ATC-58 Project Task Report

Phase 2, Task 2.3

Engineering Demand Parameters for Nonstructural Components

Prepared for the

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1. INTRODUCTION

1.1 Engineering Demand Parameters

Engineering Demand Parameters (*EDPs*) are structural response quantities that can be used to estimate damage to structural and nonstructural components and systems. Phase 2 of the ATC-58 project to develop next-generation performance-based seismic design guidelines includes tasks related to the identification of *EDP*s for structural and nonstructural components used in existing codes, guidelines and resource documents. Later phases of the project include tasks related to identification of recommended *EDP*s for use in the next-generation performance-based design guidelines for structural and nonstructural systems. This report presents the results of a literature survey of *EDP*s in use at this time for predicting the earthquake performance of nonstructural components and systems. Also presented in this report is a preliminary categorization of the broad types of nonstructural components for which damage may be best correlated with particular *EDPs*. A thorough examination and categorization of different types of nonstructural components and systems, and the EDPs emerging from current research will be the subject of future work and is not included herein.

Performance-based design can be a useful approach for mitigating the potential losses due to extreme hazards other than earthquakes (e.g., blast, fire and hurricane), but the extension of performance-based design to these other hazards is still in its infancy. A framework for performance-based blast engineering for structural systems is presented in a companion document to this report (Whittaker, 2004). The primary nonstructural component currently considered in design to resist blast hazards is exterior glazing. A brief discussion of *EDPs* presently used to evaluate glazing hazards is presented in a later Section of this report.

1.2 Reference Documents

A limited number of resource documents and papers were reviewed to generate the list of *EDP*s presented in Section 2. These resource documents are listed in Section 4 of this report. Most of the resource documents have been published by the Federal Emergency Management Agency (FEMA) over the course of the past 7 years, with authorship by the Applied Technology Council (ATC), the Earthquake Engineering Research Center (EERC) at the University of California, Berkeley, and the Building Seismic Safety Council (BSSC).

1.3 Earthquake Engineering Research Centers

The U.S. National Science Foundation (NSF), through its Engineering Research Center (ERC) program, funds research work at three Earthquake Engineering Research Centers (EERCs): MAE (Mid-America Earthquake) Center, MCEER (Multidisciplinary Center for Earthquake Engineering Research), and PEER (Pacific Earthquake Engineering Research) Center. All three Centers have contributed to the development of performance-based earthquake engineering. In particular, important contributions to the state of the performance-based design of individual buildings and structures have been contributed by PEER (methodology, structural component assessment, nonstructural component assessment, and loss estimation) and MCEER (nonstructural component assessment). The MAE center has made contributions related to assessment of the effects of the performance of large systems of buildings and structures on society, known as Consequence Based Engineering. Because the focus of this summary report is identification of *EDP*s for nonstructural components, evaluated on a building-specific basis, the work of both PEER and MCEER are briefly summarized below.

Research work at PEER has provided the technical underpinnings for many components of the ATC-58 project. Moehle (2003) notes that:

> "….PEER aims to develop a robust methodology for performance-based earthquake engineering. To accomplish this objective, the performance assessment and design process has been broken into logical elements that can be studied and resolved in a rigorous and consistent manner. Elements of the process include description, definition, and quantification of earthquake intensity measures, engineering demand parameters, damage measures, and decision variables. A consistent probabilistic framework underpins the methodology so that the inherent uncertainties in earthquake performance assessment can be represented. The methodology can be implemented directly for performance assessment, or can be used as the basis for establishing simpler performance metrics and criteria for performance-based earthquake engineering…."

MCEER's research program on hospitals is also a natural "feed" to the ATC-58 project. As Bruneau (MCEER 2004) notes:

> "…with the objective of enhancing the knowledge in the seismic performance and fragility of nonstructural components and as a supporting effort to the broader integration framework of achieving community resilience, MCEER's hospital project is planning to intensify its experimental studies on the seismic performance and fragility of nonstructural components in acute care facilities in the next three years …"

1.4 Report Organization

This summary report contains three chapters and a bibliography. Chapter 2 forms the body of the report and provides a framework for the presentation of *EDP*s for nonstructural components and systems. A preliminary categorization of the broad types of components is presented in Chapter 3. A list of references and resource documents follows Chapter 3.

