or drift) limits that have a uniform probability of occurrence. For example, if it is acceptable to have a 20% chance of spalling at the design strain limit, the fragility function relating the probability of spalling to extreme cover concrete strain can be used to determine the strain associated with a 20% probability of spalling. This partial implementation would provide a more rigorous basis for discrete deterministic deformation limits than what is currently employed in first-generation PBSD. Second, fragilities can be used in postearthquake bridge inspection prioritization; this will be discussed later in the chapter. And finally, as discussed in chapter seven, the component damage fragilities can be used during vulnerability or loss analysis in the full probabilistic form of PBSD.

For a probabilistic damage analysis methodology to be fully functional, all relevant damage states must be defined for all relevant components, which requires extensive experimental and analytical testing and investigation (Krawinkler and Miranda 2004). This research includes areas such as accelerated bridge construction (ABC) and nonconventional ductile components. Although a fully probabilistic PBSD methodology is desirable from a purist perspective, a large data gap waits to be filled. Reinforced concrete columns have seen the most laboratory research attention for bridge components. This is evidenced by the wealth of test data as found in the PEER Structural Performance Database (http://nisee.berkeley.edu/spd/). The database includes more than 400 cyclic lateral load tests of circular, octagonal, and rectangular columns with various reinforcement configurations. The Kawashima Research Group has compiled a similar database of Japanese column tests (http://seismic.cv.titech.ac.jp/).

Until fragility relationships are developed for all structural components and the dissemination of the information to structural designers, PBSD has to be implemented using deterministic methods based on the information that is currently available (Priestley et al. 2007). As is evident from the earlier discussion of ATC-58, the building industry has made relatively large strides in the development of a full suite of fragility functions. The bridge industry has only begun this development.

Perhaps one of the best recent examples of the development and application of bridge component fragility is the work of Berry and Eberhard (2003) at the University of Washington. They focused on the damage states of cover concrete spalling and longitudinal bar buckling for reinforced concrete columns. As an input for their work they surveyed more than 100 flexure-critical rectangular and circular bridge column tests with aspect ratios of 1.95 or greater. None of the tests included spliced longitudinal reinforcement. In general, rectangular columns were confined with rectangular hoops or cross ties and the circular columns were confined with spiral or circular hoops. The culmination of their research was the development of expressions that predict the expected column displacement (Δ_{calc}) at the onset of spalling and at bar buckling, see Equations 4 and 5, respectively.

$$\frac{\Delta_{spall\_calc}}{L}(\%) \cong 1.6\left(1-\frac{P}{A_g f'_c}\right)\left(1+\frac{L}{10D}\right) \qquad \text{Eq. 4}$$

$$\frac{\Delta_{bb\_calc}}{L}(\%) \cong 3.25\left(1+k_{e\_bb}\rho_{eff}\frac{d_b}{D}\right)\left(1-\frac{P}{A_g f'_c}\right)\left(1+\frac{L}{10D}\right) \qquad \text{Eq. 5}$$

Where k_{e_bb} = 40 for rectangular-reinforced columns, 150 for spiral-reinforced columns, and zero if s/d_b exceeds 6, and the remaining variables are defined in relation to Figure 17. The probability of damage can then be predicted using the fragility functions, which have been reproduced in Figure 21. The ratio of the seismic demand (Δ_{demand}) to the predicted expected displacement at the onset of damage (Δ_{cdc}) is entered as the abscissa; then the probability of the respective damage state is read as the ordinate based on fragility function. Note that if the seismic demand is equal to the calculated displacement at the onset of damage, there is a 50% probability that damage will have occurred. In general for this set of data, the fragility functions based on the normal cumulative distribution function (CDF) provide better estimates of the data than the lognormal CDF; this may not always be the case. Equation 4 has since been adopted to form the basis of the "implicit" displacement capacity expressions that are used in the AASHTO SGS to predict displacement capacity for bridges designed for SDCs B and C, as shown in Figures 4 and 5.

DAMAGE REDUCTION

Aside from the ability to predict the onset of critical damage levels, there is also a need to refine design and detailing concepts to mitigate damage within the structure. Two strategies exist, both of which rely on the design philosophy of capacity protection. The first strategy is to limit the permissible deformations in order to limit damage. This is typically done by lowering the strain (or other deformation) limit from those found in guidelines such as the AASHTO SGS. This approach can be seen in the current agency- and project-specific criteria that are reviewed in chapters eight and nine of this synthesis. The second approach is to use construction concepts that inherently reduce or prevent damage, even under large deformations and displacements. These concepts include but are not limited to the following:

- Transverse steel/confinement requirements in reinforced concrete members
- Isolation and dissipation devices
- Residual displacement control

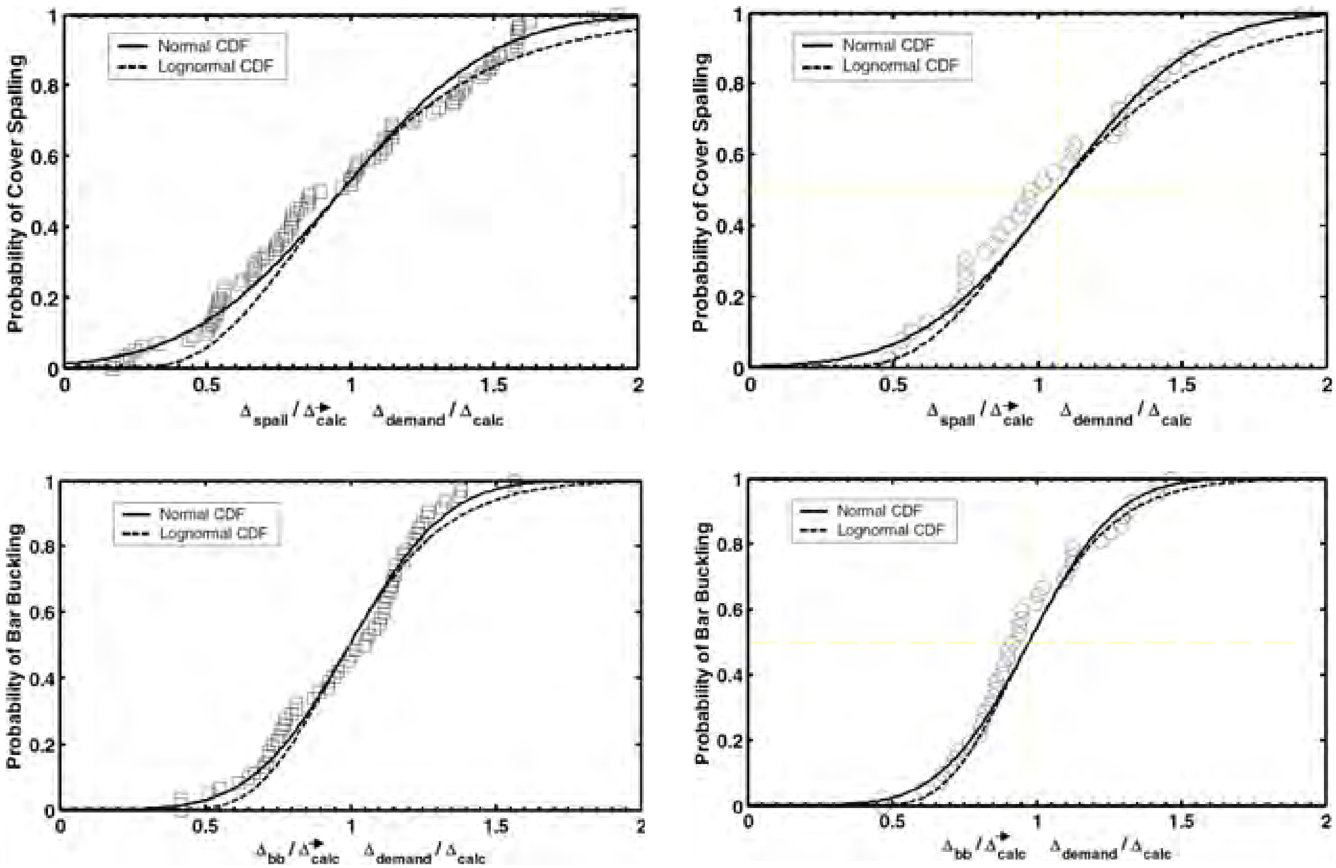


FIGURE 21 Fragility functions for the onset of spalling and bar buckling for rectangular and spirally reinforced columns.

- Damage-resistant plastic hinges
- Load path control.

These concepts will be briefly discussed to show their applicability to increasing the seismic performance of bridge structures.

Transverse Steel/Confinement Requirements

Perhaps the most complete and widely implemented application of damage mitigation concepts is that of increased transverse steel requirements in reinforced concrete columns. These provisions gradually emerged over the years subsequent to the 1971 San Fernando earthquake until the deployment of the current AASHTO seismic requirements. Experience has shown that reinforced concrete columns with low transverse steel content are susceptible to plastic hinge confinement and shear failures, resulting in significant damage and even structural collapse. The transverse steel requirements were progressively increased until finally a practical limit was reached, whereby the transverse steel content became so high as to precipitate constructability issues. Criteria such as ATC-32 (1996) pushed the limits on transverse confinement with "anti-buckling steel," where buckling of the longitudinal reinforcement was suppressed by the transverse steel so that the controlling flexural limit state of the plastic hinge was low-cycle fatigue failure of the longitudinal reinforcement. However, experience proved that too much of a good thing analytically could lead to severe reinforcement congestion. Clearly, performance at large displacements could be enhanced, but at what price? The bridge design community then settled on the transverse steel limits that are included in the current AASHTO specifications, which acknowledge that bar buckling is the likely controlling limit state of the plastic hinge.

Seismic Isolation and Energy Dissipation Devices

Another well-known application of damage mitigation is the use of isolation and energy dissipation devices, also known as base or seismic isolation, to substantially decouple the structural response from the earthquake ground motion. Such systems are also referred to as protective systems. This uncoupling is typically accomplished using a system of isolation and dissipation devices strategically located within the structure. The isolation component consists of a flexible element that elongates the structure's fundamental period of vibration or a sliding element that limits the seismic energy that enters into the structure. An energy-dissipating element or damper is then used to reduce the displacements imposed on the isolating elements to a manageable level. The theory behind these concepts is illustrated in Figure 22, which shows acceleration and displacement elastic response spectra. In general (for smooth or design spectra, short-period range excluded), as the fundamental period increases the spectral acceleration reduces while displacement increases until the constant displacement region of the response spectra is reached. In effect, the period lengthening due to seismic isolation devices trades spectral acceleration for displacement at the isolation plane (i.e., within the isolator). As these displacements can get quite large, it is then necessary to introduce supplemental damping to the system, the effect of which is shown by the family of curves in Figure 22, where ζ equals the damping as a percentage of critical. As the damping increases (larger values of ζ), the displacement can be reduced significantly. Therefore, by decoupling the structural response from the ground motion, the design actions (forces and deformations) are limited by the capacity of the isolator system, thereby preventing or reducing damage. In essence, the seismic-isolation system capacity protects the entire structure by acting as a low-strength ductile fuse. Most isolation and energy dissipation systems can absorb energy without incurring damage themselves. This aspect of the devices enables them to substantially improve structural performance over that of conventional structural systems, such as reinforced concrete frames.

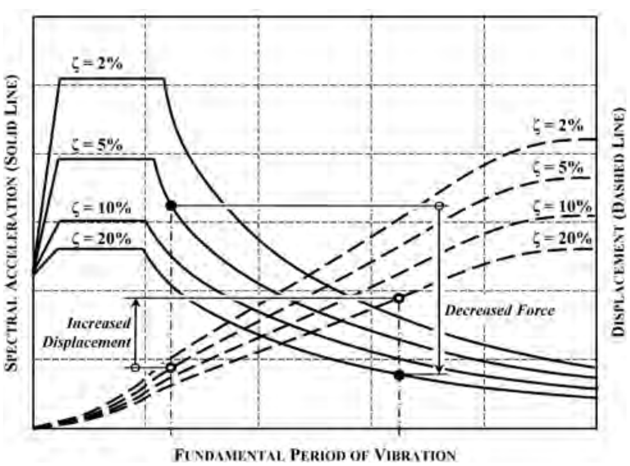


FIGURE 22 Response modification due to seismic isolation and energy dissipation devices.

The relative importance of either the period shift or the increased energy dissipation (damping) can be emphasized, depending on the structural configuration and the site conditions, leading in some cases to the exclusive use of either isolators or dampers. In general, however, structures that can accept seismic isolation and dissipation systems exhibit one or all of the following properties: (1) The bridge has stiff piers with a high natural frequency of vibration, (2) the bridge is nonregular (e.g., a combination of very short and very long columns owing to topography), and (3) the expected ground motion is well defined with a dominant high-frequency content, typical of shallow earthquakes, near fault or rock sites (Priestley et al. 1996). Damping devices typically are most effective in flexible structures such as in moment frame construction, or of course a structure that has isolation devices.

AASHTO SGS supports including protective system devices with a bridge system; they fall into the Type 3 design strategy. There is also an AASHTO Guide Specification dealing with analysis, design, and device testing for seismically isolated bridges (AASHTO 2010). The seismic isolation of bridges is often straightforward, with the isolators typically placed between the top of the columns or piers and the superstructure, thereby limiting the forces transmitted from the superstructure to the substructure. In many cases, the seismic isolators can simply replace the normal thermal expansion bearings. The application of protective system devices to bridges and buildings has reached a relatively mature level, and recommendations for the analysis and design of such structures are well documented (Skinner et al. 1993; Priestley et al. 1996; Naeim and Kelly 1999; Chopra 2007; Christopoulos and Filiatrault 2007; Priestley et al. 2007; among others) and are beyond the scope of this synthesis.

Although seismic isolation and dissipation devices might appear to be the panacea for earthquake-induced damage, several challenges still exist. First, not all structures will benefit greatly from the inclusion of protective system devices; these are primarily long-period structures, as the period elongation resulting from isolation will typically not significantly reduce the spectral accelerations. Second, the increase in displacement demand can cause practical issues such as the need for large movement joints or moats around the perimeter of the isolated portion of the structure. These can cause maintenance and aesthetic issues. Next, the elongation in the fundamental period can actually cause amplification of accelerations if soft-soil or near-fault forward-directivity (long-period pulse) effects are present. This issue arises from the concept that response spectra for individual earthquakes are not smooth as shown in Figure 22; they are actually quite jagged, and in the case of soft-soil or near-fault directivity effects the long-period accelerations may be larger than moderate-period accelerations. A poignant example of this is the 1985 Michoacán earthquake, where the deep deposit of soft lacustrine volcanic clays underlying Mexico City amplified the seismic waves significantly in the 2-second period range. In this situation, if a stiff structure was seismically isolated, the period shift could result in high spectral demands. The evaluation of site effects is therefore of critical importance in the application of seismic isolation. Furthermore, if the displacement demands of an isolated system are not properly accounted for and are underestimated, the isolation system may fail, loose stability, or run out of travel, which can impart significant accelerations to the structure.

Although seismic isolation and dissipation devices may not be the fix-all for increased seismic performance, they nonetheless provide another set of tools for mitigating earthquake-induced damage and achieving heightened seismic performance levels. If the structural configuration and site conditions permit, seismic isolation can be an effective and elegant solution for controlling structural performance.

Residual Displacement Control

The reparability and usability of a bridge after strong ground shaking is often a function of its residual displacement. In some cases, large inelastic displacement demands can result in substantial permanent offset from a plumb condition. Strategies abound for control of residual displacements, including simply limiting normal reinforced concrete column drifts. However, promising purpose-designed structural systems are being developed.

The development of unbonded post-tensioned columns to reduce these permanent deformations has seen particular attention in the past decade. Details have been developed for reinforced concrete (RC), steel-jacketed RC, and concrete-filled tube (CFT) columns. The restoring force is provided by unbonded post-tensioning steel, which is anchored into the footing and the pile cap. Energy dissipation is enhanced by including bonded or unbonded mild steel within the plastic hinge region, as described by Mahin et al. (2006) and Cohagen et al. (2009); further details are outlined in NCHRP Report 681 (Marsh et al. 2011). There are even details for RC or steel-jacketed RC segmental columns, which rely solely on longitudinal unbonded post-tensioning to develop flexural resistance. Shear resistance between the segments is developed by interface shear, with the clamping force provided by the post-tensioning and column dead load (Hewes et al. 2001). The University of Washington is currently conducting experimental tests on pretensioned concrete columns. These systems hold a great deal of promise to recenter the structure after a large seismic event, as indicated by quasi-static and dynamic testing. Although they have been used in the building community, concerns regarding long-term durability and inspection of such systems in bridges continue to prevent deployment of the concepts.

However, there are still significant challenges in the ability to predict residual displacements even for recentering systems (Mahin et al. 2006). Recent research (Yazgan 2009; Lee et al. 2010) has shown that the predicted residual displacements in RC structures are sensitive to the adopted modeling approach, fiber-discretized sections, or a predefined hysteretic rule (e.g., modified Takeda, bilinear). Yazgan (2009) noted that fiber-section elements tend to underestimate residual displacements, whereas the modified Takeda hysteresis overpredicts residual displacements. However, Lee et al. (2010) presented a method that can substantially increase the accuracy of residual displacement prediction in fiber-section elements by modifying the confined concrete constitutive model to account for the effects of imperfect crack closure on the transition between unloading in tension to reloading in compression. The constitutive model modifications follow the recommendations by Stanton and McNiven (1979). Even though prediction of residual displacements is difficult, some agencies—for example, the Japan Road Association—have adopted such strategies to help ensure reparability following strong ground shaking.

The accurate prediction of residual displacements is also critical in the determination of structural response and performance to strong aftershocks. If significant residual displacements exist, bias in the response history may initiate, causing asymmetric "ratcheting" behavior where the structure progressively "walks" towards collapse. Additionally, structures with large residual displacements may not be suitable for postevent use, as the live loading may introduce significant P-Δ forces on the already weakened structure, instigating collapse.

Damage-Resistant Plastic Hinges

Another damage control or mitigation method is the incorporation of a deformable, damage-resistant medium into an RC or prestressed concrete plastic-hinge region. These have typically been elastomeric materials placed between a concrete column member and an adjacent footing or cap beam. These concepts provide structural compliance to accommodate seismic displacements, but also prevent or delay damage to the columns.

Jellin (2008) and Stringer (2010) have conducted research on reducing damage in prestressed concrete pile connections. These connections were specifically developed for use in marine wharf and pier structures, but there is overlap for application in pile bents for bridges. Because of the need for thick cover concrete (up to 3 in.) in prestressed piles in saltwater environments, seismic performance is greatly affected by the thick cover concrete spalling away from the pile, with substantial damage to both the pile and the soffit of the deck or pile cap at low connection rotation demands. The aforementioned research was able to greatly delay pile spalling and eliminate soffit spalling by including a thin (0.5 to 0.75 in. thick) elastomeric (cotton duck or random-oriented fiber) bearing pad at the top of pile cutoff and a soft foam wrap around the perimeter of the embedded length of the pile into the cap.

Another example is the ongoing work at the University of Nevada-Reno by Saiidi (as outlined in *NCHRP Report 698*), where elastomeric bearing materials are built into the portions of columns that would normally experience plastic hinging (Marsh et al. 2011). The substitution of elastomeric material in place of the normally damage-prone RC creates a column that is much more damage resistant than conventional columns. Once such novel damage-limiting systems are available for deployment, adequate damage prediction techniques and fragility data will be needed to take such systems into PBSD. Such databases would parallel those being developed by the building community for use in PBSD of buildings, as described earlier.

Load Path Control

The final method of damage reduction is to control the lateral load path. This method includes fusible shear keys, which provide an isolating effect if proper support lengths are used, and AASHTO SGS Type 2 structures, where all inelastic action is confined to ductile steel diaphragms, whereas the rest of the superstructure and the substructure remain essentially elastic. As cross-frames and diaphragms are not direct gravity load-supporting elements, they can be repaired or replaced with relative ease. Type 2 structures are relatively new and have not been implemented in any great numbers. Research into their seismic performance is still ongoing.

POSTEVENT INSPECTION

State DOTs have recently incorporated damage analysis to aid in postearthquake bridge inspection prioritization. Both California (Turner et al. 2009) and Washington (Ranf et al. 2007) have developed systems based on the USGS ShakeMaps, Hazards U.S. (HAZUS, described in chapter seven), and regional experience. Every bridge within a region is assigned a fragility function based on the bridge geometry (span lengths, number of spans, column heights, skew), component material types, year of construction, retrofit, construction type, and regional bridge performance in past earthquakes. When a seismic event with a magnitude greater than a preset threshold occurs (typically magnitude 4.0), a ShakeMap is developed for a specified intensity level [0.3 s (Washington) or 1.0 s (California) 5% damped spectral acceleration], and the program then applies the fragility functions for every bridge within the affected area. This generates a list of bridges that may have incurred damage, including estimates of the potential damage level. Inspectors can use this list to prioritize their inspections, making as efficient use of both time and the state DOT's limited resources as possible.

Another tool developed by Caltrans is the *Visual Catalog of Reinforced Concrete Bridge Damage* (Caltrans 2006b), which relates field observations to the expected reserve capacity of the system by means of damage level definitions similar to those in Table 10. This is accomplished using photographs from more than 100 RC bridge-column laboratory tests conducted since 1990 and photographs of actual damage from 14 historic earthquakes worldwide since the early 1970s. Damage is organized according to the failure mechanism, the shape of the hysteretic backbone (ductile, strength degrading, or brittle), and the damage level. This tool is used in training both inspectors and engineers, and it assists inspectors in interpreting the damage that they see in the field. It also helps ensure that different inspectors obtain consistent results.

GEOMETRIC CONSTRAINTS AND SERVICE LEVELS

Central to the serviceability of bridges are the geometric aspects of the structure, including the approaches, the bridge roadway alignment, and the barrier configuration. Displacements that occur during or as the result of an earthquake, whether caused by the structure or the soil around the structure, may affect bridge use. Displacements that occur during the event may affect life safety, and permanent displacements that remain after the event may likewise affect the use of the bridge. Therefore, displacements, which are an EDP, affect the performance of the bridge, which in essence is a DM, from which losses, such as loss of service may be determined. These geometric constraints, which may be the result of structural damage, ground movements, or both, are necessary for evaluating the potential for service-level losses or service restrictions. Additionally, the geometric constraints would be expected to inform decision making regarding postearthquake use of a bridge.

In 2003, the ATC/MCEER Joint Venture published MCEER/ATC-49 (2003), which provided guidelines for seismic design of highway bridges. The commentary of that document included recommendations for allowable displacements relative to two service levels: immediate and significant disruption. The commentary also provided a list of potential causes for displacements and suggestions for mitigation or limitation of such displacements, shown in Table 13. The narrative that described the original table is included here verbatim, with the exception of document cross-references, which are augmented here with text in parentheses.

> Allowable displacements are constrained by geometric, structural and geotechnical considerations. The most restrictive of these constraints will govern displacement capacity. These displacement constraints may apply to either transient displacements as would occur during ground shaking, or permanent displacements as may occur due to seismically induced ground failure or permanent structural deformations or dislocations, or both. The magnitude of allowable displacements depends on the desired performance level of the bridge design. The following paragraphs discuss the geometric constraints that should be considered in establishing displacement capacities. It should be noted that these recommendations are order of magnitude values and are not meant to be precise. Structural and geotechnical constraints are discussed in (the higher seismic requirements of) Sections 7 and 8.
>
> Allowable displacements shown in Table C3.2-1 [Table 13] were developed at a Geotechnical Performance Criteria Workshop conducted by MCEER on September 10 & 11, 1999 in support of the NCHRP 12-49 project. The original intent of the workshop was to develop detailed foundation displacement criteria based on geotechnical constraints. The final recommendation of the workshop was that, except in special circumstances, foundations are able to accommodate large displacements without strength degradation and that displacement capacities are usually constrained by either structural or geometric considerations. The values in the table reflect geometric constraints and are based largely on judgment

TABLE 13
BRIDGE GEOMETRIC CONSTRAINTS ON SERVICE LEVEL (MCEER/ATC, 2003)

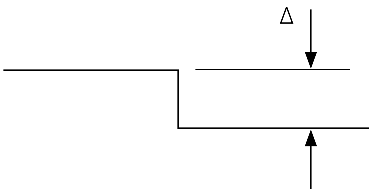
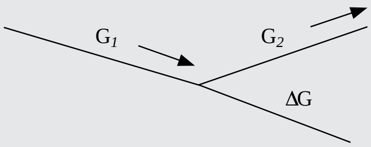
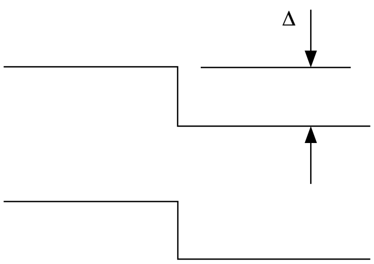
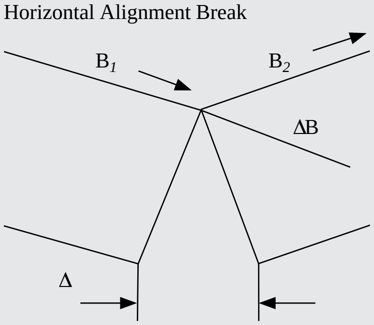
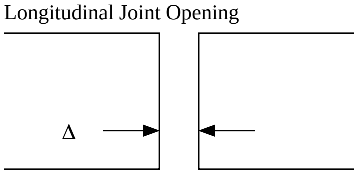
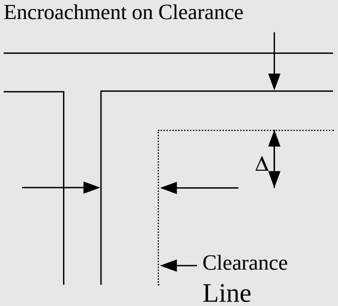
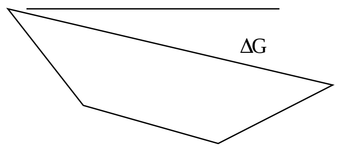
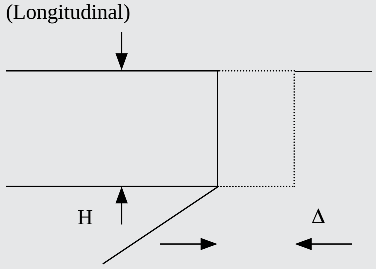
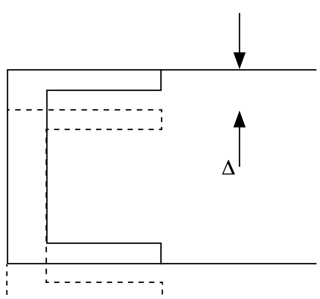
Permanent Displacement Type	Possible Causes	Mitigation Measures	Immediate	Significant Disruption
Vertical Offset 	• Approach fill settlement • Bearing failure	• Approach slabs • Approach fill stabilization • Bearing type selection	0.083 ft. (0.03 m)	0.83 ft. (0.2 m) (To avoid vehicle impact)
Vertical Grade Break 	• Interior support settlement • Bearing failure • Approach slab settlement	• Strengthen foundation • Bearing type selection • Longer approach slab	Use AASHTO "Green Book" requirements to estimate allowable grade break	None
Horizontal Alignment Offset 	• Bearing failure • Shear key failure • Abutment foundation failure	• Bearing type selection • Strengthen shear key • Strengthen foundation	0.33 ft. (0.1 m) Joint seal may fail	Shoulder width (To avoid vehicle impact)
Horizontal Alignment Break 	• Interior support failure • Bearing failure • Lateral foundation movement	• Strengthen interior support • Bearing type selection • Strengthen foundation	Use AASHTO "Green Book" requirements to estimate allowable alignment break	None Δ = 3.28 ft. (1.0 m)
Longitudinal Joint Opening 	• Interior support failure • Bearing failure • Lateral foundation movement	• Strengthen interior support • Bearing type selection • Strengthen foundation	0.33 ft. (0.1 m)	3.28 ft. (1.0 m) (To avoid vehicle impact)
Encroachment on Clearance 	• Foundation settlement • Lateral foundation movement • Bearing failure	• Strengthen foundation • Bearing type selection	Δ (Actual Clearance)	Depends on facility being encroached upon

Table 13 continued on p.53

Table 13 continued from p.52

Permanent Displacement Type	Possible Causes	Mitigation Measures	Immediate	Significant Disruption
Tilting of Cross-Section 	• Interior support settlement • Bearing failure • Approach slab settlement	• Strengthen foundation • Bearing type selection • Longer approach slab	$\Delta G = .001$ radians	None
Movement into Abutment Fill (Longitudinal) 	• Engagement of abutment backfill due to horizontal movement of superstructure	• Increase gap between superstructure and abutment backwall • Stiffen interior supports • Increase amount of fill that is engaged	$\Delta = .02H$	No Constraint Controlled by Adjacent Seat Width
Movement through Abutment Fill (Transverse) 	• Transverse movement of strengthened or supplemental interior wingwalls through approach fill	• Isolate transverse movement with sacrificial shear keys and/or isolation bearings • Increase transverse strength and stiffness of abutment	$\Delta = .02H$	No Constraint

Notes:
Geometric constraints, with the exception of longitudinal and transverse movement through abutment fill, usually apply to permanent displacements which may be difficult to predict accurately. Therefore, the constraints in this table shall be taken as order of magnitude values.