2. ENGINEERING DEMAND PARAMETERS

2.1 Framework for Performance-based Earthquake Engineering

Performance-based earthquake engineering seeks to improve seismic risk decision-making through assessment and design methods that have a strong scientific basis and that express options in terms that enable stakeholders to make informed decisions. A key feature is the definition of performance metrics that are relevant to decision making for seismic risk mitigation. The methodology needs to be underpinned by a consistent procedure that characterizes the important seismic hazard and engineering aspects of the problem, and that relates these quantitatively to the defined performance metrics.

The first generation of performance-based earthquake engineering (PBEE-1) assessment and design procedures for buildings in the United States (ATC, 1996; ATC/BSSC, 1997a, 1997b) made important steps toward the implementation of performance-based earthquake engineering. These procedures, developed in the early to mid 1990s, conceptualized the problem that is illustrated in part of Figure 2.1: a building is loaded by earthquake-induced lateral forces that produce nonlinear response (damage) in structural components. Relations were established between structural response indices (interstory drifts, plastic rotation demands, and member forces) and performance-oriented descriptions such as Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). Hamburger (2003) identified several well-accepted shortcomings with these first generation procedures, namely,

1. engineering demands were based on simplified analysis techniques, including static and linear analysis methods; where dynamic or nonlinear methods were used, calibrations between calculated demands and component performance were largely lacking;
2. the defined relations between engineering demands and component performance were based somewhat inconsistently on relations measured in laboratory tests, calculated by analytical models, or assumed on the basis of engineering judgment; consistent approaches based on relevant data are needed to produce reliable outcomes; and
3. structural performance was defined on the basis of component performance states; structural system performance was assumed to be equal to the worst performance calculated for any component in the building.

These shortcomings aside, the PBEE-1 procedures do permit, however imprecisely, the performance evaluation of building structures.

The PBEE-1 procedures address the performance of nonstructural components and systems in a less comprehensive and advanced manner than structural components and systems. Generally, the PBEE-1 procedures identify classes of nonstructural components and systems that are either essential to building occupancy or not. Design procedures primarily focus on providing sufficient anchorage and bracing of these systems and their components so that they do not become dislodged from the structure, slide or topple as a result of ground shaking. Operability or functionality of nonstructural components and systems is not directly considered, though caution is provided that for those systems where operability is critical, either shake table testing or experience data should be used to provide assurance that post-earthquake operability can be attained. No guidance is provided as to how either approach should actually be implemented.

Following the 1994 Northridge earthquake (at the time the PBEE-1 tools were being developed), FEMA funded studies by the SAC Joint Venture[1] on the repair, retrofit and design of steel moment-resisting frames. The component of work on design of new steel moment-resisting frames, although focused on

[1] SAC: a partnership of the Structural Engineers Association of California, the Applied Technology Council, and California Universities for Research in Earthquake Engineering.

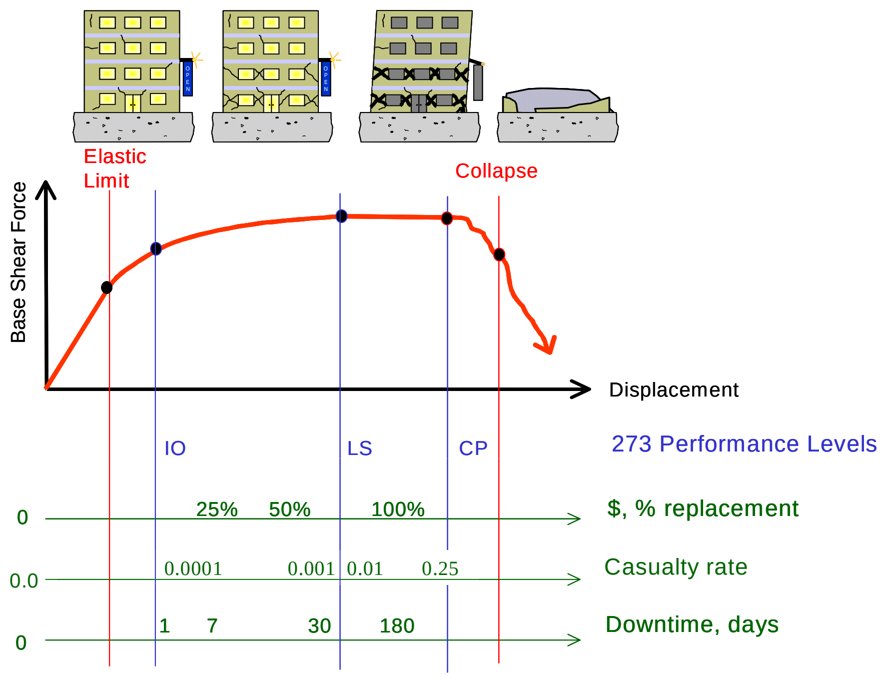

Figure 2.1 Illustration of performance-based earthquake engineering (after Holmes)

improving code-based design procedures (e.g., Yun et al., 2002), took advantage of the PBEE-1 developments and sped the introduction of the probability-based performance assessment tools (Cornell et al., 2002) that form the basis of the second generation of performance-based earthquake engineering assessment and design procedures (PBEE-2) that are described below. However, the SAC Joint Venture work did not consider the performance of nonstructural components and systems.

Although the shortcomings of PBEE-1 listed above were widely recognized by the writers of the first generation of performance-based earthquake engineering documents, limitations in simulation technologies and supporting research precluded further development. In 1997, with funding from the National Science Foundation (NSF), the Pacific Earthquake Engineering Research Center (PEER) embarked on a research and development program to develop a more robust methodology for performance-based earthquake engineering, denoted hereafter as PBEE-2. The PBEE-2 framework developed by PEER facilitates direct calculation of the effects of uncertainty and randomness on each step in the performance-based procedure.

The PBEE-2 process, illustrated in Figure 2.2, begins with the definition of one (or more) ground motion *Intensity Measures* (*IM*s) that should capture the important characteristic(s) of earthquake ground motion that affect the response of the structural framing and nonstructural components and building contents. The *IM* is expressed typically as a function of mean annual probability of exceedance, $p[IM]$, which is specific to the location of the building and its mechanical characteristics (e.g., first and second mode periods). For assessment of the performance of a building's structural system the *IM* is typically either a ground motion parameter, such as peak ground acceleration, peak ground velocity, peak ground displacement or a spectral response quantity such as spectral displacement, velocity or acceleration.

Most nonstructural components and systems, unlike structures, are not directly affected by the ground shaking, but rather are affected by the motion or shaking of the locations in the structure to which they are attached or upon which they are supported. Therefore, for nonstructural components and systems, except those mounted at grade, the *IM* must characterize not the intensity of the ground shaking, but rather the intensity of the response motion of the building structure at the points of attachment of the nonstructural components.

For building structures, the second step of the PBEE-2 process is to determine *Engineering Demand Parameters* (*EDPs*) that describe the response of the structure as a whole and of its individual structural components. This is accomplished by structural response simulations using earthquake ground motions scaled to predetermined *IM* levels. Similarly, for nonstructural components, *Nonstructural Engineering Demand Parameters* ($EDP_N s$) that describe the response of the nonstructural components and contents to earthquake shaking transmitted to them by the supporting structure, must be determined. Many nonstructural components act essentially as rigid bodies and have no response that is distinctly different from the motion of the structure that supports them. For these classes of nonstructural components, *EDPs* that quantify the structural response, e.g. peak interstory drift or peak floor acceleration demands, may be used directly to predict nonstructural performance. However, some nonstructural components have inherent flexibility and will either amplify or modulate the motions transmitted to them by the structure and in the process, will experience motions that are different from those experienced by the supporting points in the structure. For this class of nonstructural components, the second step in the performance assessment process is to select structural *EDPs* calculated from the predicted response of the structure, that predict the severity of shaking the nonstructural components are subjected to. An example of such a structural *EDP* is a floor response spectrum. In essence, these structural *EDPs* serve as *IMs* for the nonstructural components. Then for these flexible nonstructural components, a third step is accomplished by performing structural response simulations of the nonstructural components using the structural *EDPs* as inputs to the nonstructural response calculations. The products of this step are conditional probabilities of experiencing nonstructural component response of different levels, $p[EDP_N/IM]$, which can then be integrated with the $p[IM]$ to calculate mean annual frequencies of exceedance of each nonstructural EDP_N, $p(EDP_N)$.