The AASHTO publication "A Policy on Geometric Design of Highways and Streets" (otherwise known as the "Green Book") specifies criteria for determining vertical curve length based on site distance. This criteria, which is based on design speed and whether the curve is a "crest" or a "sag" can be used to determine the allowable change in grade resulting from support settlement. A curve length equal to the sum of adjacent spans may be used in the case of a continuous superstructure or a zero curve length may be used in the case of adjacent simply supported span lengths. Bridge owners may also wish to consider the AASHTO recommendations on appearance and driver comfort in establishing allowable grade changes.

In the case of horizontal curves, minimum curve radius is usually controlled by superelevation and side friction. These radii are specified in the AASHTO "Green Book". When lateral displacement of an interior support results in an abrupt angle break in horizontal alignment a vehicle shall be able to safely achieve the desired turning radius at design speed within the provided lane width minus a margin of safety at each edge of the lane. Consideration shall also be given to the opening of the expansion joint at the edge of the bridge. Joint seals may be damaged at the immediate service level. If no damage at the seal is desired the designer should check the actual longitudinal and transverse capacity or reduce some of the permissible movements.

that represents the consensus opinion of the workshop participants.

Geometric constraints generally relate to the usability of the bridge by traffic passing on or under it. Therefore, this constraint will usually apply to permanent displacements that occur as a result of the earthquake. The ability to repair, or the desire not to be required to repair, such displacements should be considered when establishing displacement capacities. When uninterrupted or immediate service is desired, the permanent displacements should be small or nonexistent, and should be at levels that are within an accepted tolerance for normally operational highways of the type being considered. A guideline for determining these displacements should be the AASHTO publication "A Policy on Geometric Design of Highways and Streets". When limited service is acceptable, the geometric constraints may be relaxed. These may be governed by the geometry of the types of vehicles that will be using the bridge after an earthquake and by the ability of these vehicles to pass through the geometric obstruction. Alternately, a jurisdiction may simply wish to limit displacements to a multiple of those allowed for uninterrupted service. In the case of a no collapse performance objective, when liquefaction occurs, post-earthquake use of the bridge is not guaranteed and therefore no geometric constraints would be required to achieve these goals. However, because life safety is at the heart of the no collapse requirement, jurisdictions may consider establishing some geometric displacement limits for this performance level for important bridges or those with high ADT. This can be done by considering the risk to highway users in the moments during or immediately following an earthquake. For example, an abrupt vertical dislocation of the highway of sufficient height could present an insurmountable barrier and thus result in a head-on type collision that could kill or severely injure occupants of the vehicle. Usually these types of geometric displacement constraints will be less restrictive than those resulting from structural considerations and for bridges on liquefied sites it may not be economic to prevent significant displacements from occurring.

CHAPTER SEVEN

LOSS ANALYSIS

BACKGROUND

The final step in the performance-based design process is to quantify the estimated loss associated with specific seismic events. Many loss metrics are of use to stakeholders and decision makers:

- How many people will be killed or injured?
- What use can the structure support?
  - Not available for any use
  - Available to emergency vehicles only
  - Available for conventional unrestricted use
- How long will the structure be out of operation?
- How much will it cost to repair?
- What is the expected financial loss over the life of the structure due to closure?
- What is the cost-benefit ratio for different structural systems?
- What is the likelihood that the structure will be damaged beyond repair in its design life?
- What is the mean annual frequency of exceeding a given financial loss?
- What is the expected performance during aftershocks of various severities?

These are all important questions to the stakeholders, and they can be boiled down to three primary categories: deaths, dollars, and downtime. The definition and prediction of losses to date has largely been subjective and qualitative, with little or no explicit quantitative accuracy. Therefore, there is a need to define, in discrete or continuous terms, the risk of incurring certain losses associated with seismic hazards of varying intensity.

To bridge the gap between expected damage and a decision variable (DV), a loss model is needed, the specifics of which are largely dependent on the expression or type of loss considered. Loss functions defining the relationship between a damage measure and the incurred death, dollar, or downtime losses can take many forms. For example, if the probability of closure after an event with a specific intensity is considered, then the loss model should relate the probability of closure by a bridge inspector given a certain amount of visual damage. However, if the probability of exceeding a certain repair cost (in proportion to the replacement value of the structure) is of interest, then a loss model relating damage to repair cost is required. How the probability of a DV value given an intensity measure, $P(DV|IM)$, is numerically evaluated depends on the desired expression of loss.

Several of the loss expression permutations, as described by Moehle and Deierlein (2004), are shown in Figure 23. They include the following:

a. The likely loss in a single scenario (Figure 23a)
b. The loss with a certain probability of exceedance (Figure 23b)
c. The losses associated with a continuum of scenarios (Figure 23c)
d. The probability of exceeding a given level of losses in a set period of time (Figure 23d).

Note that Figure 23 relates loss in terms of financial cost, as denoted by L_{BLDG} [$]; however, the two remaining loss categories (deaths and downtime) could be easily substituted.

Kunnath and Larson (2004) describe an example of a loss model, which relates the probability of closure given column spalling and bar buckling based on a survey conducted by Porter (2004), and the column fragility relationships of Berry and Eberhard (2003). The example is useful in understanding how loss calculations are made and how they are used.

In the survey by Porter, DOT bridge inspectors were asked about specific DMs, such as column concrete spalling, column longitudinal bar buckling, approach slab settlement, and abutment damage. They were asked whether they would the leave the bridge open, limit traffic (weight, capacity, and speed), or close the bridge if the damage types listed were observed after an earthquake. According to the survey; 33% of inspectors responded that they would close the bridge if column spalling was observed, and 100% responded that they would close the bridge if longitudinal bar buckling was observed.

Because spalling or bar buckling either occurs or does not occur, the associated loss models are binary (i.e., yes or no). This will not be the case if a continuous loss variable, such as replacement cost ratio, is used. The loss models can therefore be expressed as:

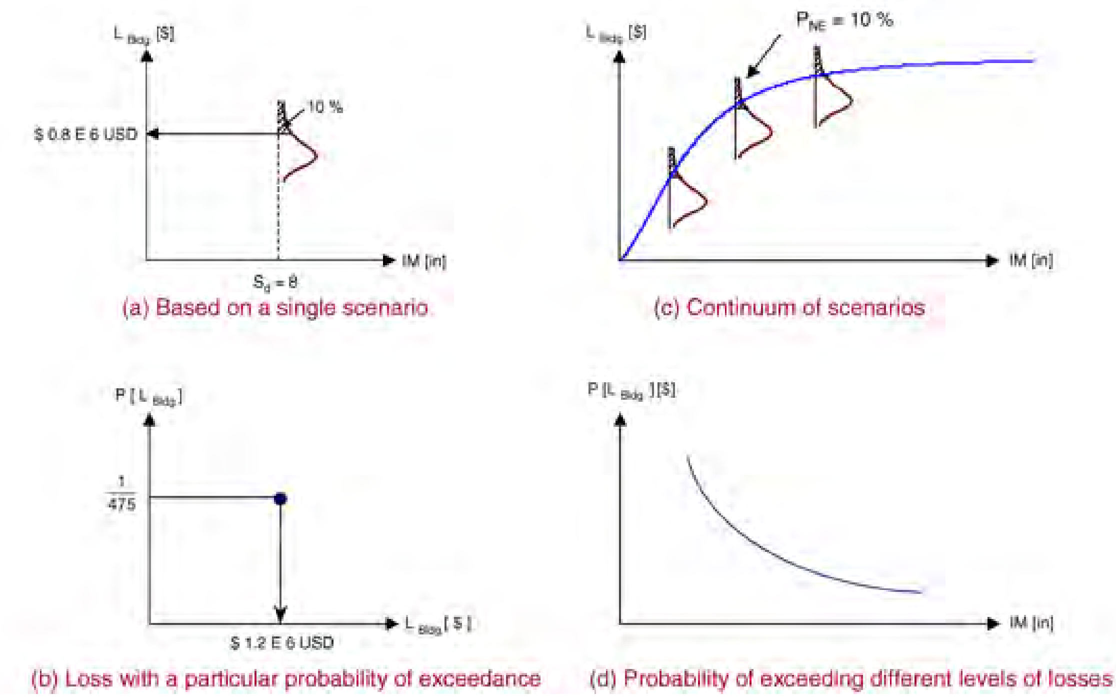


FIGURE 23 Expressions of loss (Moehle and Deierlein 2004).

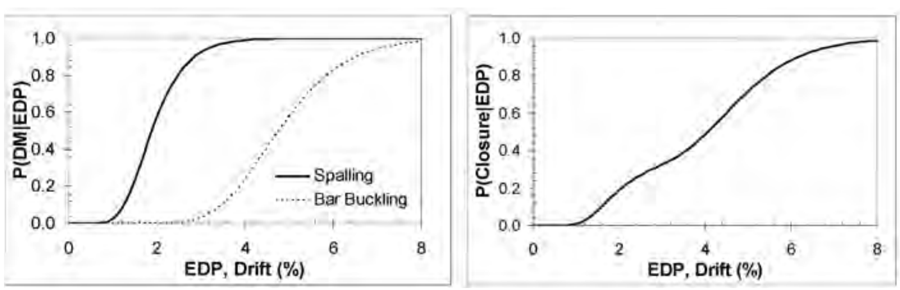


FIGURE 24 Probability of bridge closure (Kunnath and Larson 2004).

$$P(DV\,|\,DM_1) = P(Closure\,|\,Spalling) = 0.33$$

$$P(DV\,|\,DM_2) = P(Closure\,|\,BarBuckling) = 0.33$$

The first case would be read as "the probability of closure, given that spalling was observed is 33%."

These loss models can then be incorporated into a loss fragility relating an EDP to the probability of closure. This is done by integrating the loss model into the damage fragility relationship. The continuous integration of the DV given an EDP for all possible values of the EDP can be solved numerically using the two discrete DMs given earlier.

$$P(DV\,|\,EDP) = \int_{-\infty}^{\infty} P(DV\,|\,EDP)\,dP(EDP) = \sum_{i=1}^{2} P(DV\,|\,DM)\,P(DM\,|\,EDP) \quad \text{Eq. 6}$$

Although Equation 6 is initially daunting, the mathematics is actually simple. The fragility functions are factored by their contribution to the total probability of closure, and then summed along the domain. To do this, the spalling fragility function is factored by (0.33 / (0.33 + 1.00) = 0.25) and the bar buckling fragility is factored by (1.00 / (0.33 + 1.00) = 0.75). The total probability of closure is then determined by summing the factored fragility functions at each value of the EDP. This results in a continuous loss fragility that determines the probability of closure given a specific EDP, in this case defined as the drift angle of the column as shown in Figure 24b.

If this type of analysis is to be used in practice, then software similar to the PACT calculation package under development for buildings, discussed earlier, will need to be developed.

LOSS AND RISK ASSESSMENT SOFTWARE

HAZUS

The HAZUS earthquake methodology is a loss estimation procedure for the primary use by state, regional, and community governments to evaluate the wide range of losses associated with natural disasters such as earthquake, flood, and hurricane. The goal of the program is to provide a basis for preparedness and disaster response to aid in planning for mitigation and the reduction of future losses (Kircher et al. 2006). HAZUS uses geographic information systems (GIS) technology to estimate the physical, economic, and social impacts of disasters and provides "spatial relationships between populations and other more permanently fixed geographic assets or resources for the specific hazard being modeled, a crucial function in the pre-disaster planning process" (HAZUS 2012).

HAZUS uses six primary interdependent modules to quantify the losses incurred by a region due to a seismic event, as shown schematically in Figure 25. The description of the HAZUS modules is summarized from Kircher et al. (2006). Because of the modular nature of the HAZUS framework, the degree of sophistication can be modified depending on the detail the user requires, allowing the use of simplified models and limited inventory data or more refined models and detailed inventory data. The framework also permits the user to focus on specific losses if time, resource, or scope restrictions are present.

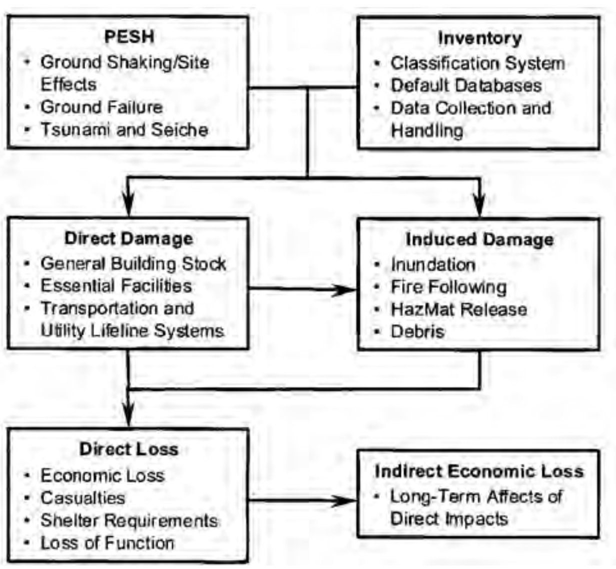


FIGURE 25 HAZUS earthquake modules (Kircher et al. 2006).

The loss estimation process begins with the selection of an earthquake hazard, such as a scenario earthquake defined by fault type, magnitude, and location. Users can also input GIS-based maps of peak ground acceleration, peak ground velocity, or spectral response. The first module that is used is the potential earth science hazards (PESH), which defines the potential hazard from strong ground shaking, ground failure (landslide, fault rupture, and liquefaction), and tsunami/seiche. This is then combined with the inventory module, which describes the physical infrastructure and demographics within the region of interest. This includes the general building stock, essential facilities (e.g., hospital, police and fire stations), transportation systems, and utility lifelines. The infrastructure components are defined using extensive default databases that can be modified to account for specific local conditions.

Damage is then quantified using two modules, direct and induced damage. The direct damage module estimates the damage sustained by infrastructure components based on the ground motion intensity or failure. This is accomplished using fragility relationships, as described previously. Induced damages are quantified as the secondary consequences of the earthquake, such as fire, inundation as a result of dam/levee failure, hazardous material release, and debris generated by damaged or collapsed structures.

Finally, losses are related to damage in terms of direct and indirect losses. Direct losses include economic loss owing to damage and repair or replacement of the structure, casualties, and loss of use (deaths, dollars, and downtime). Indirect losses detail the long-term economic and social losses associated with a damaged built environment, such as changes in employment and personal income.

Results generated using the HAZUS earthquake system have been verified by comparing the predicted losses with those actually observed during the 1994 Northridge, California, earthquake using the ShakeMap input. In general, the mean and maximum losses determined using HAZUS were able to bound the actual observed losses, with the exception of casualties and serious injuries, which were over- and underpredicted by a factor of 2, respectively. Overall, HAZUS was able to provide a reasonably accurate prediction of losses during the Northridge earthquake. Further details can be found in Kircher et al. (2006).

REDARS

In much the same way as HAZUS seeks to determine expected losses to the built environment as a result of strong ground shaking, MCEER and FHWA have developed the REDARS software specifically for loss analysis of transportation and roadway systems. The intention is to provide a tool to inform earthquake hazard mitigation planning. The following description of the highway system assessment software REDARS (MCEER 2012) was provided by Yen (2010).

> Earthquakes are inevitable natural hazards with the potential for causing large numbers of fatalities and injuries, major property and infrastructure damage, and serious disruption of everyday life. However, a systematic risk assessment process can help keep earthquake losses to a minimum. This methodology—called risk management—is a process for determining which hazards should be addressed, what priority they should be given, what should be done, and what countermeasures should be used.

Earthquake damages to highway infrastructure can go well beyond human safety and the cost of repairs. Such damage also can disrupt traffic flows and therefore affect a region's emergency response and economic recovery. Impacts depend not only on the seismic performance of the highway components, but also on the highway network's configuration, including highway redundancies, traffic capacities, and the links between interstates and arterial roads.

State DOTs usually do not consider these factors in their risk reduction activities. One reason is lack of a technically sound and practical tool for estimating impacts. Therefore, beginning in the late 1990s, FHWA sponsored multiyear seismic research projects for developing and programming REDARS (Risks from Earthquake DAmage to Roadway Systems) software, released for public use in 2006.

REDARS is a multidisciplinary tool for seismic risk analysis of highway systems nationwide. For any given level of earthquake, REDARS uses state-of-knowledge models to estimate seismic hazards (ground motions, liquefaction, and surface fault rupture); the resulting damage (extent, type, and location) for each component in the highway system; and repairs that might be needed to each component, including costs, downtimes, and time-dependent traffic (that is, the component's ability to carry traffic as the repairs proceed over time after the earthquake).

REDARS incorporates these traffic states into a highway network link-node model to form a set of system-states that reflect the extent and spatial distribution of roadway closures at various times after the earthquake. REDARS then applies network analysis procedures to each system-state in order to estimate how these closures affect systemwide travel times and traffic flows. Finally, REDARS estimates corresponding economic losses and increases in travel times to and from key locations or along key lifeline routes. Users can apply these steps for single earthquakes with no uncertainties (deterministic analysis) or for multiple earthquakes and in estimates of seismic hazards and component damage (probabilistic analysis).

Although REDARS adequately replicated the performance of the highway system in the San Fernando Valley during the Northridge Earthquake, much work still needs to be done to enable engineers to use the methodology with confidence. Indeed, the researchers developed REDARS with the expectation that new and more sophisticated modules will be developed over time to improve its accuracy and expand its range of application.

Chapter eight includes an example of the use of REDARS by the Oregon DOT.

CHAPTER EIGHT

ORGANIZATION-SPECIFIC CRITERIA FOR BRIDGES

This chapter reviews example seismic design performance criteria for several DOTs, plus FHWA's seismic retrofit criteria and those of several international agencies responsible for development of earthquake design criteria. The summaries focus on the stated performance and seismic hazard aspects of the project and not on the details of the analysis and design to achieve the stated performance goals. Comparisons of various organization-specific criteria can be found in chapter ten.

CALIFORNIA (CALTRANS)

Caltrans has long been the domestic leader in the seismic design of bridges, and was the first state DOT to develop performance-based criteria. The Caltrans SDC formed the basis of the AASHTO SGS, resulting in many similarities between the two documents. The seismic design methodology for all Caltrans bridges is outlined in the *Memo to Designers 20-1* (Caltrans 2010b), in which bridges are classified as important or ordinary based on the requirements for postearthquake operability (i.e., whether the bridge is part of an access pathway to emergency facilities), the economic impact of prolonged closure because of damage, and whether the bridge is defined as critical by a local emergency plan. Ordinary bridges are then further classified as either standard or nonstandard based on structural geometry and layout, framing (nonstandard examples are bridges with outrigger or C-bents, unbalanced mass and stiffness, or multiple superstructure types), and geological conditions (e.g., soft soil, near fault, and liquefaction potential). Based on the classification of the bridge, performance criteria are defined as shown in Table 14, along with the pertinent definitions.

TABLE 14

CALTRANS SEISMIC PERFORMANCE CRITERIA (CALTRANS 2010B)

Bridge Category	Seismic Hazard Evaluation Level	Postearthquake Damage Level	Postearthquake Service Level
Important	Functional	Minimal	Immediate
	Safety	Repairable	Limited
Ordinary	Safety	Significant	No collapse

Definitions:

Functional Level Evaluation: A project-specific hazard level will be developed in consultation with the Seismic Safety Peer Review Panel as defined in MTD20-16. Ordinary bridges are not designed for functional evaluation seismic hazards.

Safety Level Evaluation: For ordinary bridges, this is the "design earthquake" as defined below. For important bridges, the safety evaluation ground motion has a return period of approximately 1,000–2,000 years.

Design Earthquake is the collection of seismic hazards at the bridge site used in the design of bridges. The "design earthquake" consists of the design spectrum as defined in the SDC Version 1.6, Appendix B, and may include other seismic hazards such as liquefaction, lateral spreading, surface faulting, and tsunami.

Damage Levels:

- *Minimal:* Essentially elastic performance.
- *Repairable:* Damage that can be repaired with a minimum risk of losing functionality.
- *Significant:* A minimum risk of collapse, but damage that could require closure to repair.

Service Levels:

- *Immediate:* Full access to normal traffic is available almost immediately following the earthquake.
  Limited: Limited access (e.g., reduced lanes, light emergency traffic) is possible within days of the earthquake. Full service is restorable within months.
- *No collapse:* There may be no access following the earthquake.

Ordinary standard bridge design is governed by the *Seismic Design Criteria* (Caltrans 2010a). The design ground motion is taken as the maximum of a probabilistic, 5% in 50-year seismic event (975-year return period), and a deterministic ground motion characterized by the largest median response from the maximum rupture of any fault within the vicinity of the bridge. A minimum ground motion is also imposed as the median spectrum generated by a magnitude 6.5 earthquake on a strike slip fault 12 km from the site.

Acceptable performance is defined as no collapse under this single-level design earthquake. The deformation capacity is defined using material strain limits and maximum displacement ductility demands, which are defined based on the structural configuration of the bridge. Analysis can be either an equivalent static analysis (force-based linear elastic analysis) or an equivalent dynamic analysis (linear elastic multimodal analysis), depending on the complexity and configuration of the bridge. Nonlinear static (pushover) analyses are used to determine the displacement capacity of the bridge.

Performance-Based Features

- Single-level performance criteria:

  - 1,000-year event with deterministic limits, no collapse
  - Limits maximum concrete, mild steel, and prestressing steel strains, which are the same as the SGS.

Ordinary nonstandard and important bridges are designed according to project-specific criteria based on the procedures outlined in the *Memo to Designers 20-11* (Caltrans 1999). This procedure will include a peer review that consists of Caltrans staff or an external seismic safety review panel or both, depending on the project requirements and Caltrans staff expertise. In general, peer reviews are required for projects that include the following:

- Important bridges
- Nonstandard bridge types, enhanced performance requirements, or exceptions to Caltrans seismic design standards for projects where the seismic technical expertise is not available within Caltrans
- Projects where there is a determination of a public need for external peer review.
- Projects where it is required by permitting agencies.

Examples of project-specific criteria developed through Caltrans can be found in chapter nine.

OREGON (ODOT)

The Oregon Department of Transportation (ODOT) requires use of the AASHTO SGS displacement-based seismic design procedures. A two-level approach is taken in seismic design, where the AASHTO 1,000-year return period (actual is 975 years) is used for a no-collapse check and a 500-year earthquake (actual 475 years) is used for a serviceability check. The 500-year event reflects the desire to have serviceable bridges to aid in response and rescue after a Cascadia Subduction Zone event, which has an average return period of 300 to 350 years. The ODOT displacement-based checks use modified strain limits from those given in the AASHTO SGS. They are summarized here.

Performance-Based Features

- Two-level performance criteria
- New design
  - 1,000-year, no collapse
    - Limits maximum concrete strain to 90% of SDC D limit in SGS
    - Reinforcing steel strain limits the same as SGS
  - 500-year, serviceable
    - Concrete strain limited to 0.005
    - Reinforcing steel strain limited to two times the strain hardening strain in SGS
- Seismic isolation design is permitted
- Retrofit design
  - 1,000-year, performance levels same as FHWA *Retrofitting Manual*
  - 500-year, performance levels increased to PL2 from PL3 for most cases.

Additionally, ODOT (2009) conducted studies of its more heavily traveled corridors with REDARS to simulate damage to bridges within its network. From this estimated damage and delay, costs were generated for major highways in western Oregon, where most of the earthquake damage was expected. Most of Oregon's bridges were built between 1950 and 1980, and in recent years Oregon's seismic hazard has increased dramatically in the western part of the state because of its proximity to the Cascadia Subduction Zone (CSZ), which is thought to be capable of generating earthquakes as large as magnitude 9.0. The studies considered both megaquakes in the CSZ and smaller earthquakes that could occur in the crust or plate interface below western Oregon. The study found that many of the north-south routes [for example, the Interstate 5 (I-5) corridor in the Willamette Valley and the coastal highway (US-101)] would likely be severed by a large earthquake. East-west highways connecting the coastal areas with the Willamette Valley would also likely be damaged for some time after a large earthquake.

To provide some protection against the risk posed by the earthquake environment in Oregon, ODOT has specified the two-level design criteria listed previously for new bridges. These criteria provide some mitigation of seismic hazard for new construction, particularly for smaller events and the CSZ. Additionally, the two-level criteria for retrofit assessment, as outlined in the FHWA *Retrofitting Manual*, have been enhanced to use a 500-year or 15% chance of exceedance earthquake in 75 years for the lower-level event. However, because retrofit funding is difficult to develop, few retrofit projects have been undertaken in Oregon.

SOUTH CAROLINA (SCDOT)

The South Carolina Department of Transportation (SCDOT) uses a set of displacement-based criteria that were developed specifically for use in the state. It does not use or permit use of either of the AASHTO design methods. The SCDOT criteria in use today have their origins in the MCEER/ATC-49 recommendations (2003), but SCDOT modified the earthquake hazard for the lower, functional-level earthquake to a 462-year (500-year nominal) return period (15% in 75 years chance of exceedance) from the 108-year (100-year nominal) return period (50% in 75 years chance of exceedance) of MCEER/ATC-49. The upper, safety-level seismic hazard was maintained at a 2,475-year (2,500-year nominal) return period (3% in 75 years chance of exceedance). However, as implemented, the criteria are most similar to the Caltrans SDC and the AASHTO SGS specifications. The SCDOT

(2008) set of criteria is one of the more complete performance-based criteria available today. The SCDOT criteria were first released in 2001, with the second edition released in 2008.

Tables 15 through 20 provide examples of the SCDOT criteria. These criteria include definitions of operational classification, analysis requirements, performance objectives, damage-level objectives by component type, displacement performance limits, and maximum ductility demands.

TABLE 15

OPERATIONAL CLASSIFICATION (SCDOT 2008)

Operational Classification (OC)	Description
I	All bridges that are located on the Interstate system or along the following roads: US-17, US-378 from SC-441 east to I-95 I-20 Spur from I-95 east to US-76 US-76 from I-20 Spur east to North Carolina Additionally, all bridges that meet any of the following criteria: Structures that do not have detours Structures with detours greater or equal to 15 miles Structures with a design life greater than 75 years
II	All bridges that do not have a bridge OC = I and meet any of the following criteria: A projected (20 years) ADT ≥ 500 A projected (20 years) ADT < 500, with bridge length longer than 180 ft. or individual span length larger than 60 ft.
III	All bridges that do not have an OC = I or II classification.

TABLE 16

ANALYSIS REQUIREMENTS (SCDOT 2008)

Operational Classification (OC)	Analysis Description•
I, II	Seismic analysis shall be performed for the following design earthquakes: Functional evaluation earthquake (FEE) only when potential liquefiable soil or slope instability (see Geotechnical Design Manual for more information) exists and no geotechnical mitigation is performed. Safety evaluation earthquake (SEE).
III	Seismic analysis required for SEE only.

• For design requirements of temporary bridges and staged construction, see Section 3.11. For design requirements for pedestrian bridges, see Section 3.12.
Detailed seismic analysis is not required for SDC A or single span bridges; however, minimum detailing shall be provided, see Section 3.13.1.

TABLE 17

PERFORMANCE OBJECTIVES (SCDOT 2008)

Design Earthquake	Performance Level	Operational Classification (OC) I	II	III
Functional Evaluation Earthquake (FEE[2])	Service	Immediate	Maintained	See Note 2
	Damage	Minimal	Repairable	See Note 2
Safety Evaluation Earthquake (SEE)	Service	Maintained	Impaired	Impaired
	Damage	Repairable	Significant	Significant

1. Higher level seismic performance objectives may be required by SCDOT.
2. Analysis for FEE not required for OC III bridges

TABLE 18

DAMAGE-LEVEL OBJECTIVES (SCDOT 2008)

Bridge Component		Design Earthquake	Operational Classification (OC) I	II	III
Superstructure		FEE[4]	Minimal	Minimal	See Note 4
		SEE	Minimal	Minimal	Minimal
Connection Components[1]		FEE[4]	Repairable	Repairable	See Note 4
		SEE	Significant	Significant	Significant
Interior Bent Restraint Components[2]		FEE[4]	Minimal	Minimal	See Note 4
		SEE	Minimal5	Minimal[5]	Minimal[5]
End Bent Restraint Components[2]		FEE[4]	Minimal	Minimal	See Note 4
		SEE	Significant	Significant	Significant
Capacity-Protected Components[3]		FEE[4]	Minimal	Minimal	See Note 4
		SEE	Minimal	Minimal	Minimal
Substructure	Single-Column Bents	FEE[4]	Minimal	Repairable	See Note 4
		SEE	Repairable	Significant	Significant
	Multicolumn Bents	FEE[4]	Minimal	Repairable	See Note 4
		SEE	Repairable	Significant	Significant
	End Bent Piles	FEE[4]	Minimal	Repairable	See Note 4
		SEE	Minimal	Significant	Significant
	End Bent Wing Walls	FEE[4]	Minimal	Repairable	See Note 4
		SEE	Significant	Significant	Significant
	Pile Bents	FEE[4]	Minimal	Repairable	See Note 4
		SEE	Repairable	Significant	Significant
	Pier Walls Weak Axis	FEE[4]	Minimal	Repairable	See Note 4
		SEE	Repairable	Significant	Significant
	Pier Walls Strong Axis	FEE[4]	Minimal	Minimal	See Note 4
		SEE	Minimal	Minimal	Repairable

1. Include expansion joints and bearings.
2. Include shear keys, retainer blocks, anchor bolts, dowel bars.
3. Include bent caps, footings, oversized shafts.
4. Analysis for FEE not required for OC III bridges.
5. When shear keys are designed not to fuse.

TABLE 19
DISPLACEMENT PERFORMANCE LIMITS (SCDOT 2008)

Bridge System	Design Earthquake	Operational Classification (OC) I	II	III
Expansion Joints (Longitudinal Differential Displacement) (inches)[1]	FEE[3]	0.015L	0.020L	See Note 3
	SEE	0.025L	0.040L	0.050L
Expansion Joints (Transverse Differential Displacement)	FEE[3]	2"	4"	See Note 3
	SEE	4"	6"	8"
Integral/Semi-Integral End Bents (Longitudinal Displacement)	FEE[3]	2"	4"	See Note 3
	SEE	4"	8"	12"
Integral/Semi-Integral End Bents (Transverse Displacement)	FEE[3]	2"	4"	See Note 3
	SEE	4"	8"	12"
Freestanding End Bents (Longitudinal Displacement)	FEE[3]	1"	2"	See Note 3
	SEE	3"	6"	8"
Freestanding End Bents (Transverse Displacement)	FEE[3]	2"	4"	See Note 3
	SEE	4"	8"	12"
Interior Bents—Fixed Bearings (Longitudinal Displacement) (inches)[2]	FEE[3]	0.075H	0.100H	See Note 3
	SEE	0.300H	0.400H	0.500H
Interior Bents—Expansion Bearings (Longitudinal Displacement) (inches)[2]	FEE[3]	0.050H	0.075H	See Note 3
	SEE	0.200H	0.300H	0.400H
Interior Bents (Transverse Displacement) (inches)[2]	FEE[3]	0.075H	0.100H	See Note 3
	SEE	0.250H	0.400H	0.500H

1 "L" is the total expansion length at the joint. "L" shall be input in feet, with the results being in inches.
2 The displacements are measured at the top of bents. "H" is the height measured from top of cap to top of footing, or point of fixity of drilled shaft/driven pile. Input "H" in feet, with the result being in inches.
3 FEE not required for OC III bridges.
• Displacement limits shall not exceed the minimum support length as described in Section 9.1.

Performance-Based Features

- Permissible displacement limits for structural and nonstructural elements based on operational classifications (OCs) from I to III
- Damage-level objectives for bridge components based on OCs from I to III
- Two-level performance criteria
  – 2,500-year, maintained service as limited by ductility demand based on OC
  – 500-year, serviceable as limited by ductility demand based on OC.

The OC definition is based on the location of the structure, traffic volume, design life, and available detour options, and dictates the remaining performance criteria. OC I structures have the most restrictive design requirements in order to maintain critical corridors throughout a region. Furthermore, the SCDOT criteria use a holistic methodology whereby the performance criteria for bridge approaches are classified according to the bridge criteria; therefore the bridge and the approach structures have congruent performance objectives.

TABLE 20
MAXIMUM DUCTILITY DEMANDS (SCDOT 2008)

Bridge System		Design Earthquake	Operational Classification (OC) I	II	III
Superstructure		FEE	1.0	1.0	See note
		SEE	1.0	1.0	1.0
Substructure	Prestressed Concrete Pile Interior Bents	FEE	2.0	4.0	See note
		SEE	4.0	8.0	8.0
	Prestressed Concrete Pile End Bents	FEE	1.0	4.0	See note
		SEE	2.0	8.0	8.0
	Single-Column Bents	FEE	2.0	3.0	See note
		SEE	3.0	6.0	8.0
	Multicolumn Bents	FEE	2.0	3.0	See note
		SEE	4.0	8.0	8.0
	Pier Walls Weak Axis	FEE	2.0	3.0	See note
		SEE	3.0	6.0	8.0
	Pier Walls Strong Axis	FEE	1.0	1.0	See note
		SEE	1.0	1.0	2.0

Note: Analysis for FEE is not required for OC III bridges.

The design criteria in South Carolina are under continuing review and improvement. SCDOT has been active in both producing design examples for use of the specifications and in soliciting input from users through a partnership with the American Council of Engineering Companies (ACEC) (Mesa 2012). Because the criteria, as outlined in Tables 15 through Table 20, are relatively new, SCDOT is continuing to determine if unworkable or inconsistent results are occurring on projects where the criteria are used. In particular, there are issues that arise with substructure types, such as pile bents, that are widely used in South Carolina, but that are not addressed in detail in national specifications. The provisions for such structure types are examples of the continual improvement that SCDOT is attempting.

JAPAN ROAD ASSOCIATION

Following the 1995 Hyogo-ken Nanbu (Kobe) earthquake, the Japan Road Association (JRA) reexamined its 1990 specification, reviewed the guide specifications that were used for repair and reconstruction of bridges following the Kobe event and the applicable research that had been recently published, and issued a fully revised edition of its seismic design specifications in 1996 (JRA 1996). Dr. Kazuhiko Kawashima led the Special Sub-Committee on Seismic Countermeasures for Highway Bridges established to develop this new edition.

The new edition included two types of seismic hazard: Type I—“plate boundary type large-scale earthquakes” and Type II—“inland direct strike type earthquakes.” The plate boundary earthquakes were described as those such as the 1923 Great Kanto earthquake, and today would be reflective of the 2011 Great East Japan (Tohoku) earthquake. The inland direct-strike type events were postulated to be similar to the Kobe event and of a magnitude of about 7. The specifications included no data on the return period or chance of exceedance of these ground motions.

Bridges in Japan are divided into two groups of importance depending on the bridge’s function within the transportation system. Class A bridges are of standard importance and Class B bridges are of high importance. Class B bridges are those of the

> national expressways, urban expressways, designated city expressways, Honshu-Shikoku Bridge Highway and general national highways. Class B also includes double-section (double-deck) bridges and overbridges of (those crossing) prefectural highways and municipal roads, and other bridges, and highway viaducts.

All other bridges are Class A.

Additionally, guidance is given in the specification to consider the bridge’s role in regional disaster prevention plans, the traffic volume that the bridge carries and whether alternative routes are feasible, and whether the cost and time of recovery would be excessive. However, no quantitative guidance is provided for making these decisions.

Two levels of performance are required to be considered for seismic design. For smaller earthquakes, bridges should be “designed not to lose their integrity,” and this is further clarified to mean “no damage.” Table 21 summarizes the seismic hazard and performance objectives. A force-based coefficient method is provided in the specification to cover smaller earthquakes. For Class A bridges there must be no “fatal damage” (e.g., collapse) in the larger earthquake, and for Class B bridges damage is restricted to localized limited damage in the larger earthquake. These performance requirements apply to both types of ground motion, Types I and II.

The specifications require ductile design, and the ductile design checks are performed only for the upper-level ground motion, as indicated in Table 21. The design checks are of a displacement-based methodology, and (for example) RC ductile design considers, quantitatively, the confinement effect of transverse steel on section ductility. For many bridge types (e.g., RC and concrete-filled steel piers), permissible ductility demands are provided based upon the importance classification. This requirement ensures that damage is limited as appropriate for the class of the bridge. Seismic isolation is also directly considered in the JRA seismic specification.

An additional check that is required as part of the JRA upper-level ground motion (Types I and II) is of residual displacement. The check is made pier by pier and is based on the calculated ductility demand of the pier. The residual displacement is a function of the spectral acceleration coefficient, ultimate strength of the pier, post-yield stiffness, the construction type—RC, steel, and so on—and the tributary seismic weight.

The permissible residual displacement is limited to 1% of the height of the pier, unless a special study is done to develop a more refined value on a project-specific basis. The reasoning given for the 1% residual drift is reparability, because it was found after the 1995 Kobe event that bridges with larger residual drifts were difficult or cost prohibitive to repair. Specifically, the commentary cites cases where the residual drifts were larger than 1/60 (1.7% drift) or exceeded

TABLE 21

JAPAN ROAD ASSOCIATION SEISMIC MOTIONS AND TARGET SEISMIC PERFORMANCE (JRA 1996)

<table>
<tr><th colspan="2" rowspan="2">Ground Motion to Be Taken into Account in Seismic Design</th><th></th><th colspan="2">Target Seismic Performance of Bridge</th><th>Seismic Calculation Method</th></tr>
<tr><th>Bridge of standard importance (bridge of class A)</th><th>Bridge of high importance (bridge of class B)</th><th>Static analysis method</th><th>Dynamic analysis method (bridge with complicated seismic behavior)</th></tr>
<tr><td colspan="2">Ground motion highly probable to occur during service period of bridge</td><td colspan="2">No damage</td><td>Seismic coefficient method</td><td rowspan="3">Time history response analysis
Response spectrum analysis</td></tr>
<tr><td rowspan="2">Ground motion with high intensity, though less probable to occur during the service period of the bridge</td><td>Ground motion Type I (plate boundary type large-scale earthquake)</td><td rowspan="2">To prevent fatal damage</td><td rowspan="2">To limit damage</td><td rowspan="2">Ductility design method</td></tr>
<tr><td>Ground motion Type II (inland direct strike type earthquake like Hyogo-ken Nanbu earthquake)</td></tr>
</table>

a total permanent displacement greater than 15 cm, and such cases proved quite difficult to repair and to provide adequate new bearing support lengths. Accordingly, the residual drift limit was set at 1/100 or 1%.

The JRA residual drift provision is an unusual requirement not typically seen in design specifications, but at least in concept is a reasonable one. The AASHTO specifications have no limits on displacement other than indirect controls required to meet the P-Delta limits or material strain limits (in the case of the SGS), and none of those limits are specifically included to address postearthquake function or repair of a bridge.

In summary, the JRA specification has a two-level seismic hazard, although the two levels are not presented in the same format as the AASHTO seismic hazards. The hazard is classified by tectonic source and deterministic historic occurrence rather than by probabilistic methods. The importance of bridges to emergency response and to economic viability of the regions is considered when parsing bridges into two distinct importance groups. The importance then seeks to control the amount of damage that is permitted within the structure, although direct quantification of damage is not a part of the method; instead, as with AASHTO, engineering design parameters, such as strain and ductility form the metrics for evaluation of structural performance. The important exception is the residual displacement or drift limit, which was added after the experience gained from Kobe, which is intended to render a bridge repairable.

EUROCODE

The Eurocode 8 Part 2—Seismic Design of Bridges (EC8-2) (Eurocode 2008) specification governs the seismic design of bridges for the 22 national members of the Comité Européen de Normalisation (CEN). As with many of the national seismic design provisions, ductile response is the primary seismic philosophy. The EC8-2 seismic design procedure for bridges is basically predicated on a force-based design approach, but a displacement check is required if irregularity requirements are not met. In this usage, irregularity results when ductility demands are not kept relatively equal between yielding components. If some components are predisposed by structure configuration to have greater ductility demands while others have lower demands—the ratio being greater than a factor of 2—then an inelastic displacement capacity check must be made, which is similar to the AASHTO SGS procedure. Otherwise, a force-based approach similar to the AASHTO LRFD method is acceptable. Capacity design and minimum detailing are enforced for all structures.

The EC8-2 seismic design procedure uses a single-level seismic hazard, which can be set by the country using the specification, although the seismic hazard is usually taken as a 475-year return period or a ground motion with 10% chance of exceedance in 50 years.

The performance objectives are noncollapse in the design event and minimization of damage in a smaller event. Only the design event is generally checked during design. As with the AASTHO LRFD and SGS, adequate performance in the smaller event is inferred, but not checked.

The performance anticipated in the design event is that "the bridge should retain its structural integrity and adequate residual resistance, although at some parts of the bridge considerable damage may occur" (Kolias 2008). The bridge should be "damage-tolerant," meaning that following damage induced by the design earthquake, the "structure can sustain the actions from emergency traffic, and inspections and repair can be performed easily." The EC8-2 goes on to state that "the non-collapse requirement for bridges under the design seismic event is more stringent that the relevant requirement for buildings, as it contains the continuation of emergency traffic."

The EC8-2 recognizes two classes of structures—ductile behavior structures and limited ductile behavior structures—with the delineation between the two being a system ductility demand factor of 1.5. Detailing is controlled to some extent by whether a structure is of ductile or limited-ductile behavior. The force reduction factor is based upon whether a structure is ductile or limited ductile, and the application of the factor is similar to the way R is treated in the AASHTO force-based method. An importance factor is also applied, which adjusts the ground motion up or down by 1.30 or 0.85, respectively, depending on whether the bridge has greater than average or less than average importance.

Overall, the EC8-2 borrows many concepts from the AASHTO seismic design procedures and is in many ways comparable to the AASHTO methodologies, although the force-based and displacement-based procedures are combined into one specification. Although the ground motions are within the purview of the country with jurisdiction, generally they are less than the AASTHO ground motions, since a 475-yr return period is used most often.

CHAPTER NINE

PROJECT-SPECIFIC CRITERIA

This chapter reviews example seismic design performance criteria for several major projects that have been completed or are under way (as of 2012) in the United States. The summaries focus on the stated performance and seismic hazard aspects of the project, not on the details of the analysis and design to achieve the stated performance goals. Comparisons of various project-specific criteria can be found in chapter ten.

CALTRANS—WEST APPROACH SEISMIC RETROFIT OF SAN FRANCISCO–OAKLAND BAY BRIDGE PROJECT

The west approach of the San Francisco–Oakland Bay Bridge (SFOBB) is located in San Francisco on the extreme west end of the bridge that crosses from San Francisco to Oakland. The final seismic retrofit criteria were issued in 2002 (Caltrans 2002) as a result of criteria suggested by the Governor's Board of Inquiry (following the Loma Prieta earthquake), the Caltrans Seismic Advisory Board, and Caltrans itself.

The general design philosophy behind the criteria allows for controlled inelastic action, provided such action is consistent with the performance objectives. Inelastic action could be limited to predetermined locations that can be made ductile or that can have their displacements controlled by special components or devices, such as isolators or dampers. Such locations should be accessible for inspection, repair, or replacement without limiting the functionality of the bridge. In-ground damage is permissible provided the excavation for inspection and repair is compatible with the functionality requirements.

To ensure capacity protection, upper-bound forces are calculated for the yielding or ductile elements, and these forces are used to check the strength of adjacent non-ductile or force-controlled elements. The capacities of force-controlled elements must be based on nominal material properties.

The performance criteria are as follows:

- Two-level seismic hazard criteria
  - Safety evaluation earthquake (SEE)—approximately 1,000- to 2,000-year return period
  - Functional evaluation earthquake (FEE)—300-year return period
- Performance criteria
  - Following an SEE earthquake there should be immediate access for emergency vehicles and full access to normal traffic within 72 hours.
    - √ There may be repairable damage, such that:
      - Inelastic response limits the damage limited such that lateral displacement capacity is maintained following the maximum credible event.
      - Lateral strength may be reduced immediately following the earthquake due to failure of shear keys, wing walls, and other nonductile sacrificial members.
      - Damage can be repaired such that the lateral load carrying capacity can be returned to its original strength.
      - There is no reduction in the vertical load carrying capacity.
    - √ Acceptable damage may be described as:
      - Cover concrete may crack and spall.
      - The core of well-confined concrete columns may crack, but repairs will be limited to epoxy injection of these cracks.
      - Main column reinforcing steel may yield, but will not buckle or rupture.
      - Joints may crack, but repairs shall be limited to epoxy injection and patching of cracks.
      - Footing, superstructure, and bent cap members shall remain essentially elastic.
  - Following an FEE earthquake there should be immediate access to all normal traffic.
    - √ There should only be minimal damage, consisting of minor inelastic response.
    - √ Acceptable damage may be:
      - Minor cracking or spalling of column cover concrete may occur but should be avoided if economically possible.
      - Narrow cracking of cast-in-drilled-hole pile shaft cover concrete may occur, but the cover concrete will not spall.
      - Main column reinforcing steel may yield.
      - Original geometry is essentially maintained, with columns nearly plumb.

CALTRANS—ANTIOCH TOLL BRIDGE SEISMIC RETROFIT PROJECT

Antioch Toll Bridge Seismic Retrofit Project—Final Design Report was issued in March 2011 (Caltrans 2011). The bridge is located east of the San Francisco Bay Area and carries SR-160 over the San Joaquin River. The main 8,650-ft-long structure comprises two-column piers supported on driven piles, and this substructure arrangement supports two steel plate girders that are continuous over the piers. The bridge has five main span frames.

The structure has a relatively low average daily traffic (ADT) count of 15,000 vehicles per day, compared with typical values of 100,000 to 200,000 closer to major population centers in the Bay Area. The bridge was built in 1978 and evaluated for retrofit following the Loma Prieta earthquake in 1989, and owing to its recent construction date, the bridge did warrant seismic retrofit by the evaluation process in place at that time. In 2004 the bridge was reevaluated and found sufficiently deficient to warrant seismic retrofit. Retrofit designs were completed, and in 2010 a construction contract was awarded for the retrofit work. The project was substantially completed in 2011.

The seismic retrofit criteria were based on the bridge's low ADT and, accordingly, only an SEE was selected for the retrofit. This earthquake had a return period of 1,000 years, and the single-level performance criteria included a no-collapse damage state, with limited damage in the supporting piles, deck joints, abutment shear keys, and abutment backwalls. Although design of new structures uses a philosophy of restricting damage to inspectable and accessible areas for repair, for retrofit such a philosophy may not be economical. Thus, the objective was to permit damage in some piles of the main span (typically the outermost piles), but retain enough undamaged piles in the core of the pile groups to ensure that the gravity capacity of the structure is maintained. This design approach is intended to prevent collapse. Such a strategy of permitting foundation damage was judged appropriate because of the low ADT of the bridge.

CALTRANS—VINCENT THOMAS TOLL ROAD SEISMIC RETROFIT PROJECT

The Vincent Thomas Bridge on Route 47 over the Los Angeles River in Los Angeles County is a cable suspension structure (Caltrans 1996). The seismic retrofit design criteria were developed in 1996 and were based on the remaining useful life of the bridge being 150 years, which is twice the life assumed in the AASHTO specifications for a typical bridge. The ADT of the bridge in 1993 was 38,000, and the projected 2015 ADT was 38,700.

The performance criteria used to design the seismic retrofit are as follows:

- Two-level seismic hazard criteria
  - SEE—84% probability of not being exceeded during the remaining 150-year service life (return period of approximately 950 years)
  - FEE—60% probability of not being exceeded during the remaining 150-year service life (return period of approximately 285 years)
- Performance criteria
  - After an SEE event, limited service is acceptable. Limited access (reduced lanes, light emergency traffic) is to be available within days. Normal traffic access is to be available within months. No collapse is the limiting damage state that must be provided. Suspension span stiffening trusses were permitted to experience small (25% over yield) inelastic ductility demands.
  - After an FEE event, full access to normal traffic is available almost immediately. Repairable damage is acceptable. Repairable damage is defined as that which can be repaired with a minimum risk of losing functionality.

The bridge crosses the Palos Verdes fault, which underlies the Los Angeles River channel. Specific consideration of near-fault effects and fault rupture horizontal and vertical displacements were considered, and included maximum fault displacements in relation to the return period.

SOUTH CAROLINA DEPARTMENT OF TRANSPORTATION—COOPER RIVER BRIDGE (RAVENEL BRIDGE) PROJECT

The Cooper River Bridge, also known as the Arthur Ravenel Jr. Bridge, is a cable-stayed bridge completed in 2005 that carries US-17 into Charleston, South Carolina. The overall performance objective for the bridge was to obtain a 100-year service life. The bridge is highly important to the communities on either end of the structure, Charleston and Mount Pleasant. The bridge is classified as a critical bridge under the seismic criteria, which corresponds to OC I in South Carolina's seismic design criteria (SCDOT 2008).

The bridge is required to provide secondary life safety. The time that it would be closed after a large earthquake, should it be damaged, would produce a severe economic impact on the region, and the bridge is formally part of the local emergency response plan (SCDOT 2006). For these reasons, the bridge is designated with the most restrictive OC.

To functionally address the need to keep the bridge open after a major earthquake, but recognizing that not all structures within the entire bridge needed to be functional, SCDOT designated some structures as critical access path (CAP). Examples of CAP structures are the main spans, the high- and low-level approach structures, and one ramp enter-ing and one ramp exiting the bridge on each end. The CAP structures have higher performance requirements than non-CAP structures. Table 22 shows the performance criteria for the bridge.

TABLE 22
SEISMIC PERFORMANCE CRITERIA FOR COOPER RIVER BRIDGE (SCDOT 2002)

Seismic Hazard	Critical Access Path (CAP) Structures	Non-CAP Structures (Ordinary Bridge)
Functional evaluation earthquake (FEE) 500-year earthquake	Service Level—Immediate access Damage Level—Essentially elastic response (minimal damage)	Service Level—Limited access Damage Level—Limited, repairable damage
Safety evaluation earthquake (SEE) 2,500-year earthquake	Service Level—Functional (open to emergency vehicles following inspection) Damage Level—Repairable	Service Level—No collapse Damage Level—Significant (major) damage. Damage may not be repairable

Additional clarifying performance requirements were established for the bridge. For instance, in the FEE for CAP structures, the bridge should be opened immediately after inspection, which could occur within hours, and no reduction should occur in the number of lanes available.

For the SEE and CAP structures, the structures were designed to incur only limited ductility demands, although detailing to produce full ductility capacity was provided. This provides a displacement margin of safety, and the factor was set at 2/3 (i.e., the usable displacement capacity was set at 2/3 of the full ductility capacity).

The FEE and non-CAP structures should open after inspection, although lane restrictions may be necessary, meaning that not all lanes may be open. However, the load-carrying capacity of the open lanes should not be reduced below the normal carrying capacity.

For the SEE and non-CAP structures, there shall be no collapse, while significant damage is permitted. These structures would be designed as full-ductility structures, and the full-ductility capacity that they could provide may be used in the earthquake.

WASHINGTON STATE DEPARTMENT OF TRANSPORTATION—STATE ROUTE 520 BRIDGE PROJECT

The State Route 520 (SR-520) Evergreen Point Floating Bridge and Landings project between Seattle and Medina, Washington, uses project-specific essential bridge design criteria developed by WSDOT (WSDOT 2010). The project comprises a new floating bridge across Lake Washington with fixed approach structures on the west and east ends. SR-520 is one of two major corridors across Lake Washington between Seattle and Bellevue and other east-side communities and carries approximately 100,000 vehicles per day.

The SR-520 floating bridge project uses an enhancement to the AASHTO SGS that seeks to achieve seismic performance whereby the facility would be open to emergency vehicles immediately following the design seismic event. The criteria also state that the bridge shall be open for security, defense, and economic purposes immediately after the design earthquake and shall be open to all traffic within days after a design event.

The performance criteria are as follows:

- Single-level seismic hazard criteria—7% chance of exceedance in 75 years ground motion, which is the same as AASHTO, approximately a 1,000-year seismic hazard
- Performance criteria
  - Immediately following the design earthquake, the bridge shall be open to emergency traffic, and open to all traffic within days.
  - “The extent and amount of damage should be sufficiently limited that the structure can be restored to essentially its pre-earthquake condition without replacement of reinforcement or structural members. Repair should not require complete closure. Replacement of secondary members may be allowed if it can be done under traffic. Secondary members are those which are not a part of the gravity load resisting system” (WSDOT 2010).
  - Displacement capacity of the lateral load-resisting system is assessed with strain limits that are reduced below those given in the AASHTO SGS. These limits are 2/3 of the concrete strain that would be permitted by the SGS and steel strains for A706 of 0.060 and 0.050 for #4 to #10 bars and #11 to #18 bars, respectively. These strains reflect a permissible strain that is 50% of the minimum elongation permitted for A706 by ASTM.

OREGON AND WASHINGTON DEPARTMENTS OF TRANSPORTATION—COLUMBIA RIVER CROSSING PROJECT

The Columbia River Crossing (CRC) is a joint project between WSDOT and ODOT to replace the aging and under-capacity twin bridges that I-5 uses to cross the Columbia River and join Portland, Oregon, and Vancouver, Washington. The planning and design are under way, with construction scheduled to begin in 2013 and finish in 2020. Approach structures and landside bridges on both the Washington and

Oregon sides of the Columbia River are designed according to ODOT and WSDOT bridge design manuals and AASHTO SGS. Project-specific design criteria for the main bridge have been developed (ODOT/WSDOT 2008).

The performance criteria are as follows:

- Two-level seismic hazard
  - SEE—approximately 2,500-year return period
  - FEE—approximately 500-year return period
- SEE
  - The structure must not collapse and component damage is restricted as listed here:
    - √ Piers/columns—repairable damage
    - √ Superstructure and pier caps—no damage
    - √ Piles/drilled shafts—minimal damage
    - √ Pile/shaft caps—minimal damage
    - √ Bearing and shear keys—repairable damage
    - √ Expansion joints—significant damage
- FEE
  - The structure should perform such that minimal damage is incurred with no permanent offsets.

The final design of the main CRC bridge is required to be verified by nonlinear response history analyses. The input ground motions must account for spatial variation (multiple support excitation) of the ground motion along the length of the bridge structure based on wave passage, wave scattering/incoherency, and local site response effects. A minimum of three 3-component ground motions are to be used, with the maximum response in each orthogonal direction defining the design actions on structural components. Performance acceptance criteria are based on material strain limits and minimum component curvature ductility capacities.