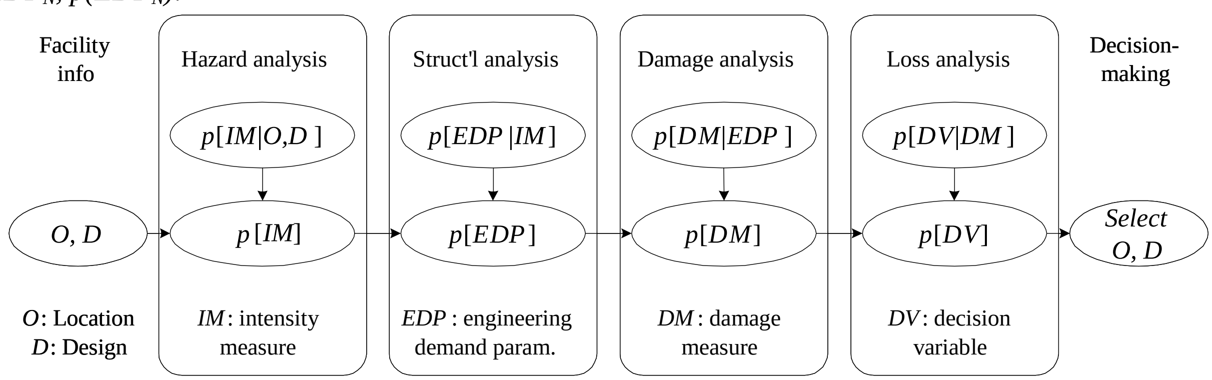

Figure 2.2 Steps in the PBEE-2 procedure (Moehle 2003)

Next, the *EDP*s for the structural and nonstructural components and building contents are linked to *Damage Measures* (*DM*s) that describe the physical condition of those components and contents. *Damage Measures* include *effective* descriptions of damage state or condition, which are then used to estimate the effects on functionality, occupancy-readiness, life safety consequences and necessary repairs of or to the building including nonstructural components and systems. The product of this step are conditional probabilities, $p[DM|\text{EDP}]$, which are then integrated with $p[EDP]$ to calculate the mean annual frequencies of exceedance for the *DM*, $p[DM]$.

The final step in the PBEE-2 process is the calculation of *Decision Variables* (*DV*s) that serve to translate damage estimates into quantities that are useful to those tasked with making risk-related decisions. The *DV*s under development at this time at PEER relate to one or more of the three decision metrics identified in Figure 2.1, namely, direct *dollar* losses, *downtime* (or restoration time), and *deaths* (casualties). The products of this step are conditional probabilities, $p[DV|DM]$, which are then integrated with $p[DM]$ to calculate the mean annual frequencies of exceedance for the *DV*, $p[DV]$.

The PBEE-2 process can be expressed in terms of a triple integral that is an application of the total probability theorem:

$$v(DV) = \iiint G[DV|DM]|dG[DM|EDP]|dG[EDP|IM]|d\lambda[IM] \tag{1}$$

where all terms have been defined previously. It should be noted that for those flexible nonstructural components for which response to structural motion must be computed in order to determine the $EDP_N s$ *(*i.e. the response of structure and nonstructural components are determined to be de-coupled*)*, an additional term is actually required in Eq. 1 to develop nonstructural EDPs from building response. This is given by Eq. 2:

$$v(DV) = \int\iiint G\left[DV|DM\right]dG\left[DM|EDP_N\right]dDM\left[EDP_N|EDP\right]dEDP_N\left[EDP\{IM\right]d\lambda\left[IM\right] \tag{2}$$

Further, the computed value of each decision variable must be summed over all structural and nonstructural components and systems that contribute significantly to the value, considering the potential inter-dependence of the value of projected losses from each component and system on the behavior of other systems and components. This is expressed in Eq. 3.

$$v(DV) = \sum_{systems} \int\iiint G\left[DV|DM\right]dG\left[DM|EDP_N\right]dDM\left[EDP_N|EDP\right]dEDP_N\left[EDP\{IM\right]d\lambda\left[IM\right] \tag{3}$$

Equation 3 provides an effective foundation for the research and design professional community. Moehle (2003) notes that the equation "…provide[s] researchers with a clear illustration of where their discipline-specific contribution fits into the broader scheme of performance-based earthquake engineering and how their individual research results need to be presented….[and]…emphasizes the inherent uncertainties in all phases of the problem and provides a consistent format for sharing and integrating data and models developed by researchers in the various disciplines."