TENNESSEE DEPARTMENT OF TRANSPORTATION—HERNANDO DE SOTO BRIDGE—INTERSTATE 40 BRIDGE PROJECT

The Hernando de Soto Bridge carries I-40 across the Mississippi River at Memphis, Tennessee. The bridge was built in the 1960s with little seismic protection. Because of the bridge's importance to the regional economy, the Tennessee Department of Transportation and Arkansas State Highway and Transportation Department, with assistance from FHWA, performed a seismic assessment, which led to seismic retrofit in 1992. The performance objective for the seismic retrofit was to keep the bridge serviceable following a maximum probable "contingency-level" earthquake (2% chance of exceedance in 50 years—approximately 2,500-year return period) (Jaramilla 2004).

CHAPTER TEN

SUMMARY OF ORGANIZATION AND PROJECT-SPECIFIC CRITERIA

This chapter summarizes the organization- and project-specific criteria reviewed and discussed in chapters eight and nine. Criteria will be compared for trends, consensus, and differences. Summary tables of material strain limits used in bridge and waterfront structure performance-based guidelines and codes also will be are provided and discussed.

SUMMARY OF CRITERIA

Table 23 provides summary information for the organizational criteria that have been reviewed in chapter eight, and Table 24 provides information for specific projects discussed in chapter nine.

The narrative on seismic hazard and performance objectives and the summary tables make it clear that no general consensus exists regarding criteria. One reason for the apparent lack of consensus is that AASHTO criteria were changing and improving during the years that are covered by the tables.

The AASHTO seismic criteria evolution started in 1990 with a single-level 500-year return period. Then, a 2,500-year ground motion was proposed by the MCEER/ATC 49 project, which first included a two-level approach (including a 108-year lower level), but the final product contained only a single-level hazard at 2,500 years. Finally, in the late 2000s AASHTO settled upon a single-level, 1,000-year seismic hazard. When comparing the entries in the tables, the trends that somewhat follow what AASHTO was using or considering at a given time are evident.

This lack of consensus is also caused by the fact that no single solution is best for all projects. This leads to agencies such as Caltrans using a peer review process to develop appropriate criteria for each major project. This process develops performance criteria on factors such as ground motion hazard, bridge type, bridge use, local emergency needs, availability of alternate routes, and the local or regional economy.

The data in the preceding tables has been further summarized and combined with other metrics discussed earlier in the report in Table 25. This table provides a compre-

TABLE 23
SEISMIC DESIGN CRITERIA FOR AGENCIES/ORGANIZATIONS

Organization	Year	Ground Motion[1]	Damage	Performance	Ground Motion[2]	Damage	Performance
Caltrans Ordinary	2010	NA	NA	NA	SEE Maximum of 5% in 50 yr (1,000-yr RP) and deterministic ground motion	Significant, no collapse	Impaired
Caltrans Important	2010	FEE project-specific defined by peer review panel	Minimal	Immediate	SEE project-specific defined by peer review panel	Repairable	Limited
SCDOT Operational Class I	2008	FEE 15% in 75 yr (500-yr RP)	Minimal	Immediate	SEE 3% in 75 yr (2,500-yr RP)	Repairable	Maintained
SCDOT Operational Class II	2008	FEE 15% in 75 yr (500-yr RP)	Repairable	Maintained	SEE 3% in 75 yr (2,500-yr RP)	Significant	Impaired
SCDOT Operational Class III	2008	NA	NA	NA	SEE 3% in 75 yr (2,500-yr RP)	Significant	Impaired
ODOT	2011	15% in 75 yr (500-yr RP)	Minimal	Open in 72 hours	7% in 75 yr (1,000-yr RP)	Significant	Impaired

Notes:
1. Return periods shown are approximate.
2. If no percent exceedance is provided, then none was provided in the source data.
FEE = Ffunctional Eevaluation Eearthquake; SEE = Ssafety Eevaluation Eearthquake.
Terms are those used by the agencies.
NA = not available.

TABLE 24
SEISMIC DESIGN CRITERIA FOR VARIOUS PROJECTS

Project and Agency	Year	Ground Motion[1]	Damage	Performance	Ground Motion[2]	Damage	Performance
Cooper River Bridge SCDOT Critical Access Path	2000	FEE 15% in 75 yr (500-yr RP)	Minimal	Immediate	SEE 3% in 75 yr (2,500-yr RP)	Repairable	Functional; emergency vehicles
SR 520 Floating Bridge WSDOT	2011	NA	NA	NA	7% in 75 yr (1,000-yr RP)	Repairable	Maintained
West Approach SFOBB Caltrans	2002	FEE 40% in Life (300-yr RP)	Minimal	Immediate to all vehicles	SEE 1,000–2,000-yr RP	Repairable	Immediate; emergency vehicles
Antioch Toll Bridge Retrofit Caltrans	2010	N/A (Low ADT)	NA	NA	SEE 1,000-yr RP	Significant, No collapse	Impaired
Vincent Thomas Bridge Retrofit Caltrans	1996	FEE 40% in 150 yr (285-yr RP)	Repairable	Immediate	SEE 16% in 150 yr (950-yr RP)	Significant	Emergency vehicles within days
Columbia River Crossing, ODOT & WSDOT	2008	FEE 15% in 75 yr (500-yr RP)	Minimal	NR	SEE 3% in 75 yr (2.500-yr RP)	Significant No collapse	NR
I-40 Bridge, Mississippi River TDOT	1992	NA	NA	NA	2% in 50 yr (2,500-yr RP)	Minimal	Serviceable

Notes:
[1] Return periods shown are approximate.
[2] If no percent exceedance is provided, then none was provided in the source data.
FEE = functional evaluation earthquake; SEE = safety evaluation earthquake.
Terms are those used by the agencies.
NA = not available, NR = not reported.

hensive summary of the data presented in this synthesis, organized by damage descriptors along the top and seismic hazard (in terms of return period) down the side. The damage descriptors were taken from visual catalog developed by Caltrans (2006b) and Hose and Seible (1999), which were discussed in the Damage States section of chapter six. Additionally, stated damage descriptors in the criteria source documents have been included in the table, although some interpretation was required to place the information in the cells. There is some ambiguity in this process in terms of where one level or definition stops and the other begins. The performance in terms of damage, reparability, and operational are actually continua, not discrete steps that can be absolutely put into definitive cells. This table melds many descriptors in order to develop a perspective on performance objectives used by different agencies and on different major projects. No effort has been made to bring in international data, such as Japan's data, because of difficulties in assigning return periods.

The damage descriptors in the table are intended to apply at a system level for a bridge, although the detailed descriptions of damage are keyed to the ductile (energy-dissipating) elements in a conventional RC substructure. For ordinary bridges, most criteria permit damage in the upper-level event to approach, but not extend into, the collapse region (DL V). In this table, the line between DLs IV and V represents the transition in the strength-degrading region of performance that would precede collapse. This would physically correspond to buckling and fracture of bars and loss of confinement as a result of the rupture of transverse reinforcement.

Several trends are evident in the table and in the description of its development. The general trend of increasing rigor or improvement in response for more important structures is similar to the conceptual layout of Figure 6 that was developed by SEAOC in the Vision 2000 document. For a given structure, the diagonal down and to the right represents more damage in larger earthquakes, and the diagonal down and to the left represents increases in design controls to minimize damage for more important structures. Also evident is a convergence of criteria toward the 1,000-year return period for either the single-level criteria or the upper level of two-level criteria. Two-level criteria are common for more important structures, but also for the FHWA *Retrofitting Manual*, SCDOT, and ODOT. One can also see a trend toward the use of longer return periods for those geographic locations where the seismic hazard at 2,500 years is much higher than that at 500 or 1,000 years (i.e., the central and eastern United States).

TABLE 25

COMBINED PERFORMANCE, DAMAGE, AND HAZARD FOR SELECTED AGENCY- AND PROJECT-SPECIFIC CRITERIA

	Damage Level	I	II	III	IV	V
Damage Descriptors	Classification	None	Minor	Moderate	Life safety	Collapse
	Damage Description	None	Minimal	Repairable	Significant	Collapse
	Physical Description (Reinforced Concrete Elements)	Hairline cracks	First yield of tensile reinforcement	Onset of spalling	Wide cracks extended spalling	Bar buckling bar fracture confined concrete crushing
	Displacement Ductility	$\mu_\Delta \le 1$		$\mu_\Delta = 2$	$\mu_\Delta = 4$ to 6	$\mu_\Delta = 8$ to 12
Repair	Reparability	None/no interruption	Minor repair/ no closure	Repair/limited closure	Repair/weeks to months closure	Replacement
Performance Descriptors	Availability	Immediate open to all traffic		Open to emergency vehicles only	Closed	
	Performance Level	Fully operational		Operational	Life safety	Collapse
	Retrofit Manual	PL3		PL2	PL1	NA
		Agency or project-specific criteria are shown below				
Seismic Hazard Return Period	100-yr RP	RM-E RM-S				
	300-yr RP	VTR SFOBB-WA				
	500-yr RP	SC-OC I	SC-OCII	ODOT CRC		
	1,000-yr RP	LRFD-C		LRFD-E SGS B/C* RM-E	LRFD-O SGS-D RM-S CA-SDC ODOT* VTR Antioch SR520* SFOBB-WA*	
	2,500-yr RP	I-40 MR (isolated)		LRFD-C SC-OC I	SC-OC II SC-OCI II CRC	

Key:

LRFD-O—AASHTO LRFD Spec Ordinary. SC-OC1—SCDOT Operational Class I also Cooper River CAP structures.
LRFD-C—AASHTO LRFD Spec Critical. SC-OCII—SCDOT Operational Class II.
LRFD-E—AASHTO LRFD Spec Essential. SC-OCIII—SCDOT Operational Class III.
SGS-D—AASHTO SGS SDC D also Caltrans SDC. ODOT—Oregon BDM.
SGS-B/C—AASHTO SGS SDC B&C Implicit Eqns. VTR—Vincent Thomas Bridge retrofit LA River, CA.
CA-SDC—Caltrans Seismic Design Criteria.
SFOBB-WA San Francisco Oakland Bay Bridge West Approach retrofit.
RM-S—FHWA Retrofit Manual Standard >50 yr. Antioch—Antioch Bridge San Joaquin River, CA.
RM-E—FHWA Retrofit Manual Essential >50 yr. I-40 MR—I-40 retrofit Mississippi River, TN.
CRC—Columbia River Crossing WA/OR.

Damage Descriptors—Caltrans (2006b), Hose and Seible (1999), and cited agency/project-specific criteria. Note that the Life-Safety classification and performance level have been moved to correspond to Damage Level IV to match actual practice.

Note: " * " indicates that the stated criteria would lie between delineations in table. For instance, SGS C would lie between B and D, and the ODOT, SR-520, and SFOBB-WA criteria would lie on the lower end of the Damage Level within which they are shown within (e.g., DL III.5).

It is difficult to put all the damage descriptors and criteria together in one table without some ambiguity or inconsistencies. This is because the terminology used in various criteria and reconnaissance and laboratory work is not always consistent; nor are the statements of performance objectives. Therefore, an effort to develop a consistent description of these terms on a national level would be useful. A large amount of data are already available, but what remains to be done is a synthesis of those data into a document that could be used consistently by the U.S. bridge engineering community as a whole. Such a project is beyond the scope of this synthesis.

MATERIAL STRAIN LIMITS

Earlier in this synthesis, the use of material strain limits to determine flexural damage limit states was discussed. Following is a summary of strain limits specified for new design in bridge and marine/waterfront performance-based codes and guidelines. This summary provides a brief survey documenting which flexural damage states have been incorporated and the specific strain values associated with each. The following tables address strain limits for tension in A706 mild reinforcement (Table 26), tension in prestressing steel (Table 27), compressive concrete strain (Table 28), and structural steel pipe piles (Table 29). These four material categories are used in the codes and guidelines to determine the flexural deformation limits for reinforced and prestressed concrete, and structural steel (pipe pile) beam-columns.

Within each table are columns listing the agency producing the code or guideline, the year of publication, the seismic hazard in terms of probability of exceedance and return period, the structural component, and location of plastic hinging. There are also columns showing whether the code or guideline explicitly relates the strain limit to a specific damage level (yes or no), then a column describing the associated damage state. If the code or guideline associates an explicit damage state to the strain limit, then the damage state is provided. In cases where the code or guideline does not explicitly relate the strain limit and damage state, the authors have provided an inferred damage state based on the overall intent of seismic design philosophy described in the code or guideline, related research, and engineering judgment. In some cases it was difficult to infer the intended damage state.

Several observations can be made regarding the strain limit tables. First, most of the codes and guidelines surveyed do not explicitly relate the strain limit to the specific damage state the strain limit is intended to prevent. For example, the strain limits for steel pipe piles do not have clear links between strain and damage. Do the limits prevent pipe bulging, buckling, or tearing? Damage can only be inferred with the information provided within the code or guideline. If a damage state is not linked to the strain, it is difficult for

TABLE 26
TENSION STRAIN LIMITS—MILD REINFORCING STEEL (A706 - GRADE 60)

Agency	Year	Ground Motion	Component and Location	Explicit Damage State?	Damage State	Strain Limit (in./in.)	
AASHTO SGS	2011	7% in 75 yr (1,000-yr RP)	RC column plastic hinge	No	Bar fracture	#4 - #10	0.090
						#11 - #18	0.060
SCDOT	2008	3% in 75 yr (2,500-yr RP)	RC column plastic hinge	No	Bar fracture	#4 - #10	0.090
						#11 - #18	0.060
Caltrans	2010	5% in 50 yr (1,000-yr RP)	RC column plastic hinge	No	Bar fracture	#4 - #10	0.090
						#11 - #18	0.060
Priestley et al.	2007	Serviceability	RC column plastic hinge	Yes	Crack control (< 1.0 mm)	0.010–0.015	
		Damage Control		Yes	Bar fracture	$0.6\varepsilon_{su} \cong 0.06$	
Kowalsky	2000	Serviceability	RC column plastic hinge	Yes	Crack control (< 1.0 mm)	0.015	
		Damage Control		Yes	Bar fracture	0.060	
POLA/POLB	2010/ 2009	50% in 50 yr (72-yr RP)	Solid concrete pile-to-deck plastic hinge	No	Crack control	0.015	
		10% in 50 yr (475-yr RP)		No	Bar fracture	$0.6\varepsilon_{su} \leq 0.06$	
		2/3 of 2% in 50 yrs (2/3 of 2,475-yr RP)		No	?	$0.8\varepsilon_{su} \leq 0.08$	
POLA/POLB	2010/ 2009	50% in 50 yr (72-yr RP)	Hollow concrete pile-to-deck plastic hinge	No	Crack control	0.015	
		10% in 50 yr (475-yr RP)		No	?	$0.4\varepsilon_{su} \leq 0.04$	
		2/3 of 2% in 50 yrs (2/3 of 2, 475-yr RP)		No	Bar fracture	$0.6\varepsilon_{su} \leq 0.06$	
POLA/POLB	2010/ 2009	50% in 50 yr (72-yr RP)	Concrete plug in steel pipe pile-to-deck plastic hinge	No	Crack control	0.015	
		10% in 50 yr (475-yr RP)		No	Bar fracture	$0.6\varepsilon_{su} \leq 0.06$	
		2/3 of 2% in 50 yr (2/3 of 2, 475-yr RP)		No	?	$0.8\varepsilon_{su} \leq 0.08$	
MOTEMS	2011	50% in 50 yr (72-yr RP)	Pile-to-deck plastic hinge	No	Crack control	0.010	
		10% in 50 yr (475-yr RP)		No	Bar fracture	0.050	
MOTEMS	2011	50% in 50 yr (72-yr RP)	RC drilled shaft or prestressed concrete pile In-ground plastic hinge	No	Crack control	0.010	
		10% in 50 yr (475-yr RP)		No	?	0.025	

ε_{su} = ultimate tensile strain of reinforcing steel.

TABLE 27
TENSION STRAIN LIMITS—PRESTRESSING STEEL

Agency	Year	Ground Motion	Component and Location	Explicit Damage State?	Damage State	Strain Limit (in./in.)
AASHTO SGS	2011	7% in 75 yr (1,000-yr RP)	Column/pile plastic hinge	No	Strand fracture	0.030
SCDOT	2008	3% in 75 yr (2,500-yr RP)	Column/pile plastic hinge	No	Strand fracture	0.035
Caltrans	2010	5% in 50 yr (1,000-yr RP)	Column/pile plastic hinge	No	Strand fracture	0.030
POLA/POLB	2010/2009	50% in 50 yr (72-yr RP)	Solid concrete in-ground plastic hinge (depth <10 diameter of pile)	No	Crack control	0.015
		10% in 50 yr (475-yr RP)		No	Strand fracture?	0.025
		2/3 of 2% in 50 yr (2/3 of 2, 475-yr RP)		No	Strand fracture?	0.035
POLA/POLB	2010/2009	50% in 50 yr (72-yr RP)	Solid concrete deep in-ground plastic hinge (depth >10 diameter of pile)	No	Crack control	0.015
		10% in 50 yr (475-yr RP)		No	Strand fracture	0.025
		2/3 of 2% in 50 yr (2/3 of 2, 475-yr RP)		No	?	0.050
POLA/POLB	2010/2009	50% in 50 yr (72-yr RP)	Hollow concrete in-ground plastic hinge (depth <10 diameter of pile)	No	Crack control	0.015
		10% in 50 yr (475-yr RP)		No	Strand fracture	0.025
		2/3 of 2% in 50 yr (2/3 of 2, 475-yr RP)		No	Strand fracture	0.025
POLA/POLB	2010/2009	50% in 50 yr (72-yr RP)	Hollow concrete deep in-ground plastic hinge (depth >10 diameter of pile)	No	Crack control	0.015
		10% in 50 yr (475-yr RP)		No	Strand fracture	0.025
		2/3 of 2% in 50 yr (2/3 of 2, 475-yr RP)		No	?	0.050
MOTEMS	2011	50% in 50 yr (72-yr RP)	Prestressed concrete pile in-ground plastic hinge	No	Crack control	0.005 (incremental)
		10% in 50 yr (475-yr RP)		No	Strand fracture	0.025 (total)

engineers to know what performance is intended (i.e., what damage is prevented) by adherence to the strain limit. If the engineers do not know this information, then it is impossible for the owner/stakeholder to know what the level of increased performance is intended by performance-based design other than the often succinct description provided within the design philosophy of the code or guideline. These descriptions typically discuss damage in terms of "no damage," "minimal damage," "repairable damage," or "significant damage," with no quantification of specific damage levels, such as crack width, depth of spalling, or onset of bar buckling or bar fracture.

Second, the strain limits for specific damage states are generally in agreement between different codes and guidelines. This is likely more a function of the heavy influence of Caltrans, M .J. N Priestley, and others in the development of performance-based guidelines for bridges and marine structures than of separate organizations coming to the same conclusions. Although some variations do exist, the difference likely results from the objectives of the performance criteria. For example, the strain at concrete cover spalling is often set at 0.008 in./in. for deep in-ground plastic hinges, while 0.004 or 0.005 in./in. is used for plastic hinges above or near the surface of the ground (i.e., plastic hinges forming less than 10 pile diameters below ground). Although this may appear to be a discrepancy at a cursory glance, the increased strain at spalling for in-ground plastic hinges is a result of the increased confinement of the cover concrete provided by the surrounding soil. An exception to the congruency between codes and guidelines is the ultimate strain allowed for prestressing strands, where strain limits range from 0.025 to 0.05 in./in. Under monotonic loading, prestressing strands can generally withstand strains up to 0.05 to 0.07 in./in.; however, the effects of low-cycle fatigue and buckling are poorly documented. Furthermore, none of the codes or guidelines provides justification or references for the strain limit adopted. It can only be assumed that the published strain limits represent conservative best estimates to safeguard against strand fracture.

Finally, the strain limits are generally based on conservative rule-of-thumb estimates of the strain at the initiation of damage. For example, strain limits for mild (A706) reinforcing steel typically use 60% to 70% of the ultimate strain under monotonic loading ($0.6\varepsilon_{su}$ to $0.7\varepsilon_{su}$) to establish a reduced ultimate strain under cyclic loading resulting from low-cycle fatigue and buckling. Although the use of such rules of thumb provides a deterministic link between strain and damage, they cannot provide a uniform and consistent level of protection against the onset of damage resulting from load history effects. However, this represents the most justifiable and accurate method currently available to the profession, as the mechanics controlling some damage limit states (such as bar buckling) are complex and not entirely understood.

TABLE 28

COMPRESSION STRAIN LIMITS—CONCRETE

Agency	Year	Ground Motion	Component and Location	Explicit Damage State?	Damage State	Strain Limit (in./in.)
AASHTO SGS	2011	7% in 75 yr (1,000-yr RP)	Column plastic hinge confined concrete	No	Transverse reinforcement fracture/core crushing	$0.004 + 1.4\frac{\rho_v f_{yh} \varepsilon_{su}}{f'_{cc}}$
			Column/pile in-ground plastic hinge confined concrete	No	Transverse reinforcement fracture/core crushing	0.020
SCDOT	2008	3% in 75 yr (1,000-yr RP)	Column plastic hinge confined concrete	No	Transverse reinforcement fracture/core crushing	$0.004 + 1.4\frac{\rho_s (f_{yh}) \varepsilon_{su}^R}{f'_{cc}}$ Note: Omission of f_{yh} in the source may be a typographical error
Caltrans	2010	5% in 50 yr (1,000-yr RP)	Column plastic hinge confined concrete	No	Transverse reinforcement fracture/core crushing	$0.004 + 1.4\frac{\rho_v f_{yh} \varepsilon_{su}}{f'_{cc}}$
Priestley et al.	2007	Serviceability	Column plastic hinge unconfined concrete	Yes	Cover spalling	0.004
		Damage control	Column plastic hinge confined concrete	Yes	Transverse reinforcement fracture/core crushing	$0.004 + 1.4\frac{\rho_v f_{yh} \varepsilon_{su}}{f'_{cc}}$
Kowalsky	2000	Serviceability	Column plastic hinge unconfined concrete	Yes	Cover spalling	0.004
		Damage control	Column plastic hinge confined concrete	Yes	Transverse reinforcement fracture/core crushing	0.018
POLA/POLB	2010/2009	50% in 50 yr (72-yr RP)	Solid concrete pile-to-deck plastic hinge extreme fiber concrete compression strain	No	Cover spalling	0.005 (cover concrete)*
		10% in 50 yr (475-yr RP)		No	Transverse reinforcement fracture/core crushing	$0.005 + 1.1\rho_s \le 0.025$ (core concrete)*
		2/3 of 2% in 50 yr (2/3 of 2, 475-yr RP)		No	NA	No limit
POLA/POLB	2010/2009	50% in 50 yr (72-yr RP)	Solid concrete in-ground plastic hinge Extreme fiber concrete compression strain	No	Cover spalling	0.005 (cover concrete)*
		10% in 50 yr (475-yr RP)		No	Core spalling	$0.005 + 1.1\rho_s \le 0.008$ (core concrete)*
		2/3 of 2% in 50 yr (2/3 of 2, 475-yr RP)		No	Transverse reinforcement fracture/core crushing	$0.005 + 1.1\rho_s \le 0.025$ (core concrete)*
POLA/POLB	2010/2009	50% in 50 yr (72-yr RP)	Solid concrete deep in-ground plastic hinge (depth > 10 diameter of pile) extreme fiber concrete compression strain	No	Cover spalling	0.008 (cover concrete)*
		10% in 50 yr (475-yr RP)		No	?	0.012 (cover concrete)*
		2/3 of 2% in 50yr (2/3 of 2, 475-yr RP)		No	NA	No Limit
POLA/POLB	2010/2009	50% in 50 yr (72-yr RP)	Concrete filled steel pipe pile-to-deck plastic hinge extreme fiber concrete compression strain	No	Concrete spalling	0.010 (cover concrete)*
		10% in 50 yr (475-yr RP)		No	Concrete crushing	0.025 (core concrete)*
		2/3 of 2% in 50 yr (2/3 of 2, 475-yr RP)		No	NA	No Limit

Table 28 continued on p.74

Table 28 continued from p.73

Agency	Year	Ground Motion	Component and Location	Explicit Damage State?	Damage State	Strain Limit (in./in.)
POLA/POLB	2010/2009	50% in 50 yr (72-yr RP)	Hollow concrete pile-to-deck plastic hinge extreme fiber concrete compression strain	No	Cover spalling	0.004 (cover concrete)*
		10% in 50 yr (475-yr RP)		No	Prevent pile implosion?	0.006 (cover concrete)*
		2/3 of 2% in 50yr (2/3 of 2, 475-yr RP)		No	?	0.008 (cover concrete)*
POLA/POLB	2010/2009	50% in 50 yr (72-yr RP)	Hollow concrete in-ground plastic hinge extreme fiber concrete compression strain	No	Cover spalling	0.004 (cover concrete)*
		10% in 50 yr (475-yr RP)		No	Prevent pile implosion?	0.006 (cover concrete)*
		2/3 of 2% in 50 yr (2/3 of 2, 475-yr RP)		No	?	0.008 (cover concrete)*
POLA/POLB	2010/2009	50% in 50 yr (72-yr RP)	Hollow concrete deep in-ground plastic hinge Extreme fiber concrete compression strain	No	Cover spalling	0.004 (cover concrete)*
				No	Prevent pile implosion?	0.006 (cover concrete)*
				No	?	0.008 (cover concrete)*
MOTEMS	2011	50% in 50 yr (72-yr RP)	Pile-to-deck plastic hinge maximum concrete compression strain	No	Cover spalling	0.004 (cover concrete)*
		10% in 50 yr (475-yr RP)		No	Transverse reinforcement fracture/core crushing	$0.004 + 1.4\frac{\rho_v f_{yh} \varepsilon_{su}}{f'_{cc}} \le 0.025$ (core concrete)*
MOTEMS	2011	50% in 50 yr (72-yr RP)	In-ground plastic hinge Maximum concrete compression strain	No	Cover spalling	0.004 (cover concrete)*
		10% in 50 yr (475-yr RP)		No	Transverse reinforcement fracture/core crushing	$0.004 + 1.4\frac{\rho_v f_{yh} \varepsilon_{su}}{f'_{cc}} \le 0.008$ (core concrete)*

* The POLA/POLB Seismic Codes and MOTEMS do not explicitly state whether the strain limits apply to the unconfined cover, or the confined core concrete, but only refer to the "extreme fiber concrete compression strain" or the "maximum concrete compression strain" for POLA/POLB and MOTEMS, respectively. Below the strain limit given, the authors have included a remark clarifying what they believe is the intended "extreme fiber," whether it belongs to the unconfined cover or the confined core concrete. The clarification was based on research reports and commonly used strain limits within bridge practice and their applicable fiber location.

ρ_v = confining transverse steel volumetric ratio.
f_{yh} = nominal yield stress of transverse reinforcing steel.
ε_{su} = ultimate tensile strain of transverse reinforcing steel.
f'_{cc} = confined concrete compressive strength.
ε_{su}^R = reduced ultimate tensile strain of reinforcing steel to account for buckling and low-cycle fatigue.
ρ_s = confining spiral volumetric ratio.
NA = not available.

To put strain limit data into a probabilistic PBSD format will require the use of distribution functions using a central tendency (mean or median value) and a dispersion measure, as has been discussed previously. If this format is implemented, the judgment-based strain limits will give way to more objective data. However, a consensus within the design community must still be achieved for a uniform damage definition.

An example of such consensus may be illustrated by consideration of how damage states and performance levels might be linked. Consider the generic fragility or probability of occurrence curves shown in Figure 20. If performance levels, such as those shown in Table 25, are mapped onto these same fragilities, the break points between performance levels must be positioned relative to the damage state fragilities. A logical and conservative way to do this may be to set the break points such that 90% or 95% of the occurrences of the various damage states lie to the right or above the break point. This approach would then result in a 5% or 10% probability of bar buckling at the collapse performance-level break point, as illustrated in Figure 26. Alternatively, this conservatism might only apply at the life safety/collapse break point, and less conservatism might be chosen for the lower break points.