The process described by Eqs. 1, 2 and 3, and Figure 2.2 represents the detailed assessment of a *building*, where the building is defined in terms of all structural and nonstructural components and systems and contents. Evaluation of $EDP_N s$ is an intermediate (and not final) step in performance evaluation in this PBEE-2 framework, where much emphasis is also placed on evaluating *DM*s and *DV*s.[2]

2.2 Review of Engineering Demand Parameters Concepts

The following sections of this chapter focus on one component of the summed integral of Eq. 3, namely, EDP_Ns. Current prescriptive seismic design procedures and 1st generation performance-based design procedures use a rather limited set of $EDP_N s$ to characterize the performance of nonstructural components. However, design professionals working in industries where the reliability of the performance of nonstructural components and systems is critical, such as the nuclear power industry, have developed design procedures that consider a somewhat more diverse set of $EDP_N s$. In addition, a number of individual researchers and research projects have explored alternative $EDP_N s$ that could be used to improve performance prediction and reliability. In the development of this report, contemporary seismic design codes, design guidelines and design procedures and some archival literature were reviewed to identify those EDP_Ns in use at the time of this writing or suggested for use in the future.

In a companion document to this report (Whittaker, et al., 2004) *EDP*s useful in characterizing structural performance were categorized by the authors as either *direct* or *processed*. Direct *EDP*s are those *EDP*s calculated directly by analysis or simulation and contribute to Equation 3 through $p[EDP|IM]$. For

[2] In the PBEE-1 procedures of FEMA 273, *Guidelines for the Seismic Rehabilitation of Buildings* (ATC/BSSC, 1997a) and current design provisions and building codes such as the *2000 FEMA 368 NEHRP Recommended Provisions for New Buildings* (BSSC, 2000a), *EDP*s play a more central and significant role because evaluation of *EDP*s is the final step in the assessment or design process.

example, for evaluation of structural framing, useful direct *EDPs* include interstory drift and beam plastic rotation. Processed *EDP*s (for example, a damage index) are derived from values of direct *EDP*s and data on component or system capacities. Processed *EDP*s could be considered either *EDPs* or as *Damage Measures* (*DM*s) and as such, could contribute to Equation 3 through $p[DM|EDP]$. While the processed *EDPs* (damage indexes) described in the document by Whittaker, et al. (2004) were developed for building structural framing, the indexes could be applied to some types of nonstructural components. However, since the processed *EDPs* (damage indexes) described in the companion document have not been typically applied to nonstructural components, they will not be included in this report and the reader is therefore referred to the companion document for further discussion regarding them.

2.3 Traditional Nonstructural Engineering Demand Parameters

2.3.1 Building Code Nonstructural EDPs

Traditional EDP_Ns have typically been limited to component forces and for some limited cases, interstory relative displacements (drifts). These EDP_N*s* form the basis for design provisions contained in all contemporary and earlier building codes, as well as the PBEE-1 procedures. Component forces (demands) are determined by applying a lateral load to the center of mass of the component and computing the forces in the bracing and attachments. Except in the case of a limited number of nonstructural components, such as steel storage racks, the nonstructural component is treated as a "black box" and the load path through the component, and the component's adequacy to transmit this load, may not ever be evaluated.

Prior to 1997, the building codes (e.g., ICBO, 1994) calculated the design loading, F_p, on nonstructural components through a simple formulation:

$$F_p = ZIC_pW_p \tag{4}$$

where, Z is the seismic zone coefficient, I was an importance factor, C_p was a component response factor and W_p the component weight. As can be seen, the calculation of *EDPs* was only indirectly related to intensity of design motion, through the seismic zone and importance factor, and did not consider such important characteristics of ground shaking as proximity to nearby faults or site soil conditions. These coefficients also did not consider the response characteristics of the supporting structure or location of the nonstructural component within the structure. The effect on response caused by the fundamental period of the building, or the nonstructural component, was typically ignored, except that certain types of components, such as cantilevered parapets, were assigned larger C_p coefficients than other elements. It is not clear if these larger C_p factors were assigned based on expectation of larger dynamic response of these components, on the basis of anticipated fragility of the component, or a combination of these factors. EDP_N*s* were most commonly calculated using equivalent static lateral forces. Component demands (forces) were combined with forces resulting from other loads, including dead and operating, and checked against component-specific permissible values.

Some nonstructural items, such as cladding, were specifically designed using interstory drift as the EDP_N, rather than or in addition to a design force. Typically, the EDP_N was determined based on the maximum drifts permitted for the structural system or an arbitrarily amplified value of this permissible drift, and not on the actual computed drift for the specific structure under design loading. Thus, this EDP_N was often even less related to the *IM* or structural response, than were the force-based EDP_N*s*. Internal member forces caused by or imposed by interstory drifts were added to the forces resulting from other loadings when drift was a consideration.

For steel members, connections and attachments, allowable values for the inertial and drift-related forces were determined using the *AISC Manual of Steel Construction, Allowable Stress Design* (AISC, 1989). Allowable forces for post installed anchor bolts were typically based on ICBO Evaluation Service (ES) Reports. In some cases, prescriptive industry standards regulating the type and size of bracing and attachments, such as *NFPA-13* (NFPA, 2000) for sprinkler systems, were deemed to comply with building code loadings, and no formal determination of design forces was made. In fact, for most buildings (with hospitals in California being the rare exception) nonstructural components, bracing and attachments are not normally designed for seismic loads (even though such design is specified in the code) and if design is done it is rarely executed properly by subcontractors during construction.

Codes and guidelines commonly used in the building industry and published in 1997 (FEMA 273 [ATC/BSSC, 1997a], FEMA 274 [ATC/BSSC, 1997b], ICBO, 1997) and later years have evolved significantly. These later building codes continue to use inertial forces calculated using equivalent lateral force coefficients and interstory drift as the primary EDP_N*s*. However, the determination of these EDP_N*s* has become more sophisticated and somewhat more representative of actual demands. In these later procedures, site soil conditions, location-specific site ground motions and location of the component within the structure are all considered in the determination of the design coefficients and forces. In addition new terms have been added to the equation used to compute EDP_N*s* to account for amplification of response based on the period of the component and anticipated inelastic response capacity of the component. The formulation of this newer procedure for calculating EDP_N*s* as contained in (ICBO, 1997) is:

$$F_p = \frac{a_p C_a I_p}{R_p}(1 + 3\frac{h_x}{h_r})W_p \tag{5}$$

where, a_p is a factor relating to dynamic amplification of input motion based on the component's dynamic characteristics, C_a is an intensity measure dependent on site location and site geologic characteristics, I_p is an importance factor, R_p is a component-specific inelastic response coefficient, the term h_x/h_r is a measure of the location of the component vertically within the structure, and W_p is as previously defined. Some recent contemporary codes and guidelines use a slightly different formulation; however, this basic approach to seismic design is still used in the latest codes for new building construction. Guidelines for new building construction are presented in the FEMA 368/369 *NEHRP Recommended Provisions for New Buildings* (BSSC, 2000a, 2000b), which have been adopted in the large part into ASCE-7-02, *Minimum Design Loadings for Buildings and Other Structures* (ASCE, 2002). It should be noted that ASCE 7-02 requires more explicit consideration of relative displacement effects for deformation-sensitive components, but except for the case of glazing, provides no guidance on how to determine allowable values for relative displacement load combinations.

2.3.2 Building Code Seismic Qualification of Nonstructural Components

Borrowing from practice in the nuclear industry and other industries with critical safety concerns, ASCE 7-02 requires that active mechanical components that are necessary for operation after an earthquake be certified to satisfy this performance capability through shake table testing or the use of experience data. Manufacturers of nonstructural equipment — using various interpretations of model building code requirements — have pursued seismic qualification testing of nonstructural components for many years. Shake-table testing is the preferred industry approach for qualifying nonstructural equipment to meet the requirements contained in model building codes. However, neither ASCE 7-02 nor building code provisions define nor offer any guidance on how to correctly translate static lateral-force requirements into appropriate input motions for use in shake-table testing. This situation results in multiple code interpretations with individual manufacturers claiming seismic qualification on the basis of very different testing procedures and severity levels. Resolution of this inconsistency in code interpretation regarding qualification testing has been addressed, at least in part, by a generic AC156 test procedure that has been recently adopted by the International Code Council (ICC) Evaluation Service organization (ICC ES, 2000b). The *AC156* document provides shake table testing protocols specifically intended for seismic qualification testing of nonstructural components. The document includes the definition of the test floor response spectra, the characteristics of the test motion, requirements for tested units and acceptance criteria for seismic qualification. The document can be used to satisfy the requirements to demonstrate postearthquake operability capability for nonstructural components under any model building code or standard that adopts the FEMA 368 *NEHRP Recommended Provisions for New Buildings* as the primary source document (for example, 2000 IBC or 2003 IBC, 1997 UBC, 2002 ASCE-7, 2003 NFPA 5000 and others). The development of the seismic qualification protocol is based upon the existing nonstructural lateral force procedure in conjunction with the building design response spectrum. This approach accounts for above grade-level equipment installations, with or without knowledge of the building's dynamic characteristics. A well-defined pass/fail acceptance criterion is established that utilizes the equipment importance factor to define post-test acceptability. In essence, this generic test protocol establishes seismic qualification shake-table test motions that can be used to qualify any nonstructural component for any given equipment location in a building and for any given building location in the United States.