TABLE 29
COMPRESSION AND TENSION STRAIN LIMITS—STRUCTURAL STEEL

Agency	Year	Ground Motion	Component	Explicit Damage State?	Damage State	Strain Limit
POLA/POLB	2010/2009	50% in 50 yr (72-yr RP)	Steel pipe pile in-ground plastic hinge—extreme strain	No	Local bulging?	0.010
		10% in 50 yr (475-yr RP)		No	Local buckling?	0.025
		2/3 of 2% in 50 yr (2/3 of 2, 475-yr RP)		No	Tearing?	0.035
POLA/POLB	2010/2009	50% in 50 yr (72-yr RP)	Steel pipe pile filled with concrete in-ground plastic hinge—extreme strain	No	Local bulging?	0.010
		10% in 50 yr (475-yr RP)		No	Local buckling?	0.035
		2/3 of 2% in 50 yr (2/3 of 2, 475-yr RP)		No	Pipe tearing?	0.050
POLA/POLB	2010/2009	50% in 50 yr (72-yr RP)	Steel pipe pile deep in-ground plastic hinge—extreme strain (depth > 10 diameter of pile)	No	Local bulging?	0.010
		10% in 50 yr (475-yr RP)		No	Local buckling?	0.035
		2/3 of 2% in 50 yr (2/3 of 2, 475-yr RP)		No	Pipe tearing?	0.050
POLA/POLB	2010/ 2009	50% in 50 yr (72-yr RP)	Steel pipe pile filled with concrete in-ground deep plastic hinge—extreme strain (depth > 10 diameter of pile)	No	Local bulging?	0.010
		10% in 50 yr (475-yr RP)		No	Local buckling?	0.035
		2/3 of 2% in 50 yr (2/3 of 2, 475-yr RP)		No	Pipe tearing?	0.050
MOTEMS	2011	50% in 50 yr (72-yr RP)	Steel pipe pile plastic hinge—extreme strain	No	Local bulging?	0.008
		10% in 50 yr (475-yr RP)		No	Pipe tearing?	0.025
MOTEMS	2011	50% in 50 yr (72-yr RP)	Steel pipe pile filled with concrete plastic hinge—extreme strain	No	Local bulging?	0.008
		10% in 50 yr (475-yr RP)		No	Pipe tearing?	0.030

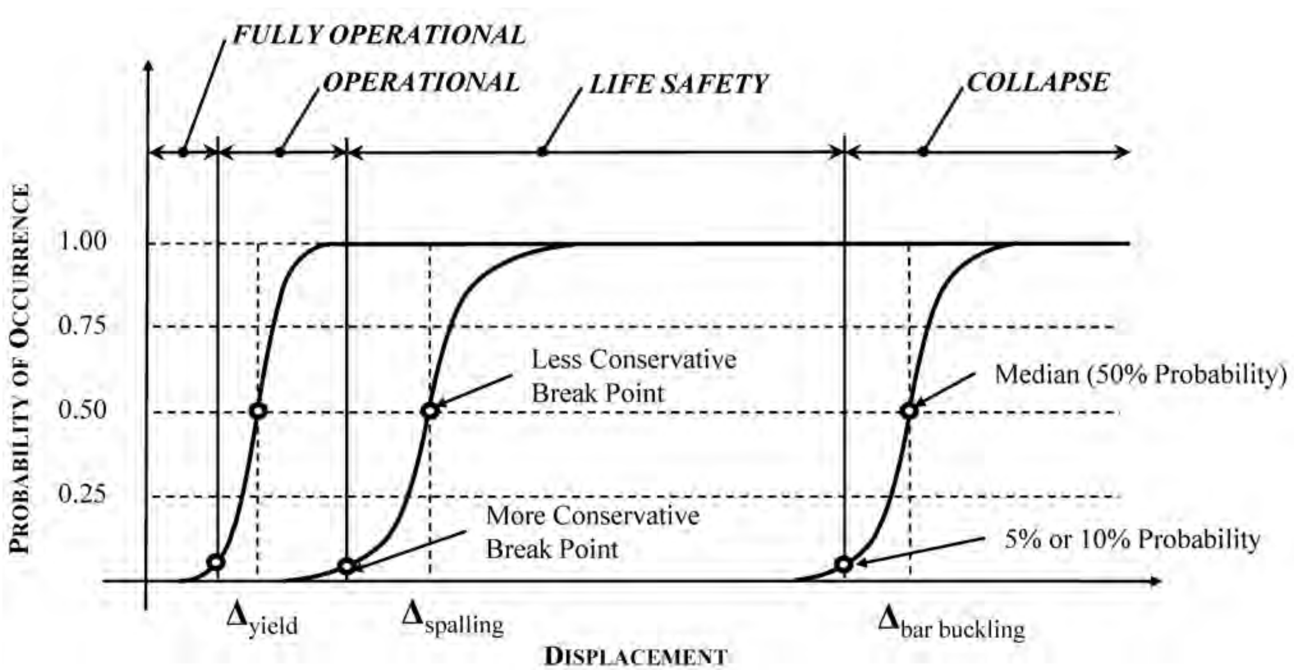


FIGURE 26 Relationship between probability of damage and performance level.

CHAPTER ELEVEN

SUMMARY OF QUESTIONNAIRE RESULTS

OVERVIEW

A questionnaire was sent to all 50 states regarding their seismic design practice. Questions covered conventional and large signature structures for both new design and retrofit practice. The questions were grouped into the following categories: seismic classification, seismic design, seismic research sponsored by the organization, and decision making for seismic design. These categories form the following subsections. There were 27 questions in the survey. The question number is used to present the summary data here. Appendix A is a copy of the questionnaire, and Appendix B is a summary of the responses.

SEISMIC CLASSIFICATION QUESTION

Question 1. Is your state or region a seismic state?

Forty-one states responded to the survey, and most of the respondents were states that include areas of the higher seismic zones or categories. In the two AASHTO seismic design specifications, the higher categories include Seismic Zones 2, 3, or 4, or Seismic Design Categories B, C, or D for the force-based and displacement-based specifications, respectively. There are 34 states that include these three higher categories, known here as "seismic states," with all but three seismic states represented in the survey. Figure 27 shows the states that fall under the seismic state designation for the 48 contiguous states. Because the categories are functions of both seismic hazard and site classification, the total number of states in the higher categories corresponds to those states that fall into the higher categories when Site Class E soils are present. These are poorest soils that are standardized in the AASHTO specifications. There may be cases where sites requiring special studies (Site Class F) could place other states beyond the basic 34 states into the higher categories.

It is of note that 27 states (about 66% of responding states) recognized that they are seismic states, which means that they have some territory that falls into the upper three design categories. Some states still may not fully recognize that the categories are now dependent on site classification, and therefore, some poor soil sites (Site Class E, for example) could put the state nominally in the higher categories. Many of these states will fall into SDC A only when Site Class A, B, C, or D soils are present.

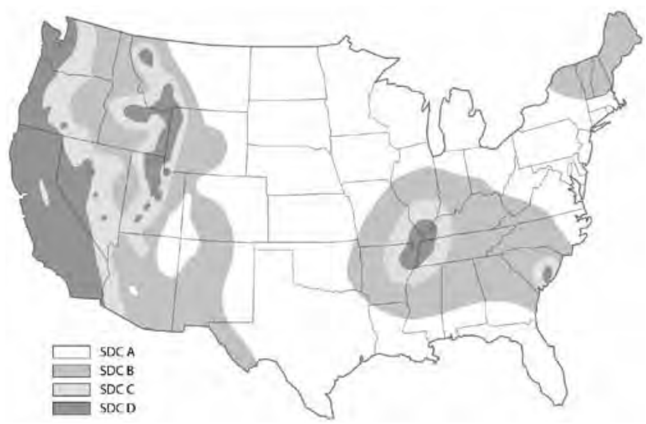


FIGURE 27 Seismic Design Categories (SDCs) for Site Class E.

SEISMIC DESIGN QUESTIONS

Question 2. What seismic design provisions does your state or agency use?

Approximately half of the 27 states that acknowledge being seismic states use the force-based procedure of the LRFD specification, whereas roughly the other half use the displacement-based procedure of the SGS or some other displacement-based criteria. As outlined earlier in the report, two states, California and South Carolina, use their own custom-developed seismic design specifications. Two states responded that they permit both the LRFD and the SGS methods to be used, while another state responded that it prefers the SGS displacement-based method, but the LRFD method better addresses critical and important bridges. A single state still uses the seismic provisions from the 17th edition of the AASHTO Standard Specifications.

Question 3. If you are using the AASHTO LRFD force-based procedure currently, do you have any plans to change over to the AASHTO Seismic Guide Specification displacement-based procedure in the near future?

Only three states that are still using the force-based procedure have any plans to change over to the newer specifications (i.e., the displacement-based provisions).

Interestingly, nine states or 35% of the respondents indicated that they have no plans to change over to the SGS displacement-based methodologies. This response might be expected primarily of states that are generally in the lower seismic categories. However, five of these states actually have the potential, depending on site classification for bridges, to be in the highest seismic categories, meaning Seismic Zone 4 or Seismic Design Category D.

Question 4. If you are using "Other" seismic design provisions currently, do you have any plans to change over to the AASHTO Seismic Guide Specification displacement-based procedure in the near future?

Of the two states that use custom-developed seismic criteria, neither plans to change over to the AASHTO SGS provisions in the near future. Included beyond the fully customized specifications are a number of states that modify their seismic criteria into more rigorous requirements, such as Oregon, and that also have no plans to modify their requirements down to the AASHTO minimum levels.

Question 5. Does your state or agency include additions or modifications to the AASHTO seismic provisions in your bridge design manual?

Eleven of 26 states that responded to this question (42%) make additions or modifications in their bridge design manuals to the seismic requirements that AASHTO provides. Modifications include the following:

- Two-level seismic design requirements
- Cold weather effects as they affect seismic design
- Additional requirements for single-span bridges
- Modifications to the seat length provisions and R-factors
- More rigorous requirements for high-population areas of the state
- Minimum ground acceleration and spectral acceleration levels, and so on.

Question 6. Does your state or agency use either single- or multi-level, performance-based seismic design for new bridges, other than the "no collapse" performance level in AASHTO LRFD and the Seismic Guide Specifications? If yes, how were these used?

Of 27 respondents to this question, 10 (37%) answered that they used requirements other than the no collapse requirements in AASHTO. Of course, how these additional requirements are used is important in assessing the significance of this response. Figure 28 illustrates the distribution of the types of projects that use criteria other than no collapse. Sixteen respondents contributed answers to the "check all that apply if yes" portion of the question, and eight of these cited project-specific criteria.

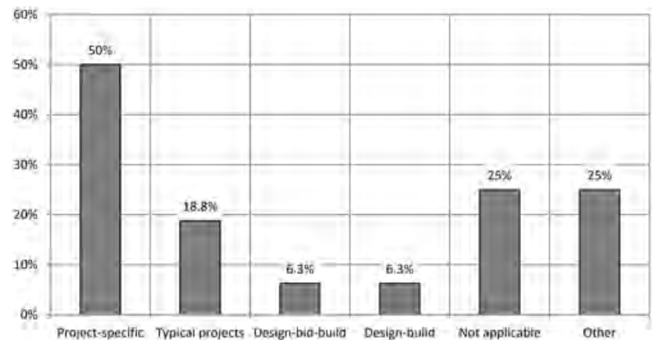


FIGURE 28 Types of projects on which criteria other than "no collapse" were used.

Question 7. If you use an alternate or more specific definition of Performance-Based Design than the working definition provided in this survey, please summarize your definition below.

The survey expressed the working definition of PBSD as "criteria and methodology that links post-earthquake operation or specific other behavioral outcome (e.g., damage level) to typical engineering design parameters (strain levels, displacements, forces, etc.)."

Respondents summarized alternative definitions, which included the following:

- Operational; minimal damage
- No collapse in large earthquakes and serviceability in smaller, more frequent earthquakes.

These specific definitions could be interpreted as subsets of the more general survey definition.

Question 8. If your state or agency uses multi-level seismic performance criteria (multiple return periods with different performance objectives), please describe the reason(s) for doing so.

There were a number of responses to this question, but the overarching theme was that such multilevel criteria were primarily used for major or critical bridges. However, several states responded that they apply multilevel seismic criteria to ordinary bridges, and in at least one case the location within the state, in addition to importance, determined whether multi- or single-level criteria were to be used. In this case, the more populous area of the state had the higher criteria requirements.

Question 9. If performance-based design criteria have been used, would you be willing to share the criteria with the NCHRP 43-07 project?

About 52% of the 27 respondents to this question were willing to share information with this project. Many provided either sample criteria or links to such criteria. In the cases where there was reluctance to share data, it is presumed that the time and effort to compile and send data was the reason for not participating.

Question 10. Have you ever used performance-enhancing measures, such as isolation or damping systems?

Again, 27 states responded to this question, and of those states 56%, or 15 states, have used performance-enhancing measures.

Question 11. If yes, what was the desired performance level?

Figure 29 provides the responses of the states that had used performance-enhancing measures. Fourteen states responded, the distribution of response is well beyond the no collapse or life safety objective. Most states were looking to achieve at least repairable damage, although no attempt was made to quantify "repairable" in the survey. Five states responded that they had used such measures or devices to achieve operational performance, either full operations or operations for emergency vehicles only.

Question 12. Does your state or agency have a seismic retrofit program?

Of 26 responses to this question, nine states or 35% of respondents have a seismic retrofit program. When queried whether seismic improvements are made outside of a formal retrofit program, for instance when other bridge improvements are made, many states responded that they do implement seismic retrofits at that time. A common improvement is to replace or improve bearings when maintenance projects are completed. One state indicated that its seismic program has been phased out, but that seismic improvements are still recommended when other substructure work is required.

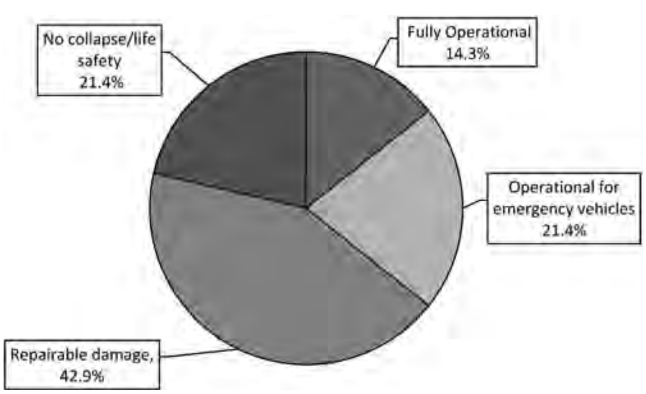


FIGURE 29 Performance objective, given that performance-enhancing measures were used.

Question 13. If you have a seismic retrofit program, do you use performance-based criteria beyond new bridge provisions intended to meet the "No Collapse" damage state to design retrofits?

Of the states that have a retrofit program, about half use the same performance requirements that are used for new design (e.g., no collapse in a single-level rare earthquake—1,000-year return period). The other half tend to use FHWA's *Seismic Retrofitting Manual for Highway Structures: Part 1, Bridges* (FHWA 2006). However, several states use a higher return period for the lower-level or serviceability check, and this is taken at a 500-year level rather than a 100-year level for two states. Additionally, retrofits of major or critical structures often have project-specific criteria that are different and more rigorous than the FHWA basic requirements.

SPONSORED SEISMIC RESEARCH QUESTIONS

Question 14. Does your state or agency sponsor seismic design related research?

Two-thirds of the states responding sponsor seismic design research. Most of the states provided links to websites where such research could be found. These links can be found in Appendix A, which includes the full listing of the survey questions and responses. Sample projects include the following:

- Development of pile-bent seismic response data (South Carolina)
- ABC for high seismic areas
- State-specific seismic hazard mapping (Arizona) or state-specific site effects (Tennessee)
- Design guidelines for highly populated areas of New York state
- Risk analysis for regional transportation networks within a state (Oregon)
- Instrumentation of major bridges in the New Madrid seismic zone.

Some of this research may be funded through pooled studies, such as the work being done in the Pacific Northwest on decision making for ABC. Thus, not all work is being sponsored solely by one organization, and FHWA also contributes to this work.

Question 15. What is the purpose of the research (check all that apply format)?

Roughly two-thirds of the research is done to develop design criteria that are not adequately covered by the AASHTO specifications (see Figure 30). Such criteria often include types of structures that are not covered in depth in the AASHTO seismic specifications, such as pile bents, or the emerging area of ABC for use in seismic regions. Almost equal in response numbers is work to improve the seismic performance of standard bridges (61.1% of responses). By contrast, less than 30%

of the research in the seismic arena is being done to develop performance-based design criteria and methods.

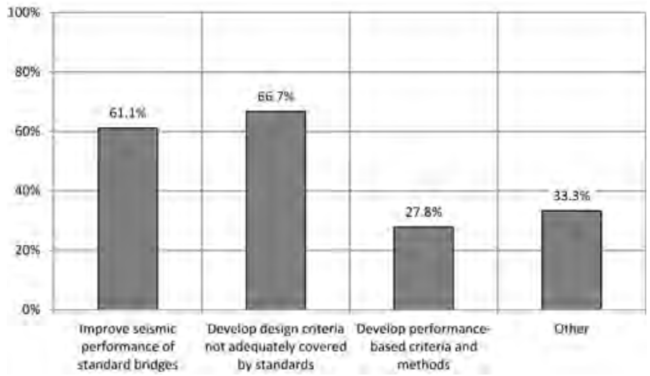


FIGURE 30 Purpose of seismic-related research.

Question 16. Has your state or agency sponsored research correlating damage to engineering design parameters (strain levels, displacements, forces, etc.)?

Only three of 25 states (12%) responded in the affirmative. An implication of this is that many states may be relying on others to develop data for use in the PBSD area, and this may reflect a lack of data on types of bridges that may be unique to certain areas of the country.

Question 17. What are the most important areas for which research is needed to deploy Performance-Based Seismic Design?

Table 30 lists the most important areas that states believe need to be addressed before PBSD can be deployed on a routine basis. States were asked to rank a preset list of areas, and the table provides the results of that ranking. Areas that are not listed in the table may also be important, but are not represented.

TABLE 30
MOST IMPORTANT TOPIC AREAS THAT NEED TO BE ADDRESSED, LISTED IN RANK ORDER

Areas that need to addressed	Total Score[1]	Overall Rank
Correlation between performance level and damage states	140	1
Structural displacement limits for operability	127	2
Strain or rotation limits for given damage states	105	3
Improved structural analysis techniques	93	4
Construction cost data for higher performance levels	90	5
Nonstructural displacement limits for operability	88	6
Probabilistic data for damage states	75	7
Total Respondents: 26		

[1] Score is a weighted calculation. Items ranked first are valued higher than the following rankings; the score is the sum of all weighted rank counts.

Correlation between performance level and damages states is clearly the most important area, whereas probabilistic data comes in last. Both areas must be addressed before full PBSD can be achieved, and the disparity in rank likely bespeaks the preference of designers to use deterministic data rather than probabilistic data. For example, it is likely that designers prefer a single level of strain limit for a given damage state, rather than a fragility curve. Although the latter is more complete, the single level is more understandable at the level of most practicing engineers.

Between the top and lowest rank are items that also must be developed, such as displacement limits for operability, which is not entirely independent from the top-ranking item. The close order of these two items shows consistency in the responses. Improved limits, construction data, and nonstructural effects are also important.

QUESTIONS RELATED TO DECISION MAKING FOR SEISMIC DESIGN

Question 18. Does your state or agency have a policy for establishing bridge operational classification (for example: Critical, Essential, or Other bridges as defined in AASHTO LFRD Bridge Design Specifications)?

The responses to this question were split evenly between yes and no. As the profession moves forward with PBSD, this answer must shift to 100% yes. This is an area that perhaps could use some research effort.

Question 19. Does your state or agency use criteria for the design of Critical or Essential bridges beyond that given in the AASHTO LRFD Bridge Design Specifications?

Again, there is a definite split between positive and negative responses: 27% yes, 65% no, and 8% varies on a case-by-case basis. The negative responses to this question were almost all contributed by the same responders who indicated no for the previous question.

The case-by-case responses provided some insight into the types of additional criteria in use:

- Limits on ductility and strains, which in a way correspond directly to the approach in the SGS (thus, the question may not have been as clearly worded as it could have been.)
- Operational performance
- Repairable (essential) and operational (critical) performance
- Essentially elastic and repairable damage targets for large projects
- Multilevel seismic performance (all two-level hazard with corresponding performance)

Question 20. If you implemented Performance-Based Design, would you prefer if the procedures were optional or mandatory?

This question sought to determine the users' preference for PBSD as an optional extension of the current specification approach or as a mandatory element. As indicated in Figure 31, 65% prefer to keep PBSD as an optional methodology. This could either be as an extension of the existing methodologies or a replacement approach. Only three of 26 (12% of respondents) preferred to have PBSD be mandatory. The results for this question represent an important preference for the bridge design community, that optional PBSD is the way forward.

Question 21. If you implemented Performance Based Design, would you be more likely to apply it to major bridges or conventional bridges?

The most common application for PBSD is for major, critical, or essential bridges, as indicated in Table 31, where 54% prefer to apply this design approach only to larger, more important structures. This parallels actual practice for major projects where project-specific seismic criteria have been used. For such projects, some form of PBSD is typically used, as was shown in the "Project-Specific Criteria" summary in chapter nine.

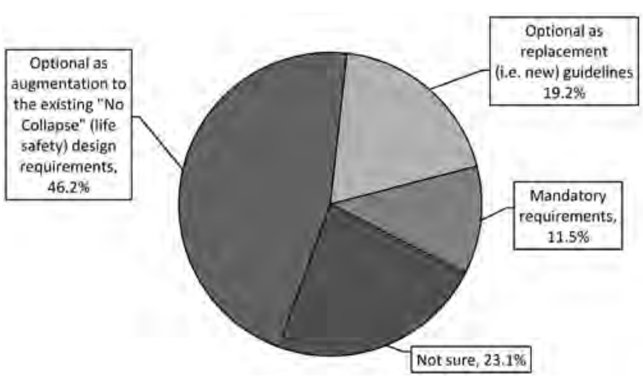


FIGURE 31 Preference for manner of potential PBSD implementation.

TABLE 31
PREFERENCE FOR PBSD BY BRIDGE OPERATIONAL CLASSIFICATION

Value	Count	Percentage
Major/Critical/Essential Bridges	13	54.2
Both	7	29.2
Not Sure	4	16.7
Conventional Bridges	0	0
Total Respondents: 24		
Skipped: 2		

Seven of the 24 responders also indicated that they would use PBSD for conventional bridges and major bridges. Several DOTs already are using this as policy, particularly for corridors where there is heavy traffic, high population, and high risk to the economy.

Question 22. Would you consider Performance Based Design of conventional bridges if design or construction costs increased, but bridge performance was significantly improved? Check the box corresponding to the cost increase range you would be willing to bear for each performance level listed on the left.

The purpose of this question was to determine the tolerance for cost increases relative to potential benefits of improved performance for conventional bridges. Obviously, obtaining increased performance at little or no cost increase is a highly desirable objective. Such is the case in the aggregate of the responses. Table 32 shows that most respondents would be willing to spend up to 10% more for their conventional bridges to obtain increased seismic performance. However, one can see the reluctance to spend money for improved seismic performance when the costs reached the 25% and over range. Also of note are the percentages of respondents who would not consider PBSD for conventional bridges, even if fully operational performance could be bought with only a 10% increase in cost. This is curious,

TABLE 32
RESPONDENTS' TOLERANCE FOR COST INCREASE RELATIVE TO PERFORMANCE FOR CONVENTIONAL BRIDGES

Performance Level	Cost Increases				Would Not Consider PSBD for This Category	Total Responses
	0–10%	10–25%	25–50%	>50%		
Immediate Access for Emergency Vehicles in Frequent Earthquake	16 (61.5%)	5 (19.2%)	0 (0.0%)	1 (3.8%)	4 (15.4%)	26 (100%)
Immediate Access for All Vehicles in Frequent Earthquake	15 (57.7%)	4 (15.4%)	1 (3.8%)	0 (0.0%)	6 (23.1%)	26
Immediate Access for Emergency Vehicles in Rare Earthquake	10 (38.5%)	7 (26.9%)	1 (3.8%)	0 (0.0%)	8 (30.8%)	26
Immediate Access for All Vehicles in Rare Earthquake	9 (34.6%)	3 (11.5%)	0 (0.0%)	0 (0.0%)	14 (53.8%)	26

TABLE 33
TOLERANCE FOR COST INCREASE RELATIVE TO PERFORMANCE FOR MAJOR/CRITICAL/ESSENTIAL BRIDGES

Performance Level	Cost Increases				Would Not Consider PBSD for This Category	Total Responses
	0–10%	10–25%	25–50%	>50%		
Immediate Access for Emergency Vehicles in Frequent Earthquake	9 (34.6%)	7 (26.9%)	4 (15.4%)	3 (11.5%)	3 (11.5%)	26 (100%)
Immediate Access for All Vehicles in Frequent Earthquake	11 (42.3%)	7 (26.9%)	4 (15.4%)	1 (3.8%)	3 (11.5%)	26
Immediate Access for Emergency Vehicles in Rare Earthquake	7 (26.9%)	10 (38.5%)	4 (15.4%)	2 (7.7%)	3 (11.5%)	26
Immediate Access for All Vehicles in Rare Earthquake	8 (30.8%)	6 (23.1%)	2 (7.7%)	3 (11.5%)	7 (26.9%)	26

because it would appear a bargain to achieve operational performance at less than a 10% cost increase.

Question 23. Would you consider Performance Based Design of major/critical/essential bridges if design or construction costs increased, but bridge performance was significantly improved? Check the box corresponding to the cost increase range you would be willing to bear for each performance level listed on the left.

The purpose of this question was to determine the tolerance for cost increases relative to potential benefits of improved performance for major/critical/essential bridges, which would also typically be the signature or important structures to a region. For these bridges, there is a much higher tolerance of cost increase to achieve improved performance. This can be seen in the broader spread of responses over the higher cost ranges in Table 33. Additionally, fewer respondents indicated that they would not consider PBSD for such bridges, where the numbers dropped by about 50% relative to the same answer for conventional bridges in Table 32. This is consistent with practice, where many major, critical, or essential bridges have been designed using project-specific criteria in which some type of performance-enhancing criteria has been used. Increasingly, such criteria are based on performance-based design principles.

Question 24. Please rank the following impediments to the use of Performance Based Design.

Respondents were asked to rank impediments to the use of PBSD from a list of seven items. This list (Table 34) may not be complete, but major impediments are included. The most severe impediment, by its ranking at the top of the list is the lack of proven methodologies for PBSD and the lack of appropriate design standards. However, not far back in the ranking are the lack of decision-making tools and the potential for increased construction costs of bridges designed using PBSD. The respondents generally believed that they had sufficiently trained staff and could control the work, or at least these issues fell further down the list. The perception of legal issues with applying PBSD was not one of the higher concerns.

TABLE 34
RANKING OF COMMON IMPEDIMENTS TO THE IMPLEMENTATION OF PERFORMANCE-BASED SEISMIC DESIGN

Item	Total Score[1]	Overall Rank
Lack of proven methodologies and design standards	145	1
Lack of decision-making tools for establishing performance levels	132	2
Increased construction costs	122	3
Increased design costs	95	4
Lack of staff with adequate technical skills and expertise	87	5
Legal risks of not meeting targeted performance goals	76	6
Difficulty in quality control, in-house or contracted work	52	7
Total respondents: 26		

[1] Score is a weighted calculation. Items ranked first are valued higher than the following rankings; the score is the sum of all weighted rank counts.

Question 25. Do you have the necessary tools/data to frame PBSD or operational criteria decisions for upper management, policy makers, and/or the public?

Regarding the necessary tools or data to frame meaningful decisions for upper management or policy makers within an organization, 81% said they either did not have the tools or were not sure. Only 19% of respondents indicated that they had sufficient tools and data to help decision makers.