2.3.3 Current Performance Levels of Building Codes and Standards

The lateral force coefficients used in nonstructural component evaluations and simulations are established for a design basis earthquake for which the intended performance in most buildings, albeit not explicitly checked, is Life Safety. This performance level and its corresponding damage state (in cartoon form) were illustrated previously in Figure 2.1, together with other performance levels. In the figure of the damaged building, the marquee, windows and lights represent nonstructural components. The Life Safety (LS) performance point is shown as LS in the figure. The corresponding damage cartoon is closest to the second of the four cartoons in the figure although typically the lights are not expected to be functional for the LS performance level. General statements regarding the assumed levels of damage to nonstructural components at the LS level are provided in Table 2.1 below in bolded text. Information in this table is drawn from multiple sources, including Comartin (2003).

Table 2.1 Building performance levels per 2000 NEHRP and FEMA 273/274/356 (ASCE, 2002)

Performance level	*Damage description*	*Downtime/Loss*
Immediate Occupancy	Negligible structural damage; essential systems operational; minor overall damage	24 hours
Life safety	**Probable structural and nonstructural damage; no collapse; minimal falling hazards; adequate emergency egress**	Possible total loss
Collapse Prevention	Severe structural and nonstructural damage; incipient collapse; probable falling hazards; possible restricted access	Probable total loss

2.3.4 First Generation EDP_Ns for Performance-based Earthquake Engineering

As indicated above, funding from the Federal Emergency Management Agency (FEMA) to the Applied Technology Council (ATC), American Society of Civil Engineers (ASCE) and the Building Seismic Safety Council (BSSC) in the early to mid-1990s led to the development of the FEMA 273 *NEHRP Guidelines* and FEMA 274 *Commentary for Seismic Rehabilitation of Buildings* (ATC/BSSC, 1997a, b). This development effort (referred to as PBEE-1 above) marked a major milestone in the evolution of performance-based seismic design procedures and articulated several important earthquake-related concepts essential to a performance-based procedure.

The key concept in PBEE-1 was that of a performance objective, consisting of the specification of the design event (earthquake hazard), which the building is to be designed to resist, and a permissible level of damage (performance level) given that the design event is experienced. Other important features of the FEMA 273 *Guidelines* were the introduction of (a) standard performance levels, which characterized in a general manner, levels of structural and nonstructural damage based on values of standard structural response parameters, and (b) nonlinear methods of analysis and performance assessment for building structural frames. Figure 2.1, presented previously, illustrates the qualitative *performance levels* of FEMA 273/274 (IO = Immediate Occupancy; LS = Life Safety; CP = Collapse Prevention) superimposed on a global force-displacement relationship for a sample building. The cartoons in the figure show the corresponding levels of damage from the onset of structural response up to the point of collapse. Brief descriptions of the building damage and business interruption (downtime) for the three FEMA 273/274 performance levels are given in Table 2.1. For nonstructural components, levels of nonstructural damage were much less quantified than for building structures and analysis approaches tended to follow equivalent static approaches found in the current FEMA 268/269 NEHRP *Recommended Provisions for New Buildings* (BSSC, 2000a, BSSC, 2000b) where the calculated elastic response forces are divided by a force reduction factor R_p to account for inelastic capacity of the component. It should be noted that although the linkage between damage state and the value of key decision variables such as life loss, repair costs and occupancy interruption time were qualitatively considered in the development of FEMA-273/274 documents, no direct relationship between the damage states and these decision variables was presented in the documents.

One of the valuable concepts presented in FEMA 273/274 was the categorization of nonstructural components into two classes of sensitivity. Nonstructural components that were considered primarily sensitive to and subject to damage from inertial loading were classified as *acceleration-sensitive* components. Nonstructural components that were considered primarily sensitive to deformation imposed by interstory drifts of the structure were classified as *deformation-sensitive* components. Components that were considered sensitive to both inertial loading and interstory drifts were also classified with the more sensitive effect denoted as Primary (P) and the less significant effect denoted as Secondary (S). Table 2.2 summarizes inertial and/or deformation sensitivity of selected nonstructural components as identified in FEMA 273/274.