The respondents were also asked to check entries from a list of information types that would be needed. This portion of the question was in a check-all-that-apply format, and the results are shown in Figure 32. The most checked

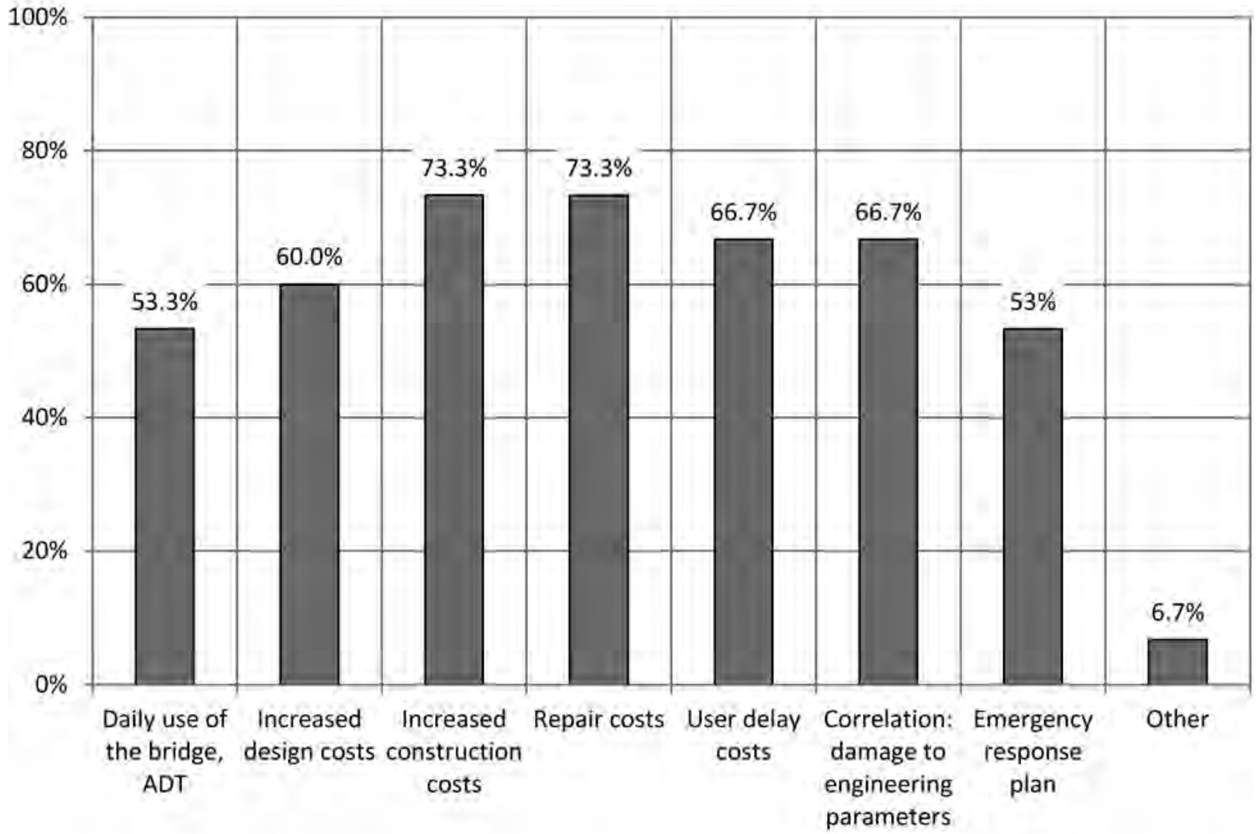


FIGURE 32 Information needed to assist decision makers in deciding whether to allocate resources to improve earthquake performance.

data types were construction costs and repair costs associated with the potential seismic performance improvements. User delay costs and correlation between EDPs and damage states were also high on the list, followed closely by design costs. With all responses being contributed by 50% to 75% of respondents, these are clearly areas where information is to be developed to assist the decision-making process.

Question 26. Is adequate information available for your agency to formulate a statement for the public explaining what to expect in terms of availability and recovery of bridges following a design earthquake?

Regarding adequate information and data to formulate a meaningful statement to the public about what to expect in terms of seismic response, most (62%) indicated that they had enough information. This may reflect that since the 1989 Loma Prieta and 1994 Northridge earthquakes, there has been some effort to make sure stakeholders understand that the design methodologies are intended to permit some damage and that bridges may not be immediately available following a seismic event. This is an area where the tools that are being developed for PBSD will ultimately help designers articulate what the public should expect following a large earthquake. Currently, as one respondent pointed out, the engineering community may understand what to expect, but the public may not fully understand that bridges are expected to suffer damage, even if those bridges have been seismically retrofitted.

Respondents who indicated that they did not have enough information cited the following issues.

- More studies need to be done to assist in this area.
- Some states have not prepared statements of what to expect, especially the lower seismic states.
- Screening for vulnerabilities needs to be done.
- Some departments are still trying to catch up to other requirements, such as load ratings.
- What to expect in terms of recovery is not well understood.
- There is such a short history with the current design methods that the actual performance in earthquakes remains to be understood, particularly in those areas of the country where earthquakes occur infrequently.

Data development and dissemination to help the public understand what to expect probably are areas where continual efforts need to be made, in addition to keeping engineers and owners up-to-date with new design developments.

SUMMARY QUESTION

Question 27. Do you have any comments you would like us to consider?

This final query was intended to let the respondents add comments that may not have been addressed in the other questions. There were eight additional comments, and they paraphrased here.

- PBSD needs to be pursued, especially for critical and essential bridges.
- Where only the lower two seismic zones are present seismic design is not critical, seismic issues have a minimal effect on bridges.
- Until FHWA comes out with design examples showing how to use the displacement-based method and PBSD, the LRFD force-based method will continue to be used. A series of examples, such as the 1996 FHWA series, would be helpful. (Fortunately, several similar examples have recently been developed for the displacement-based approach, and these are being distributed with NHI Course 130093, which covers seismic design by both methods.)
- Additional links to state-specific design procedures in lower-seismic, but high-population states were provided.
- In some states and areas, higher performance may need to be considered when alternative routes are not readily available should bridges be damaged by earthquakes.
- Another respondent indicated that in high-population centers, even though seismic hazard is low, the risk to the economy if bridges are damaged could be high. Therefore, higher performance objectives need to be considered.
- Design for frequent earthquakes continues to be a concern for some and an area for which design criteria do not appear to be available for new bridge design.

Overall, the final comments suggest that continual development of earthquake design criteria is needed in the areas of performance, decision making, low seismic hazard requirements, and policy for assigning critical and essential bridges. These comments tend to suggest that PBSD is worthwhile and necessary to pursue.

SURVEY SUMMARY

The questionnaire indicates that there is support for a two-level seismic hazard in a PBSD format (guide specification or other), provided the application of the design guidelines is optional. The responses show that states are using PBSD when it makes sense to them, which typically is on large and important projects. There also is tolerance for limited project cost increases when those costs can provide improved seismic performance. However, the states mostly indicated that there is a need for development of PBSD specifications or guidelines and training on the use of the methodology. Additionally, methods and data to help decision makers choose performance objectives are sorely needed. Such methodology and data should be widely available for all public agencies responsible for bridge design, maintenance, and improvements.

CHAPTER TWELVE

IDENTIFICATION OF KNOWLEDGE GAPS

There does not appear to be a single entity that is driving the industry toward PBSD. Instead, there is a perception that the bridge industry could better predict likely performance in large, damaging earthquakes than it is doing at the present. This perception is driven from within the engineering and research communities as the state of knowledge advances. However, there are gaps in that knowledge base that need to be closed. There is also a perceived need from outside the engineering community, because the ability to clearly describe to the public what performance to expect in earthquakes is less developed than public decision makers would prefer.

Knowledge gaps certainly exist in all facets of PBSD; however, key knowledge gaps that need to be closed in order to implement PBSD are covered here. Gaps related to seismic hazard prediction include the following:

- Currently, probabilistic ground motion data are related on a uniform-hazard basis. This basis also includes mean response. For PBSD to work, the entire chain of seismic design calculation must be formulated in terms of a central tendency—median or mean—and a measure of dispersion about that central tendency—coefficient of variation, standard deviation, and so on. Probabilistic density functions or distributions of failure rates must be quantified. This has been done already with the ground motion data; thus, there are fewer knowledge gaps with seismic hazard than from the other portions of the PBSD process. The science and art of seismic hazard characterization continue to evolve, and it is reasonable to expect that they will continue to do so.
- The potential impact of developing risk-adjusted spectral accelerations for the 1,000-year design earthquake is not known. The use of risk-adjusted accelerations could reduce the design accelerations in some parts of the country, based on the experience of the building industry.

Gaps related to structural analysis include the following:

- Improved nonlinear static analysis procedures that provide median response. Additionally, means for establishing the dispersion of structural response around the mean or median (β factors) will be required for implementation of PBSD. Guidance is needed for each analysis technique or methodology, including the coefficient and the capacity spectrum methods, to ensure consistent application of probabilistic data.
- Modeling guidelines for nonlinear analysis specific to the analytical technique used and the objectives selected, in terms of detail and refinement. Such guidelines are to focus on areas where simplification can be employed without loss of accuracy and on the uncertainty that is introduced with simplification.

Gaps related to damage prediction include the following:

- A national consensus document relating damage levels and performance descriptors. Currently, there is inconsistency in the use of terms and the use of different terms. A more uniform application of PBSD could eventually be implemented if researchers report consistent data; state DOTs use consistent terms for performance, damage, and repair; and the design community uses consistent terms.
- Lack of sufficient fragility relationships and the associated uncertainty regarding damage levels (e.g., strain, curvature, rotation, displacement). Some fragility information has been developed, and these relationships may provide approximate data until more detailed information becomes available.
- Lack of fragility relationships for different types of structural systems, particularly substructure systems where energy dissipation is expected. In some areas, sufficient data have already been developed, and they simply need to be put into a usable form. Some of the PEER databases already have addressed this for some elements, for instance circular and rectangular columns, although relevant data may always be added as they become available. For other substructure types, insufficient data exist to formulate reliable fragilities.
- The influence of loading history. The sequence of loading cycles may affect the available deformation capacity (and ductility capacity) owing to concentration of strains in reinforced concrete members.
- The quantification of seismic performance of innovative or novel designs. Useful items currently missing include standardized formats for proof testing, specifications, and design procedures or methodology.
- Clearly written descriptions of damage types. These could be limited to EDPs so that designers may clearly understand the damage limit states that are

being considered. In general, it would be useful if design codes could provide detailed commentaries describing the physical damage states that each prescriptive requirement or performance methodology is intended to control. Currently, the further the design specification migrates from the originator of the knowledge—researchers, field investigators, reconnaissance teams—the more likely it is that knowledge of the failure state is lost. This must be corrected for PBSD to be successful. If the design engineer cannot identify and confidently suppress potential failure modes, the design is likely not to be successful.

Performance-related gaps include the following:

- Performance information related to permanent deformations of a bridge. This includes deformations of columns and other substructure units and deformations, offsets, slopes and misalignment of the roadway, barriers, approach slabs, and other features that are important to the driving public, emergency responders, and continued use of bridges.
- Performance related to clearly defined damage states that are unambiguous regardless of location within the country or type of structure used.
- Expected performance and damage states in smaller earthquakes. Sufficient data may not be available today to describe the expected damage in smaller earthquakes; so the only alternative is to analyze the structure for the smaller events. It is expected that this will continue to be the case. However, development of clear definitions of the performance that would be preferred in smaller earthquakes would be useful. For instance, is it acceptable to be above first yield, but below full yield of the section, or below a small displacement ductility factor, say 1.5?

Gaps related to loss prediction are the following:

- Cost data are inconsistent and generally lacking for performance-based design, although many agencies are willing to pay somewhat more to achieve improved seismic performance. Some cost data exist, but are perhaps not generally available. For example, Caltrans potentially has relevant repair and replacement cost data from the Loma Prieta and Northridge earthquakes, which could be used to support this effort.
- There is a need for objectively developed data, which are calibrated against cost models, for quantifying the risks of losing a structure by collapse or of losing some or all levels of service for the structure.
- Methodologies for involving the public or other shareholders are either not available or are not sufficiently developed. Again limited data exist, but may not be generally available.

Gaps related to regulatory oversight and training are the following:

- Current PBSD guidelines, for example the FHWA *Retrofitting Manual* and the ICC Performance Code, require user- or project-specific decisions to link performance with EDP) and DMs, and such guidelines may benefit from additional prescriptive provisions or additional guidance to augment the basic performance data.
- Many engineers are not prepared, either by practice or by their educational training, to deal with inelastic response as a result of earthquakes. This gap is gradually being closed as younger engineers are trained in inelastic behavior, primarily in graduate school, and as younger faculty, who are similarly trained, enter the workforce. Their students then enhance public agencies' and private practices' skill sets in this area.

Gaps related to decision makers are the following:

- Development of clear and agreed-upon definitions of critical, essential, or any other designation of important bridges. Currently, little is stated in the AASHTO LRFD, and the definitions that are included may need clarification. Decision makers need a national consensus approach or guideline for defining the importance of bridges and then for defining the appropriate seismic design criteria for such bridges.
- The integration or lack thereof of seismic effects with other hazards is not considered in the AASTHO documents. This falls under the category of multihazard performance. Based on the observations of the direction in which PBSD is likely to move, where calculation of the risk of loss of a bridge or loss of use of a bridge is a likely goal, there may be an opportunity to integrate seismic effects with other hazards. This might be accomplished on the basis of uniform risk (i.e., some percentage risk of loss) for combined hazards, rather than on the basis of the occurrence of the hazard itself (i.e., chance of an earthquake or flood occurring). Analytical procedures for this combined risk approach would need to be developed.
- Guidance regarding the development of consistent criteria for nonseismic bridge improvement projects needs to be developed. An example is widening of bridges where the existing bridge may be older and have seismic deficiencies, and the widening may be used to improve the existing bridge. However, funding requirements often restrict the scope of improvements for such cases. Therefore, a logical framework for deciding when, if, and how seismic issues will be addressed on such nonseismic projects needs to be developed to assist decision makers. Such decisions could be made on the basis of bridge performance, thus drawing on principles of PBSD.

CHAPTER THIRTEEN

CONCLUSIONS AND SUGGESTED RESEARCH

PERFORMANCE-BASED SEISMIC DESIGN IMPLEMENTATION

Performance-based seismic design (PBSD) comprises four primary activities or steps: hazard analysis, structural analysis, damage analysis, and loss analysis. The literature survey and state-of-the-practice assessment indicate that the earthquake engineering community is most highly skilled and its practices most developed in the seismic hazard and structural analysis areas. As one moves further along the PBSD activity list, engineers are less capable in terms of their ability to predict damage, and then even less capable in terms of calculating loss or quantifying risk of loss of service. Therefore, efforts to develop effective tools in these areas would help the engineering community deliver true PBSD.

The seismic hazard calculation procedures in use today routinely use probabilistic methods, although deterministic limits are often used to limit the maximum predicted ground motions. However, our structural analysis methodologies are, by contrast, mainly rooted and applied using deterministic methods. The usual approach is to start with a probabilistic expression of seismic input (e.g., spectral acceleration that has a 7% chance of being exceeded in 75 years), and use this input in a single analysis to determine whether permissible forces or displacements (actually strains) have been exceeded. The resulting judgment then is binary—is the demand strain less than the capacity strain? This method is simple. It is not fully probabilistically based, and should not be misconstrued as a precise prediction. However, such methods for design will likely remain the tool of choice in the near future. Thus, adaptation of PBSD features into this format is likely the logical near-term first step.

Given the uncertainties in the seismic input, properties of the structure, quality of construction, and accuracy of analysis, to name a few, a probabilistic approach may ultimately be preferred, and such a direction would need to be taken to complete all four PBSD steps and have a probabilistically based risk of loss (e.g., ASCE 7-10's 1% chance of structural collapse in 50 years). Thus, strategically the profession may move toward this more complete PBSD approach, but this will take time.

An observation based on the numerous knowledge gaps to be closed in developing full PBSD is that a multiphased approach might be the most likely way to implement PBSD, assuming this is a goal of the bridge industry. The preferences cited in the survey performed for this synthesis suggest that implementation of PBSD follow a nonmandatory path, where the method is an accepted alternative or addition to the current design approaches, and may be restricted primarily to use with more important structures. This is the trend in applications of PBSD to date.

A strategic development panel or steering group may help provide consistency in making progress toward all the goals—some of which are applied research, some of which are policy and philosophically based. This panel might be a facilitated subgroup of the AASHTO T-3 committee or a panel formed within TRB/NCHRP. Such a panel would probably be more effective if it is not as large as the TRB AFF50 seismic committee, so a subgroup of that committee might be an appropriate avenue. The panel could keep track of efforts by individual states, research centers, and related industries, such as the building industry.

One of the first activities of such a panel might be to coordinate and oversee a road-mapping session. There are parallels to this approach in the buildings industry Such a session might consist of a range of bridge industry stakeholders who would participate in a 1- or 2-day facilitated workshop. This session might be coordinated by a strategic development panel, as outlined previously. The goal would be to produce a document outlining different approaches to PBSD implementation and potential impediments that might require additional research and development. Prioritized and coordinated research needs statements might also be generated from this session or by the panel following the session.

The phases of implementation based on the results of this synthesis might comprise both near- and long-term goals, and suggestions for research to support implementation and close the knowledge gaps described earlier are provided here:

Near-term research (next 5 years)—the work products might include additional sections or appendices to the AASHTO *Guide Specifications for LRFD Seismic Bridge Design*.

1. Develop prescriptive deformation limits (e.g., strain) that reflect clearly identifiable damage states that can

be linked to user- or owner-selected performance levels and then matched with user-selected seismic hazard levels. Envisioned are multiple performance-based deformation limit states, such as no collapse, minimal damage, and no damage. The method would build on the databases and damage catalogs reviewed herein and would also leverage department of transportation (DOT) experience from actual damaging earthquakes. This research would produce an alternative design method for use on major/critical/essential structures.

2. Develop a national consensus document describing deformation limits, damage states, and performance objectives and levels. The beginning point would be the damage catalog data, current database information, and compiled project- and agency-specific criteria, similar to that used for Item 1. This effort would help produce consistent use of the PBSD concept and lead to more uniform application of the methods. Or, if it is too difficult to develop a single document that everyone agrees to use, then develop protocols for reporting laboratory and field test data and for using analytical data for design. For example, it would be useful if everyone related data on a pushover curve in the same fashion using the same performance terminology.

3. Develop guidance for use of the prescriptive deformation limits, similar to those in Item 1, which could be matched with multilevel seismic hazard levels. This could be used, for example, to assess bridges for loading at a lower-level event and at the current 1,000-year event, or at other user-selected hazard levels.

4. The research efforts outlined in Items 1, 2, and 3 might be made in conjunction with each other. The commonality between the requirements for different practice areas and different structure types is evident in the criteria reviewed for this synthesis. The implication is that consensus of standards for different performance levels and for different earthquake levels would be possible. Assuming that the application of the near-term results would be major/critical/essential structures, the combination of multiple performance levels with multiple hazards (multiple return periods) outlined in this item is perhaps the most logical for a research project to undertake.

5. Develop clear guidelines for establishment of desired performance, based on facility importance to emergency response, postearthquake recovery, regional transportation network, and regional economy. This effort would develop more clearly defined guidelines than those included in the current AASHTO documents. This work could include case studies of DOT experience and approaches. For example, Caltrans' approach to selection of performance objectives for bridges in the Bay Area would be useful in informing the development of the proposed guidelines. DOTs from around the country must have a stake in this effort, because seismic hazard and highway network use vary significantly across the country. Thus such guidelines should provide flexibility to address regional differences.

6. Development of cost data, estimated downtime, and reparability for different structural alternatives and associated levels of damage (e.g., spalling repair versus partial column replacement in the plastic hinge region). Some DOTs, such as Caltrans should have detailed information on repair costs and time to repair different types and levels of earthquake damage. A database could be created that could be geared for use by decision makers and senior design management during the establishment of performance objectives early in a project.

7. Use of local, state, or regional-level applied research projects, which use risk assessment tools such as REDARS, to help assess seismic risk to regional transportation networks. The objective would be to focus resources on specific economically important, but seismically vulnerable regions, as Oregon has done with its transportation network.

Longer-term research (5 years and beyond)—work products to fill the larger knowledge gaps might include completely new guide specifications with software tools for application of the PBSD methodology, but additional products might also accompany this guide.

8. Develop improved, enforceable performance objective requirements to support a PBSD specification that might be a stand-alone document similar in philosophy to the *International Code Council-Performance Code* (ICC-PC), but including more detail. This document might also include guidance on, or reference to, minimum prescriptive measures that would be applied where sufficient performance information is not available to provide a link between structural demands and risk of failure. An example of such a prescriptive measure might be the minimum ductility capacity of a column.

9. Compile databases of fragility information (probabilistic data for potential damage states) similar to those under development by the buildings industry. These would include data for all types of bridge structures and substructure commonly used in the United States. As part of this effort, or possibly a separate intermediate effort, the damage state descriptions

developed under Items 1, 2, and 3 in the short-term category would be used with fragility relationships. Such damage states may include primary structural systems of bridges and nonstructural systems, such as the roadway alignment. These should be standardized for consistency across the country.

10. Seismic risk should be considered relative to risk of loss by other hazards that are considered by the AASHTO specifications. Research is under way in the multiple-hazard area, and perhaps PBSD methodologies could be incorporated or extended to that work. Consideration of risk would combine hazard (loading) with response (capacity) to define outcomes (risk of nonperformance).

11. Development of facilitation tools to assist decision makers in making risk-informed or risk-based choices regarding seismic performance. Engineers could develop methods to clearly and succinctly explain the choices, impacts, and risks of earthquake damage to the public.

12. Education of bridge designers and stakeholders should be focal areas. Explaining and framing the questions related to PBSD will be challenging, and tools and training could be developed to improve the effectiveness of PBSD.
    a. For example, use of scenario-type approaches might be more effective with the general public than probabilistic data alone. ATC-58 (2002) encountered this in the building design area.
    b. Meaningful metrics, such as downtime, costs of repair, and loss of life, are more relevant to the public than EDPs.

Drawing from the trajectory toward PBSD in the buildings practice area, a few observations extrapolated to the bridge practice area are provided here. A regulatory framework will likely need to be developed to support PBSD. The framework should be robust enough to permit innovation and unique solutions that may not have been used before, but realistic enough to ensure that minimum standards are met. This means that simple statements of desired performance are likely not going to be sufficient. It is important that some means of accountability be incorporated to ensure that reasonable solutions are achieved. Such methods may include peer review, limited prescriptive requirements, demonstration projects, laboratory testing, and other controls. The ICC-PC provides some of the framework for buildings, but more detail is likely to be added as the practice of PBSD evolves.

Some of the previously listed research recommendations can be handled in the traditional research arena, primarily at universities or research centers. However, some of this work should be completed, or at least augmented, by teams of practicing bridge engineers, and, therefore, might be completed outside of the traditional research avenues. The code development work will likely require this second type of team. Work in the buildings area is following a similar approach.

Performance-based design is not unique to seismic design, and there have been pressures to develop performance-based specifications for other areas of engineering. The ICC-PC is an example, where earthquake loading is but one of many topics covered. Elsewhere, there have been efforts to put such things as concrete mix design into a fully performance-based framework. These efforts have had their champions and detractors, their successes and failures. It would be prudent to keep abreast of such activities, their progress, their success, and their stumbles. An overall steering panel may be able to provide continuity in these areas.

CONCLUSIONS

Performance-based seismic design of bridges will likely be achieved in incremental steps, with small extensions of existing practice occurring first. More of the reliability-based probabilistic design features may be added, but it will take time to develop the necessary methodology, databases, technical expertise, and willingness on the part of the public and of owners to use PBSD in a meaningful way.

Moves toward PBSD have already occurred in project-specific criteria that have been developed out of the desire on the part of informed owners to protect the public's investment in new major/critical/essential bridges or retrofit of existing important bridges. The knowledge developed for such projects provides an ideal departure point for implementing nationally applicable consistent guidelines for such major/critical/essential bridges. As ambitious as this sounds, such a guide would be a first step on the road to full PBSD, where decisions are made by an informed public/owner and stakeholders using metrics that are not engineering based, but rather based on impacts and risks to the public and stakeholders that include direct losses, indirect losses, costs of construction, cost of repair, and estimates of downtime—more meaningful metrics than engineering design parameters such as strains, rotations, and displacements.

It is likely that PBSD is a tool that will be used for larger, more important projects and perhaps not for conventional or ordinary bridges without good justification. The experience of the building industry is that PBSD requires significant technical skills beyond those of the ordinary engineer. Such skills typically do not come cheaply, and the analysis and design effort are proportionately more sophisticated. That said, this does not mean that some aspects of PBSD cannot be used on conventional bridges, such as checks of operational performance under smaller earthquake shaking.

The bottom line is that PBSD may be useful to the profession in several ways, and there likely is a short-term strategy that will serve the engineering community well and a long-term strategy that will serve the public with more meaningful data and decision points regarding the design for natural hazards in a way that heretofore has not been possible. This will be a technical and an educational challenge for the bridge engineering community in the coming years.

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APPENDIX A

NCHRP 43-07 Performance Based Seismic Bridge Design

Agency and Responder Information

Dear *State Bridge Engineer.*

The Transportation Research Board (TRB) is preparing a synthesis on *Performance Based Seismic Bridge Design*. This is being done for NCHRP, under the sponsorship of the American Association of State Highway and Transportation Officials, in cooperation with the Federal Highway Administration.

The current AASHTO seismic provisions are based on a single "no collapse" damage state, and as such are not fully based on performance-based design (PBD) concepts. However, PBD has been used on a limited number of projects, and the concept shows promise for improving bridge seismic design. An NCHRP Synthesis Topic 43-07 project is underway to gather information and develop a status summary of performance-based seismic design for bridges. This questionnaire is being sent to request information from your organization regarding 1) your use or potential use of PBD and 2) your methods of seismic design.

This questionnaire is being sent to *U.S. state departments of transportation.* Your cooperation in completing the questionnaire will ensure the success of this effort. **If you are not the appropriate person at your *agency* to complete this questionnaire, please forward it to the correct person.**

Please compete and submit this survey by *March 26, 2012.* We estimate that it should take approximately *45 minutes* to complete. If you have any questions, please contact our principal investigator *Lee Marsh, Lee.Marsh@abam.com, 206-431-2340.* Any supporting materials can be sent directly to *Lee* by email or at the postal address shown at the end of the survey.
QUESTIONNAIRE INSTRUCTIONS

1. Questions with an asterisk * are required.
2. To view and print the entire questionnaire, Click on the following link and print using "control p" http://surveygizmolibrary.s3.amazonaws.com/library/64484/NCHRP4307Survey.pdf
3. To save your partial answers and complete the questionnaire later, click on the "Save and Continue Later" link in the upper right hand corner of your screen. A link to the incomplete questionnaire will be emailed to you from *SurveyGizmo*. To return to the questionnaire later, open the email from *SurveyGizmo* and click on the link.
4. To pass a partially completed questionnaire to a colleague, click on the on the "Save and Continue Later" link in the upper right hand corner of your screen. A link to the incomplete questionnaire will be emailed to you from *SurveyGizmo*." Open the email from *SurveyGizmo* and forward it to a colleague.
5. To view and print your answers before submitting the survey, click forward to the page following question 29. Print using "control p."

6. To submit the survey, click on "Submit" on the last page.

Thank you very much for your time and expertise.

A working definition of PBD for seismic design of bridges is: criteria and methodology that links post-earthquake operation to a specific damage state that is in turn linked to typical engineering design parameters (strain levels, displacements, forces, etc). Thus: Performance Level > Damage State > Design Parameters. A feature of PBD is that a designer or owner may chose what performance is desired, then the design is developed to acheive such performance.

Please enter your contact information.

First Name *

Last Name *

Title

Agency/Organization *

Street Address

Suite

City *

State *

Zip Code

Country *

Email Address

Phone Number

Please enter the date (MM/DD/YYYY). *

Seismic Classification

1. Is your state or region a "seismic" state? *
   In other words, do you routinely use seismic design provisions beyond those for the lowest category? For example in the US, do you have areas where the Seismic Zone by AASHTO LRFD is 2, 3 or 4, or where the SDC by the AASHTO Seismic Guide Specification is B, C or D?
   Note that if you check "NO" you will exit the survey at this point.

   - Yes
   - No

Seismic Design Questions

2. What seismic design provisions does your state or agency use? *

   - AASHTO LRFD Bridge Design Specifications (Force-Based Procedure)
   - AASHTO Guide Specifications for LRFD Seismic Bridge Design (Displacement-Based Procedure)
   - Other

If you answered "Other" above, please briefly describe or provide a link to your design provisions.
Note that at the end of the survey you will be provided an opportunity to upload files that may be relevant to this project.

3. If you are using the AASHTO LRFD force-based procedure currently, do you have any plans to change over to the AASHTO Seismic Guide Specification displacement-based procedure in the near future?

   - Yes
   - No
   - Not Applicable

4. If you are using "Other" seismic design provisions currently, do you have any plans to change

over to the AASHTO Seismic Guide Specification displacement-based procedure in the near future?

- Yes
- No
- Not Applicable

5. Does your state or agency include additions or modifications to the AASHTO seismic provisions in your Bridge Design Manual?

- Yes
- No

If yes, please provide description, link or URL.

6. Does your state or agency use either single or multi-level performance-based seismic design for new bridges, other than the "No Collapse" performance level in AASHTO LRFD and the Seismic Guide Specifications? *

- Yes
- No

If yes, how were these used?
Check all that apply.

- project-specific basis
- typical projects
- design-bid-build projects
- design-build projects
- not applicable
- other, please describe

7. If you use an alternate or more specific definition of Performance-Based Design than the working definition provided in this survey, please summarize your definition below.

A working definition of PBD for seismic design of bridges is: criteria and methodology that links post-earthquake operation to a specific damage state that is in turn linked to typical engineering design parameters (strain levels, displacements, forces, etc).

8. If your state or agency uses multi-level seismic performance criteria (multiple return periods with different performance objectives), please describe the reason(s) for doing so.

9. If performance-based design criteria have been used, would you be willing to share the criteria with the NCHRP 43-07 project? *

- Yes
- No

If yes, please provide description, link or URL.

10. Have you ever used performance-enhancing measures, such as isolation or damping systems?

- Yes
- No

11. If yes, what was the desired performance level?

- No collapse/life safety
- Repairable damage

- Operational for emergency vehicles
- Fully operational

12. Does your state or agency have a seismic retrofit program?

- Yes
- No

If no, do you implement seismic improvements when other work is done? Please provide brief description.

13. If you have a seismic retrofit program, do you use performance-based criteria beyond new bridge provisions intended to meet the "No Collapse" damage state to design retrofits?

- Yes
- Yes - Use FHWA's Seismic Retrofitting Manual for Highway Structures - Part 1, Bridges, including the two-level performance criteria
- No
- Not Applicable, don't have a retrofit program

If yes, please provide a brief description, link or URL to the criteria?

Seismic Research Sponsored by Your Organization

14. Does your state or agency sponsor seismic design related research?
If yes, please provide a listing or link to recent projects (last 10 yrs) and researchers.

- Yes
- No

If yes, please provide brief listing, link or URL.

15. What is the purpose of the research?
Check all that apply.

- Improve seismic performance of standard bridges
- Develop design criteria not adequately covered by standards
- Develop performance-based criteria and methods
- Other

If you checked other, please describe the intent of such research or provide a link or URL if relevant to this synthesis project.

16. Has your state or agency sponsored research correlating damage to engineering design parameters (strain levels, displacements, forces, etc)?

- Yes
- No

If yes, please provide description, link or URL.

17. What are the most important areas for which research is needed to deploy Performance Based Design *
Ranking: 1 - most important, and each entry must have a unique ranking (i.e. a ranking value may only be used once).

	1	2	3	4	5	6	7
Correlation between performance level and damage states							
Construction cost data for higher performance levels							
Structural displacement limits for operability							
Non-structural displacement limits for operability							
Strain or rotation limits for given damage states							
Probablilistic data for damage states							
Improved structural analysis techniques							

Decision Making for Seismic Design

18. Does your state or agency have a policy for establishing bridge operational classification (for example: Critical, Essential, or Other bridges as defined in AASHTO LFRD Bridge Design Specifications)?

- Yes
- No

19. Does your state or agency use criteria for the design of Critical or Essential bridges beyond that given in the AASHTO LRFD Bridge Design Specifications?

- Yes
- No
- Varies on case-by-case basis

If yes, what performance levels are targeted?

20. If you implemented Performance Based Design, would you prefer if the procedures were *

- Optional as augmentation to the existing "No Collapse" (life-safety) design requirements
- Optional as replacement (i.e. new) guidelines
- Mandatory requirements
- Not sure

21. If you implemented Performance Based Design, would you be more likely to apply it to

- major/critical/essential bridges
- conventional bridges
- both
- not sure

22. Would you consider Performance Based Design of conventional bridges if design or

construction costs increased, but bridge performance was significantly improved? Check the box corresponding to the cost increase range you would be willing to bear for each performance level listed on the left. *

In the table below, the Frequent EQ may be considered one that happens once every 50 to 200 years on average. The Rare EQ may be considered the current 1000-year design event.

	Costs Increase 0 - 10%	Costs Increase 10 - 25%	Costs Increase 25 - 50%	Costs Increase > 50%	Would Not Consider PBD for this Category
Immediate Access for Emergency Vehicles in Frequent EQ	○	○	○	○	○
Immediate Access for All Vehicles in Frequent EQ	○	○	○	○	○
Immediate Access for Emergency Vehicles in Rare EQ	○	○	○	○	○
Immediate Access for All Vehicles in Rare EQ	○	○	○	○	○

23. Would you consider Performance Based Design of major/critical/essential bridges if design or construction costs increased, but bridge performance was significantly improved? Check the box corresponding to the cost increase range you would be willing to bear for each performance level listed on the left. *

In the table below, the Frequent EQ may be considered one that happens once every 50 to 200 years on average. The Rare EQ may be considered the current 1000-year design event.

	Costs Increase 0 - 10%	Costs Increase 10 - 25%	Costs Increase 25 - 50%	Costs Increase > 50%	Would Not Consider PBD for this Category
Immediate Access for Emergency Vehicles in Frequent EQ	○	○	○	○	○
Immediate Access for All Vehicles in Frequent EQ	○	○	○	○	○
Immediate Access for Emergency Vehicles in Rare EQ	○	○	○	○	○
Immediate Access for All Vehicles in Rare EQ	○	○	○	○	○

24. Please rank the following impediments to the use of Performance Based Design. *

Rank - 1 is the highest impediment, and each entry must have a unique ranking (i.e. a ranking value may only be used once).

	1	2	3	4	5	6	7
Lack of proven methodologies and design standards	○	○	○	○	○	○	○
Increased design costs	○	○	○	○	○	○	○
Increased construction costs	○	○	○	○	○	○	○
Legal risks of not meeting targeted performance goals	○	○	○	○	○	○	○
Lack of decision-making tools for establishing performance levels	○	○	○	○	○	○	○
Lack of staff with adequate technical skills and expertise	○	○	○	○	○	○	○
Difficulty in quality control, in-house or contracted work	○	○	○	○	○	○	○

25. Do you have the necessary tools/data to frame PBD or operational criteria decisions for upper management, policy makers, and/or the public? *

- ○ Yes
- ○ No
- ○ Not sure

If answer to previous question is no, what information is needed to determine whether increased design or construction costs should be expended to improve post-earthquake performance? Check all that apply.

- Daily use of the bridge, ADT
- Estimate of increased design costs
- Estimate of increased construction costs
- Estimate of repair costs in the event a damaging earthquake occurs
- Estimate of user delay costs in the event a damaging earthquake occurs
- Correlation between performance, damage state, and engineering design parameters
- Emergency response plans
- Other, please describe

26. Is adequate information available for your agency to formulate a statement for the public explaining what to expect in terms of availability and recovery of bridges following a design earthquake?

- Yes
- No

If no, briefly describe information you need that would be helpful in preparing a statement.

27. Do you have any comments you would like us to consider?

File Upload Option

28. If you have any information in electronic file format that you would like to share, please upload here. For example, agency specific design specification excerpts, project criteria, or similar documents are welcome.

Choose File No file selected Upload

Review of Questionnaire

Thank You!

Thank you for taking our survey. Your response is very important to us. If you have any questions or comments, please feel free to contact *Lee Marsh* at:

- E-mail: *Lee.Marsh@abam.com*
- Phone: 206-431-2340
- Mailing Address: *BergerABAM, 33301 Ninth Ave South, Federal Way, WA 98003*

APPENDIX B

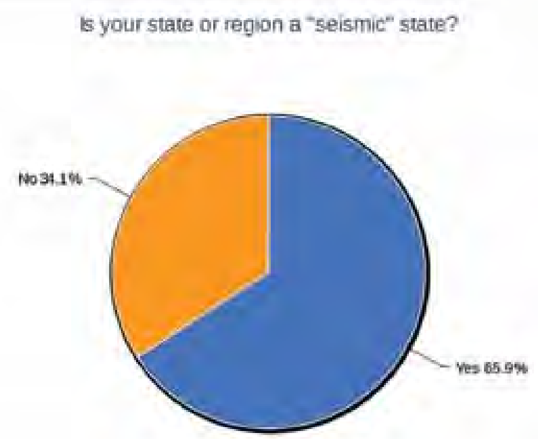
Online Surveys, Data Collection and Integration
www.SurveyGizmo.com

Summary Report - Oct 24, 2012

Survey: NCHRP 43-07 Performance Based Seismic Bridge Design

Agency/Organization

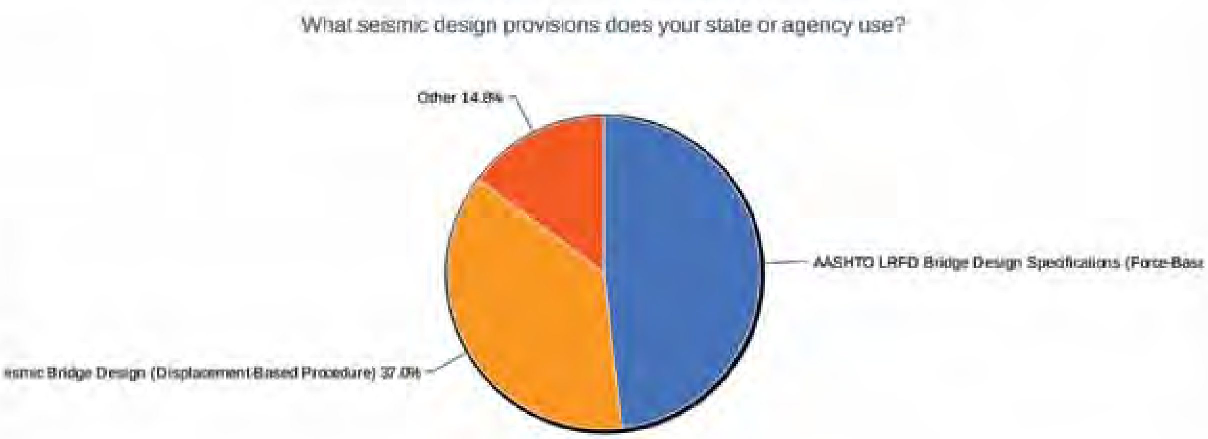
Count	Response
1	Aalbama Department of Transportation
1	Arizona Department of Transportation
1	Arkansas Highway and Transportation Department
1	CDOT Staff Bridge
1	California Department of Transportation
1	Delaware Department of Transporation
1	Georgia DOT
1	Hawaii DOT
1	INDOT
1	Idaho Transportation Department
1	Illinois DOT
1	Iowa Department of Transportation
1	Kentucky Department of Highways
1	LADOTD
1	Maryland State Highway Administration
1	MassDOT
1	Minnesota Department of Transportation
1	MoDOT/Bridge Division
1	Montana Dept of Transportation
1	NCDOT
1	NDDOT
1	NHDOT - Bureau of Bridge Design
1	NMDOT
1	Nebraska Department of Roads
1	Nevada DOT
1	New Jersey DOT
1	OK DOT
1	Office of Strucures / New York State Dept of Transportation
1	Ohio DOT
1	Oregon DOT
1	Pennsylvania Department of Transportation
1	SCDOT
1	South Dakota Department of Transportation
1	State of Alaska DOT&PF
1	Tennessee DOT Structures Division
1	Texas Department of Transportation
1	Utah Department of Transportation
1	VIRGINIA DOT
1	Vermont Agency of Transportation
1	WSDOT
1	WYDOT



1. Is your state or region a "seismic" state?

Value	Count	Percent %
Yes	27	65.9%
No	14	34.2%

Statistics	
Total Responses	41



2. What seismic design provisions does your state or agency use?

Value	Count	Percent %
AASHTO LRFD Bridge Design Specifications (Force-Based Procedure)	13	48.2%
AASHTO Guide Specifications for LRFD Seismic Bridge Design (Displacement-Based Procedure)	10	37.0%
Other	4	14.8%

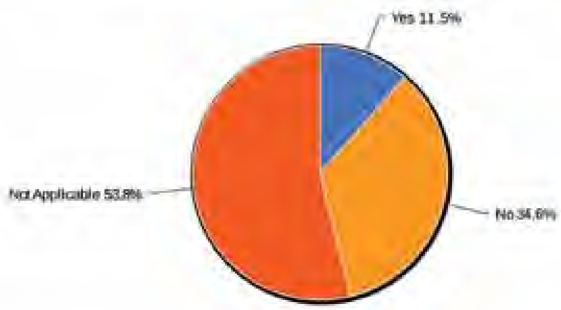
Statistics	
Total Responses	27

If you answered "Other" above, please briefly describe or provide a link to your design provisions.

Count	Response
1	AASHTO 17th Edition
1	Engineer has the option to use either Force-Based or Displacment-Based design procedures.

1	The Department allows either Force-Based or Displacement-Based Procedure.
1	We allow both but have a preference for the Design Specs. because the Guide Specs. do not directly address essential bridges.
1	We use AASHTO LRFD Bridge Design Specifications modified by NYS Blue pages (we analyze all bridges even in Seismic Zone 1 and design substructuers for the actual forces rather using minimum provisions as defined in code). Alternatively, we also allow AASHTO Guide Specifications for LRFD Seismic Bridge Design.
1	Caltrans Seismic Design Criteria (SDC) http://www.dot.ca.gov/hq/esc/earthquake_engineering/SDC/#SDC
1	SCDOT Seismic Design Specifications for Highway Bridges. http://www.scdot.org/doing/bridge/bridgeseismic.shtml

If you are using the AASHTO LRFD force-based procedure currently, do you have any plans to change over to the AASHTO Seismic Guide Specification displacement-based procedure in the near future?

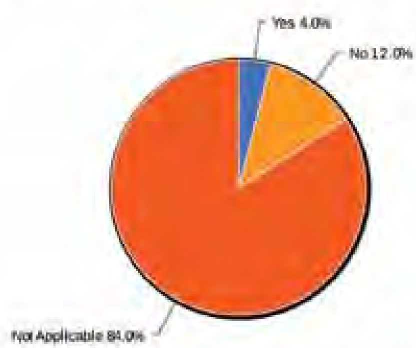


3. If you are using the AASHTO LRFD force-based procedure currently, do you have any plans to change over to the AASHTO Seismic Guide Specification displacement-based procedure in the near future?

Value	Count	Percent %
Yes	3	11.5%
No	9	34.6%
Not Applicable	14	53.9%

Statistics	
Total Responses	26

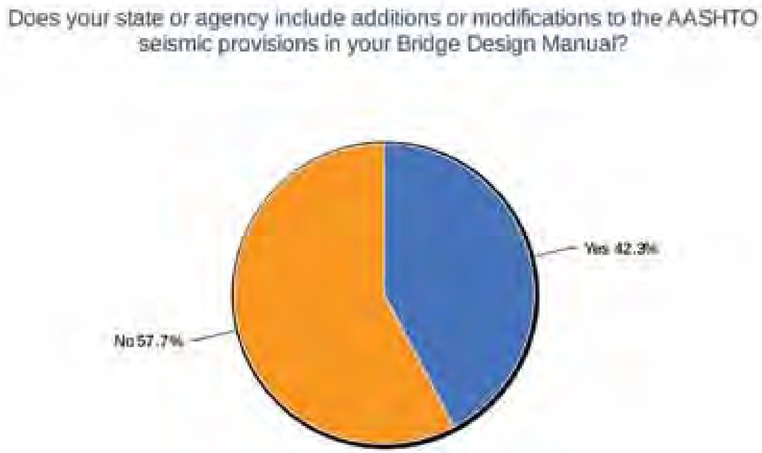
If you are using "Other" seismic design provisions currently, do you have any plans to change over to the AASHTO Seismic Guide Specification displacement-based procedure in the near future?



4. If you are using "Other" seismic design provisions currently, do you have any plans to change over to the AASHTO Seismic Guide Specification displacement-based procedure in the near future?

Value	Count	Percent %
Yes	1	4.0%
No	3	12.0%
Not Applicable	21	84.0%

Statistics	
Total Responses	25



5. Does your state or agency include additions or modifications to the AASHTO seismic provisions in your Bridge Design Manual?

Value	Count	Percent %
Yes	11	42.3%
No	15	57.7%
If yes, please provide copy or link	0	0.0%

Statistics	
Total Responses	26

If yes, please provide description, link or URL.

Count	Response
1	Specify minimum PGA, Ss, S1 on a regional basis, per section 12.3.5 of Structures Manual.
1	http://www.dot.ca.gov/hq/esc/earthquake_engineering/SDC/#SDC
1	http://www.oregon.gov/ODOT/HWY/BRIDGE/standards_manuals.shtml
1	http://www.udot.utah.gov/main/uconowner.gf?n=11402707871066991
1	http://www.wsdot.wa.gov/Publications/Manuals/M23-50.htm
1	still working on Bridge Manual provisions -- mostly cold climate effects on material properties
1	Critical/Essential Bridges are to be designed allowing for a seismic hazard corresponding to a Two Percent Probability of Exceedance in 50 years (approximately 2500-year Return Period)
1	As explained under #2, we have modified AASHTO Code by NYS Blue Pages. Link will be provided at the end.
1	We require single-span bridges to be analyzed except for those in Seismic Zone 1 or Seismic Category A depending of which Specification is used.
1	NJDOT Design Manual for Bridges and Structures Section 38 http://www.state.nj.us/transportation/eng/documents/BSDM/
1	Seat length, R-Factors, and some details requirements See:http://www.dot.il.gov/bridges/brmanuals.html

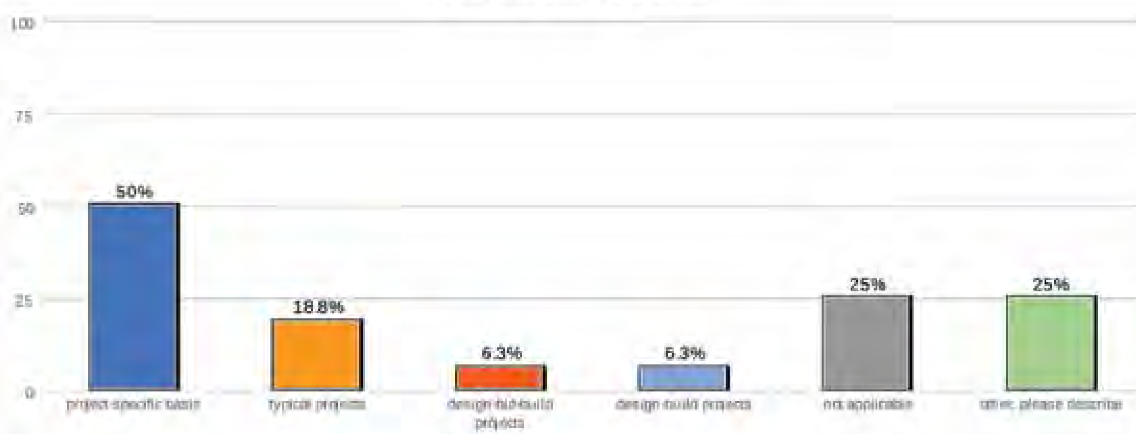
Does your state or agency use either single or multi-level performance-based seismic design for new bridges, other than the "No Collapse" performance level in AASHTO LRFD and the Seismic Guide Specifications?

6. Does your state or agency use either single or multi-level performance-based seismic design for new bridges, other than the "No Collapse" performance level in AASHTO LRFD and the Seismic Guide Specifications?

Value	Count	Percent %
Yes	10	37.0%
No	17	63.0%

Statistics	
Total Responses	27

If yes, how were these used?



If yes, how were these used?

Value	Count	Percent %
project-specific basis	8	50.0%
typical projects	3	18.8%
design-bid-build projects	1	6.3%
design-build projects	1	6.3%
other	0	0.0%
not applicable	4	25.0%
other, please describe	4	25.0%

Statistics	
Total Responses	16

Open-Text Response Breakdown for "other, please describe"	Count
For very large major bridges, we also develop project specific performance-based provisions	1

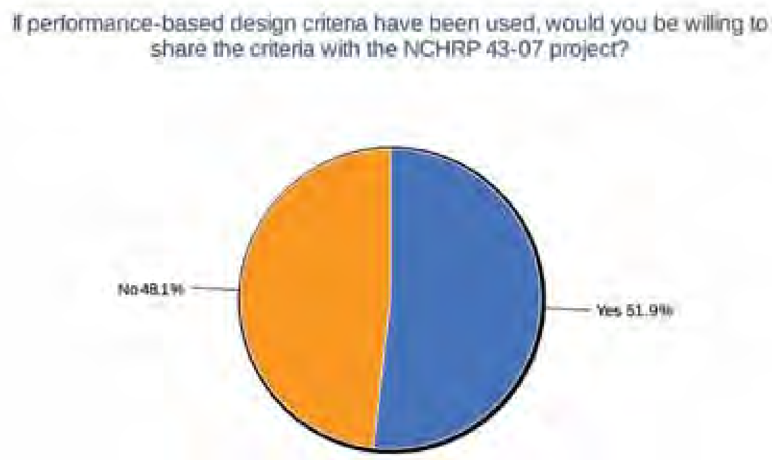
Two level criteria are used for Critical Category Bridges.	1
We used a fully operational performance criteria on a cable stayed bridge over the Mississippi River in St Louis	1
Some large, unconstructed bridge projects had seismic criteria developed for multiple level hazards (100, 500 and 2500) but is old and "force based"	1

7. If you use an alternate or more specific definition of Performance-Based Design than the working definition provided in this survey, please summarize your definition below.

Count	Response
1	Definition is OK.
2	N/A
2	NA
1	Operational; Minimal damage
1	The proposed definition is acceptable.
1	na
1	UDOT uses this definition with supplemental information for performance levels. Refer to design memorandum link above.
1	We use a 2-level design criteria for typical bridges - no collapse (life safety) at the 1000-year return interval and serviceability (usable to emergency vehicles within 72 hours) at the 500-year recurrance interval.
1	Design typical bridge (project-specific base) based on displacement verificaion in accordance with seismic guide spec.

8. If your state or agency uses multi-level seismic performance criteria (multiple return periods with different performance objectives), please describe the reason(s) for doing so.

Count	Response
1	Based on operational classification of bridge.
1	For critical bridges
1	Mega Projects only
1	N/A
1	NA
1	Post-earthquake emergency response Regional economic impact of bridge closure
1	UDOT does not use multiple return periods for seismic performance.
1	We may consider multi-level seismic performance criteria for major bridges.
1	na
1	The major seismic event Oregon is focused on preparing for is a Cascadia Subduction Zone event. The return period is about 300 to 350 years. In order to capture that event with serviceable bridges for response and rescue, we chosse to design for service at the 500-year recurrance interval in addition to the 1000-year no collapse criteria in the Guide Specs.
1	Critical Bridges are to be analyzed for two earthquake hazard design levels; a lower level event (functional/design level) for 1000 year Return Period and an upper level event (safety evaluation/design level) for 2500 year Return Period. We also have divided the State of New York into two areas, Downstate Zone and the rest of New York. For Downstate Zone, we have zone specific spectra hazard values to be used in the seismic analysis. Also, we have little different performance crireia for Downstate Zone for Critical, Essential and Other category of bridges. For the rest of NY state, we use AASHTO/USGS spectra values.
1	The FEE (Functional Evaluation Earthquake) to ensure that at a low level earthquake our bridges do not suffer any damage.



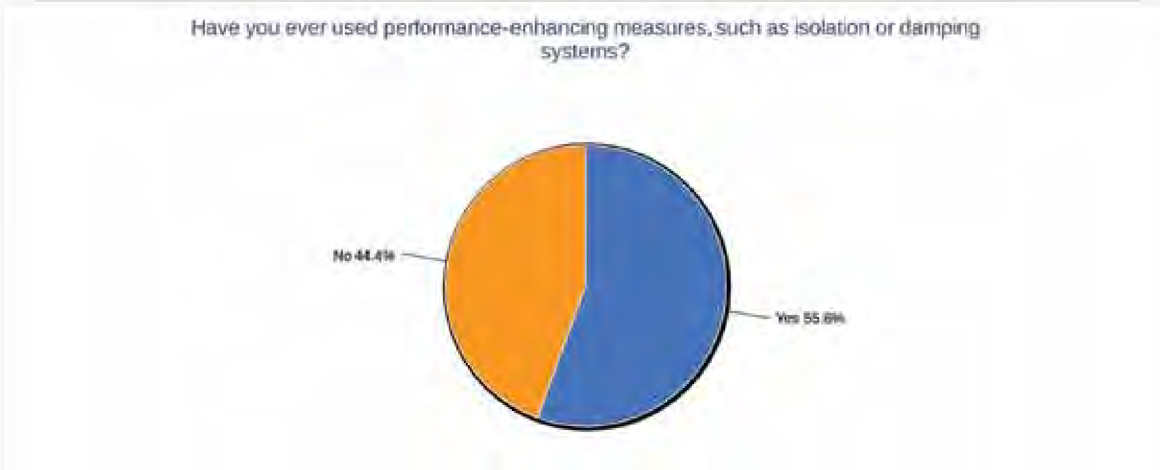
9. If performance-based design criteria have been used, would you be willing to share the criteria with the NCHRP 43-07 project?

Value	Count	Percent %
Yes	14	51.9%
No	13	48.2%
If yes, provide a link or email for follow up	0	0.0%

Statistics	
Total Responses	27

If yes, please provide description, link or URL.

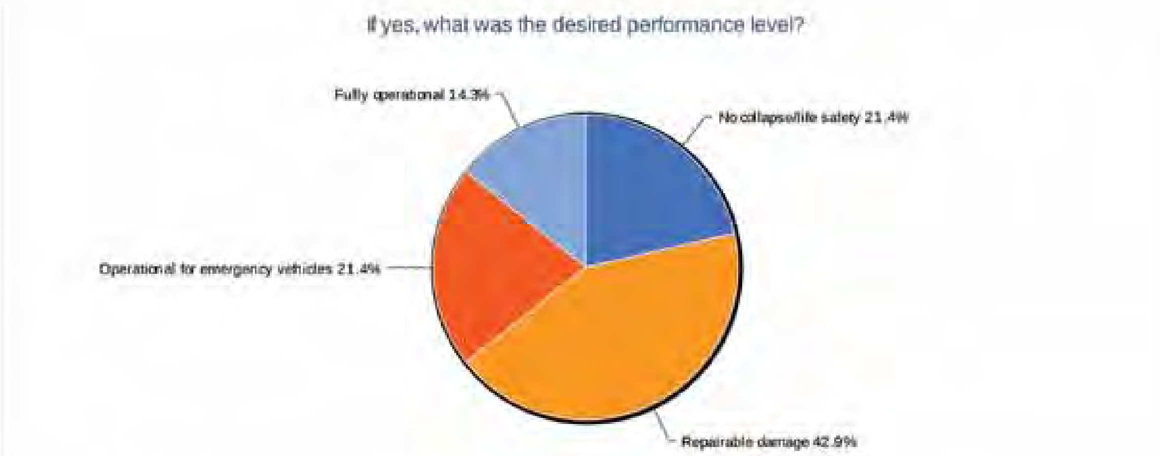
Count	Response
1	Columbia River Crossing Design Criteria
1	Link will be provided at the end.
1	Loaded on FTP site already.
1	N/A, If performance based design had been used, NC would be willing to share
1	Please let me know and I will search for the seismic criteria for the large projects
1	We haven't used any.
1	We'd have to search the design files and contract documents which would take a lot of time.
1	http://www.udot.utah.gov/main/uconowner.gf?n=11402707871066991
1	http://www.wsdot.wa.gov/Publications/Manuals/M23-50.htm
1	mesale@scdot.org
1	same as #5; To be modified if necessary based on recent State research project



10. Have you ever used performance-enhancing measures, such as isolation or damping systems?

Value	Count	Percent %
Yes	15	55.6%
No	12	44.4%

Statistics	
Total Responses	27



11. If yes, what was the desired performance level?

Value	Count	Percent %
No collapse/life safety	3	21.4%
Repairable damage	6	42.9%
Operational for emergency vehicles	3	21.4%
Fully operational	2	14.3%

Statistics	
Total Responses	14

12. Does your state or agency have a seismic retrofit program?

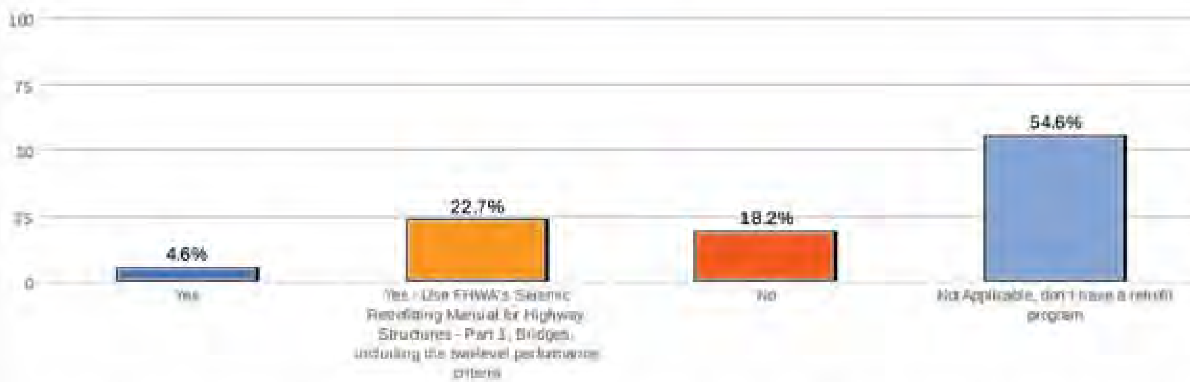
Value	Count	Percent %
Yes	9	34.6%
No	17	65.4%
If no, do you implement seismic improvements when other work is done? Please provide brief description.	0	0.0%

Statistics	
Total Responses	26

If no, do you implement seismic improvements when other work is done? Please provide brief description.

Count	Response
1	Bearings & Integral/Semi-integral abutments
1	Case by Case basis.
1	No
1	No.
1	The steel rocker bearings are replaced with a concrete or steel pedestal.
1	We have retrofitted a major structure - Interstate 40 over the Mississippi River
1	We upgrade bearing restraint features on maintenance projects.
1	Yes - Will evaluate each bridge when designing rehabilitation projects
1	Yes but only if costs do not exceed complete replacement.
1	Yes. If it is within the seismic zone and it is necessary.
1	no
1	Seismic retrofit program is implemented whenevr the bridge work is included for Capital Program. In Downstae Zone (NYCDOT projects), some bridges are programmed for seismic retrofitting only.
1	The seismic program has recently been phased out, but seismic improvments are still recommended when other bridge substructure work is required.
1	We include phase 1 retorfit on all rehab projects. We evaluate the bridge for phase 2 and include it when economically feasible.

If you have a seismic retrofit program, do you use performance-based criteria beyond new bridge provisions intended to meet the "No Collapse" damage state to design retrofits?



13. If you have a seismic retrofit program, do you use performance-based criteria beyond new bridge provisions intended to meet the "No Collapse" damage state to design retrofits?

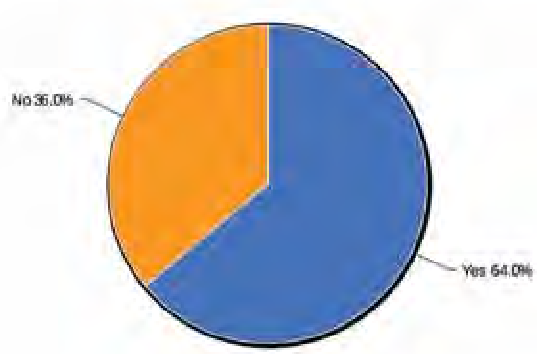
Value	Count	Percent %
Yes	1	4.6%
Yes - Use FHWA's Seismic Retrofitting Manual for Highway Structures - Part 1, Bridges, including the two-level performance criteria	5	22.7%
No	4	18.2%
Not Applicable, don't have a retrofit program	12	54.6%
If yes, could you provide criteria, references or links to the criteria?	0	0.0%

Statistics	
Total Responses	22

If yes, please provide a brief description, link or URL to the criteria?

Count	Response
1	Require higher level on a project specific basis. Only used for a limited number of bridges.
1	Same as #5 NJDOT Design Manual flow chart
1	We do not use spectra valuse for 100 year Return Period.
1	We use the FHWA Retorfit manual, except we use a 500-year serviceability criteria instead of a 100-year criteria.

Does your state or agency sponsor seismic design related research?



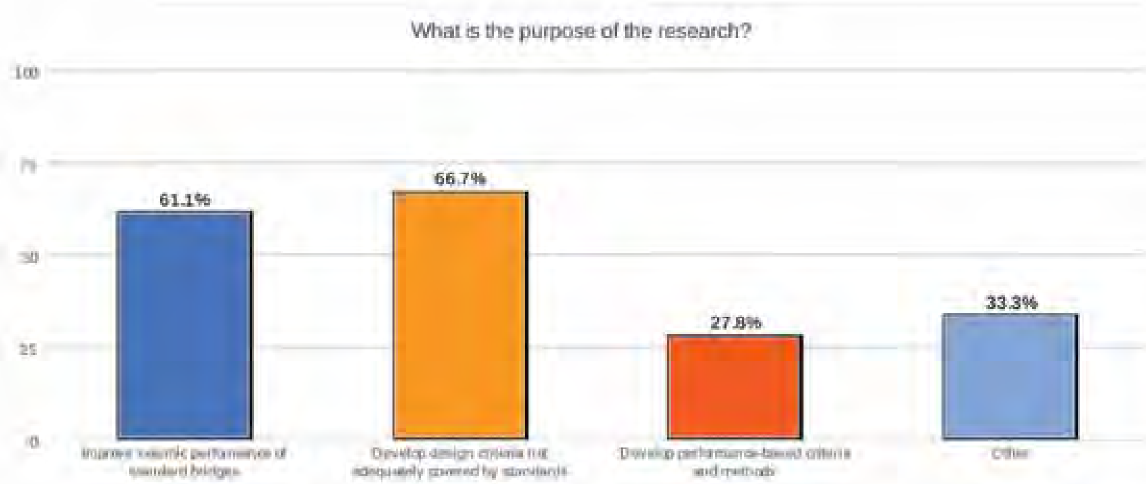
14. Does your state or agency sponsor seismic design related research?

Value	Count	Percent %
Yes	16	64.0%
No	9	36.0%
If yes, please provide listing or link	0	0.0%

Statistics	
Total Responses	25

If yes, please provide brief listing, link or URL.

Count	Response
1	Behavior of Pile to Cast-in-place & Precast Pile Cap Connections Subject to Seismic Forces
1	Evaluation of the Performance of steel Bridge Pedestals under Low Seismic Moad - May 2007
1	Not yet published. "Seismic Design Consideations"
1	There is currently a seismic research project still underway and not yet finalized.
1	Typically through pooled fund studies.
1	University of Washington seismic connections, grouted duct tests, Highways for LIFE project
1	http://cceer.unr.edu/ndot/twowayhome.html
1	http://docs.lib.purdue.edu/jtrp/324/
1	http://ine.uaf.edu/autc/projects/
1	http://www.ce.memphis.edu/pezeshk/publications.html
1	http://www.dot.ca.gov/hq/esc/earthquake_engineering/research_reports_site/
1	http://www.eng.auburn.edu/users/antons/downloads/Seismic_HRC_Report_Draft_2011.pdf
1	http://www.oregon.gov/ODOT/TD/TP_RES/ResearchReports.shtml
1	http://www.udot.utah.gov/main/f?p=100:pg:0::1:T,V:3836,
1	1. Seismic instrumentation of two bridges located within the New Madrid seismic zone. 2. Site specific ground motion analyses 3. Evaluation and mitigation of soil liquefaction in Arkansas
1	Seismic Design Guidelines for Bridges (1998 & 2008) sponsored by New York City DOT. 1998 report was adopted by NY State DOT for Downstate Zone. 2008 report is being reviewed by a researh team at present.



15. What is the purpose of the research?

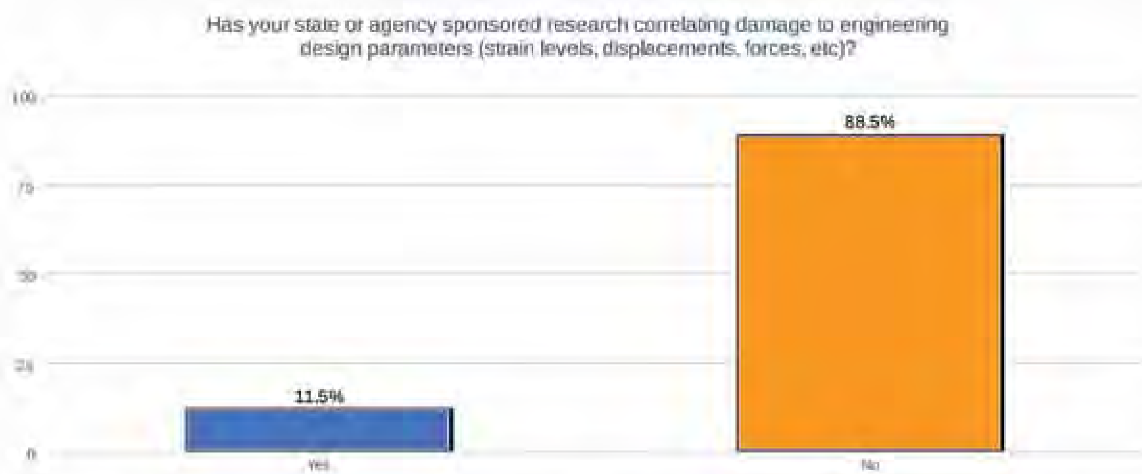
Value	Count	Percent %
Improve seismic performance of standard bridges	11	61.1%
Develop design criteria not adequately covered by standards	12	66.7%
Develop performance-based criteria and methods	5	27.8%

Statistics	
Total Responses	18

Other	6	33.3%
If you checked other, please describe the intent of such research or provide links if relevant to this synthesis project.	0	0.0%

If you checked other, please describe the intent of such research or provide a link or URL if relevant to this synthesis project.

Count	Response
1	Liquefaction, utility response curves, square column retrofit
1	Seismic Acceleration Map for AZ
1	To verify if bridge jacking pedestals would operate satisfactorily under low seismic load.
1	1. Measure potential damage and to provide warning system to motorists. 2. Reduce ground design motions 3. Improve method to determine liquefaction in Arkansas
1	To learn how the joint behaves with the actual SCDOT standard practice and to learn how the performance can be modeled correctly.



16. Has your state or agency sponsored research correlating damage to engineering design parameters (strain levels, displacements, forces, etc)?

Value	Count	Percent %
Yes	3	11.5%
No	23	88.5%
If yes, listing or link	0	0.0%

Statistics	
Total Responses	26

If yes, please provide description, link or URL.

Count	Response
1	See above research.
1	http://docs.lib.purdue.edu/jtrp/324/

17. What are the most important areas for which research is needed to deploy Performance Based

Design

Item	Total Score[1]	Overall Rank
Correlation between performance level and damage states	140	1
Structural displacement limits for operability	127	2
Strain or rotation limits for given damage states	105	3
Improved structural analysis techniques	93	4
Construction cost data for higher performance levels	90	5
Non-structural displacement limits for operability	88	6
Probabilistic data for damage states	75	7

Total Respondents: 26

[1] Score is a weighted calculation. Items ranked first are valued higher than the following ranks, the score is the sum of all weighted rank counts.

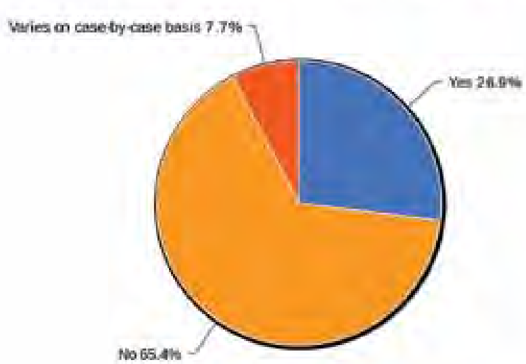
Does your state or agency have a policy for establishing bridge operational classification (for example: Critical, Essential, or Other bridges as defined in AASHTO LFRD Bridge Design Specifications)?

18. Does your state or agency have a policy for establishing bridge operational classification (for example: Critical, Essential, or Other bridges as defined in AASHTO LFRD Bridge Design Specifications)?

Value	Count	Percent %
Yes	13	50.0%
No	13	50.0%

Statistics	
Total Responses	26

Does your state or agency use criteria for the design of Critical or Essential bridges beyond that given in the AASHTO LRFD Bridge Design Specifications?



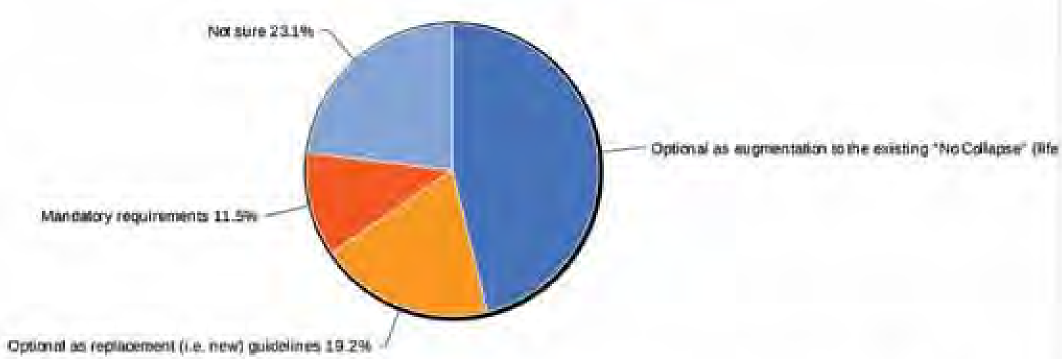
19. Does your state or agency use criteria for the design of Critical or Essential bridges beyond that given in the AASHTO LRFD Bridge Design Specifications?

Value	Count	Percent %
Yes	7	26.9%
No	17	65.4%
Varies on case-by-case basis	2	7.7%
If yes, what performance levels are targeted?	0	0.0%

Statistics	
Total Responses	26

If yes, what performance levels are targeted?

Count	Response
1	Limits on ductility, strains in steel and concrete
1	Operational
1	Repairable (essential) and Operational (critical)
1	varies
1	Critical/Essential Bridges are to be designed for a seismic hazard corresponding to a Two Percent Probability of Exceedance in 50 years (approximately 2500-year Return Period)
1	A Critical Bridge must provide immediate access after the lower level (functional - 1000 year Return Period) event and limited access after the upper level (safety - 2500 year Return Period) event. Critical Bridges shall survive the upperl level event (2500 year RP) with repairable damage. After the lower level event (1000 years RP), the bridge shall suffer only minimal damage.
1	Essentially elastic and repairable damage are targeted for large projects -- none of which have been constructed
1	OC I-Service Maintained; Damage Repairable (Standard classifications on the SDS also OC II and OC III). Only the CRB Cable Stayed Bridge was designed with the Critical Access Path (CAP Structures) to allow minimal damage and immediate service)



20. If you implemented Performance Based Design, would you prefer if the procedures were

Value	Count	Percent %
Optional as augmentation to the existing "No Collapse" (life-safety) design requirements	12	46.2%
Optional as replacement (i.e. new) guidelines	5	19.2%
Mandatory requirements	3	11.5%
Not sure	6	23.1%

Statistics	
Total Responses	26

If you implemented Performance Based Design, would you be more likely to apply it to

21. If you implemented Performance Based Design, would you be more likely to apply it to

Value	Count	Percent %
major/critical/essential bridges	13	54.2%
conventional bridges	0	0.0%
both	7	29.2%
not sure	4	16.7%

Statistics	
Total Responses	24

22. Would you consider Performance Based Design of conventional bridges if design or construction costs increased, but bridge performance was significantly improved? Check the box corresponding to the cost increase range you would be willing to bear for each performance level listed on the left.

	Costs Increase 0 - 10%	Costs Increase 10 - 25%	Costs Increase 25 - 50%	Costs Increase > 50%	Would Not Consider PBD for this Category	Responses
Immediate Access for Emergency Vehicles in Frequent EQ	61.5% 16	19.2% 5	0.0% 0	3.8% 1	15.4% 4	26
Immediate Access for All Vehicles in Frequent EQ	57.7% 15	15.4% 4	3.8% 1	0.0% 0	23.1% 6	26
Immediate Access for Emergency Vehicles in Rare EQ	38.5% 10	26.9% 7	3.8% 1	0.0% 0	30.8% 8	26
Immediate Access for All Vehicles in Rare EQ	34.6% 9	11.5% 3	0.0% 0	0.0% 0	53.8% 14	26

23. Would you consider Performance Based Design of major/critical/essential bridges if design or construction costs increased, but bridge performance was significantly improved? Check the box corresponding to the cost increase range you would be willing to bear for each performance level listed on the left.

	Costs Increase 0 - 10%	Costs Increase 10 - 25%	Costs Increase 25 - 50%	Costs Increase > 50%	Would Not Consider PBD for this Category	Responses
Immediate Access for Emergency Vehicles in Frequent EQ	34.6% 9	26.9% 7	15.4% 4	11.5% 3	11.5% 3	26
Immediate Access for All Vehicles in Frequent EQ	42.3% 11	26.9% 7	15.4% 4	3.8% 1	11.5% 3	26
Immediate Access for Emergency	26.9%	38.5%	15.4%	7.7%	11.5%	

Vehicles in Rare EQ	7	10	4	2	3	
Immediate Access for All Vehicles in Rare EQ	30.8% 8	23.1% 6	7.7% 2	11.5% 3	26.9% 7	26

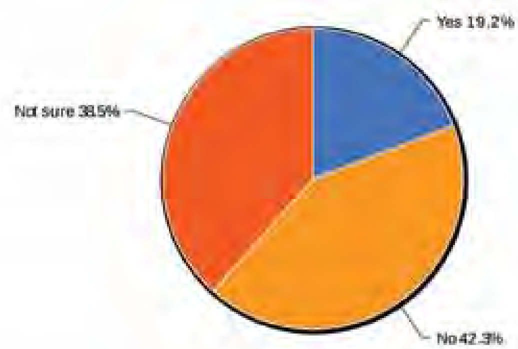
24. Please rank the following impediments to the use of Performance Based Design.

Item	Total Score[1]	Overall Rank
Lack of proven methodologies and design standards	145	1
Lack of decision-making tools for establishing performance levels	132	2
Increased construction costs	122	3
Increased design costs	95	4
Lack of staff with adequate technical skills and expertise	87	5
Legal risks of not meeting targeted performance goals	76	6
Difficulty in quality control, in-house or contracted work	52	7

Total Respondents: 26

[1] Score is a weighted calculation. Items ranked first are valued higher than the following ranks, the score is the sum of all weighted rank counts.

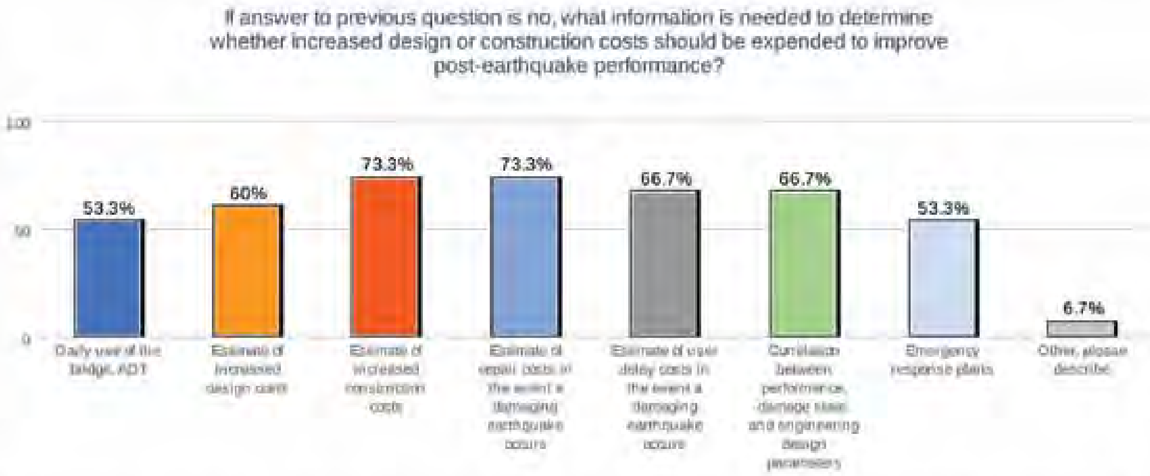
Do you have the necessary tools/data to frame PBD or operational criteria decisions for upper management, policy makers, and/or the public?



25. Do you have the necessary tools/data to frame PBD or operational criteria decisions for upper management, policy makers, and/or the public?

Value	Count	Percent %
Yes	5	19.2%
No	11	42.3%
If no, what information is needed?	0	0.0%
Not sure	10	38.5%

Statistics	
Total Responses	26

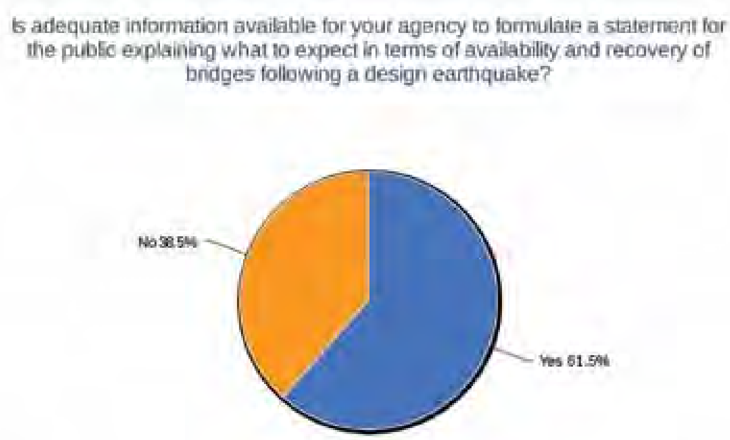


If answer to previous question is no, what information is needed to determine whether increased design or construction costs should be expended to improve post-earthquake performance?

Value	Count	Percent %
Daily use of the bridge, ADT	8	53.3%
Estimate of increased design costs	9	60.0%
Estimate of increased construction costs	11	73.3%
Estimate of repair costs in the event a damaging earthquake occurs	11	73.3%
Estimate of user delay costs in the event a damaging earthquake occurs	10	66.7%
Correlation between performance, damage state, and engineering design parameters	10	66.7%
Emergency response plans	8	53.3%
Other, please describe	1	6.7%

Statistics	
Total Responses	15

Open-Text Response Breakdown for "Other, please describe"	Count
All of the above, plus availability of detours. Complex decision requiring input from planners, locals, operations, and bridge engineering.	1



26. Is adequate information available for your agency to formulate a statement for the public explaining what to expect in terms of availability and recovery of bridges following a design earthquake?

Value	Count	Percent %
Yes	16	61.5%
No	10	38.5%
Maybe	0	0.0%
If no, briefly describe information you need that would be helpful in preparing a statement.	0	0.0%

Statistics	
Total Responses	26

If no, briefly describe information you need that would be helpful in preparing a statement.

Count	Response
1	Do more studies
1	New York State being a low seismic state, seismiv vulnerability statement has not been prepared.
1	Screening of all existing bridges to determine their seismic vulnerability.
1	The Department has not done any seismic assessment of our existing bridges. We are still working on the FHWA requirement to load rate our existing bridges
1	From the preconstruction PBD yes: but from the recovery point of view no. As per Q/25 costs/user delays
1	1. Items in No. 25. 2. Establish criteria for PBD (AASHTO & State). 3. Vulnerability assessment of existing bridges in vicinity
1	Yes, but not sure it is well understood by the public that bridges may be closed due to damage after an earthquake. Even retrofitted bridges.
1	Because we have just used LRFD designs for only 5 years and haven't had an earthquake to test those designs, we need about 15 to 20 years of more designs built to actually see how those bridges will perform. Any other public statement could not be backed by facts on bridge performance during and after an earthquake.

27. Do you have any comments you would like us to consider?

Count	Response
1	None
1	PBE needs to be pursued, especially for the sake of critical and essential bridges.
1	Delaware has a small region in Zone 2 in the northern part of our state. The majority of the state is in Zone 1. Seismic design is not critical for Delaware.
1	Until the FHWA or AASHTO comes out with design examples showing how to use AAHSTO Guide Specifications for LRFD Seismic Bridge Design or the Performance Based Seismic Design, we will probably continue to use the Force Based Performance Seismic Design. You may want to look at the 1996 FHWA Publication on Seismic Design of Bridge Design Examples (Publication No. FHWA-SA-97-006 thru 012). The design examples were very helpful in implementing the Force Based Performance Seismic Design.
1	New York state being a low seismic state, for the design of conventional bridges especially on good soil seismic may not control the design. Cost increase factor for question # 22 is not an issue at all. Here is the link to our modifications to LRFD AASHTO Code where we have revised seismic provisions for bridges in Zone 1 as explained earlier. https://www.dot.ny.gov/divisions/engineering/structures/repository/manuals/LRFD_Blue_Pages_9-2011.pdf Please note this link is for NYS Blue Pages modifications which inclue non seismic changes as well. Thanks.
1	Georgia is a low risk seismic state having maximum accelerations of about 0.12 g. Seismic design minimally affects our bridges.
1	Many parameters would need to be considered for establishing a bridge's performance objectives (ADT, detour length, facilities served, economic impacts, etc). In smaller states, most bridges may be required to deliver a higher performance since there are not transportation alternatives.
1	New Jersey is located in relatively low seismic zone, but if damage occurs and bridges are closed (especially connected to NYC and Barrier islands) due to earthquake, the most concern is significant social and economical impacts. Therefore, higher performance level needs to be considered. Note: #17 and #24 answers do not show in summary. Here they are: #17 2,1,3,7,6,5,4 #24 2,6,3,5,1,4,7
1	If Frequent earthquake criteria available, We may consider Frequent earthquake criteria as shown in answer 22 & 23. Cost increase shown in answer 22 and 23 will be considered based on seismic design category and project-specific basis.

Abbreviations used without definition in TRB Publications:

AAAE	American Association of Airport Executives
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACI–NA	Airports Council International–North America
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	Air Transport Association
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
NASA	National Aeronautics and Space Administration
NASAO	National Association of State Aviation Officials
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
SAE	Society of Automotive Engineers
SAFETY-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation

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