2.3.5 Discussion Regarding Traditional Code Nonstructural Engineering Demand Parameters

As noted earlier, current codes and guidelines utilize arbitrarily reduced, equivalent static forces and internal member forces resulting from imposed interstory drifts as the primary EDP_N*s.* This is true for both code-based design and PBEE-1 performance evaluation of nonstructural components. In code-based design and PBEE-1 evaluations, nonstructural component design forces are calculated using indirect and imprecise procedures and empirical relationships. Interstory drifts are typically not computed but rather the maximum allowable drift levels are generally assumed. The resulting force-based EDP_Ns that are typically used for nonstructural component checking and system performance evaluation are judged by indirect measures of component behavior.

While more advanced and rational techniques for evaluating building structures were provided with the development of PBEE-1, advances in the area of nonstructural components was minimal. This lack of progress is surprising since earthquake damage associated with nonstructural components constitutes well over 50% of the total losses in recent earthquakes in the United States. There are many shortcomings with the EDP_N*s* currently used for design and evaluation of nonstructural components. These shortcomings include:

1. The design forces for nonstructural components are based on indirect and unproven procedures. The reduced equivalent static values provided in codes and in FEMA 273/274 are based on judgment and it is not known if they correlate well with actual performance.
2. Procedures to evaluate the adequacy of nonstructural components subjected to specified relative displacements are not provided. Therefore, there is no current way, except in the case of glazing, to determine whether a design is acceptable for specified imposed displacements other than requiring elastic performance, which may be very conservative and counterproductive.

Table 2.2 Response Sensitivity of Selected Non Structural Components from FEMA 274 (ATC/BSSC, 1997b)

		Sensitivity	
	COMPONENT	Acc.	Def.
A. ARCHITECTURAL			
1.	**Exterior Skin**		
	Adhered Veneer	S	P
	Anchored Veneer	S	P
	Glass Blocks	S	P
	Prefabricated Panels	S	P
	Glazing Systems	S	P
2.	**Partitions**		
	Heavy	S	P
	Light	S	P
3.	**Interior Veneers**		
	Stone, Including Marble	S	P
	Ceramic Tile	S	P
4.	**Ceilings**		
	a. Directly Applied to Structure	P	
	b. Dropped, Furred, Gypsum Board	P	
	c. Suspended Lath and Plaster	S	P
	d. Suspended Integrated Ceiling	S	P
5.	**Parapets and Appendages**	P	
6.	**Canopies and Marquees**	P	
7.	**Chimneys and Stacks**	P	
8.	**Stairs**	P	S

		Sensitivity	
	COMPONENT	Acc.	Def.
B. MECHANICAL EQUIPMENT			
1.	**Mechanical Equipment**		
	Boilers and Furnaces	P	
	General Mfg. and Process Machinery	P	
	HVAC Equipment, Vibration Isolated	P	
	HVAC Equipment. Nonvibration Isolated	P	
	HVAC Equipment, Mounted In-line with Ductwork	P	
2.	**Storage Vessels and Water Heaters**		
	Structurally Supported Vessels (Category 1)	P	
	Flat Bottom Vessels (Category 2)	P	
3.	**Pressure Piping**	P	S
4.	**Fire Suppression Piping**	P	S
5.	**Fluid Piping, not Fire Suppression**		
	Hazardous Materials	P	S
	Nonhazardous Materials	P	S
6.	**Ductwork**	P	S

Acc.=Acceleration-Sensitive P = Primary Response
Def.=Deformation-Sensitive S = Secondary Response

3. Current EDP_Ns for nonstructural components are not directly linked to the nonlinear dynamic response of the actual building in which the components are located.
4. Current EDP_Ns for nonstructural components are not directly linked to the nonlinear dynamic response of the actual component itself.
5. The use of reduced lateral force coefficients masks the fact that that the real demand on nonstructural components is floor accelerations and associated displacements, not forces.
6. For some nonstructural components, the EDP_Ns are not truly understood and rarely calculated. For example, in a piping system, the EDP_N most likely to be correlated with damage is the maximum plastic rotation of individual piping connections. However, such rotation demands are rarely calculated and even if they were, there is little data to indicate the tolerable levels of these rotations to achieve various performance states.
7. There are several orders of magnitude more types of nonstructural components and systems than building structural systems, which makes the development of specific performance evaluation procedures less tractable.
8. There is no direct way to evaluate the reliability of the performance of a given nonstructural component in a given structure.

Many of these topics are being tackled at this time through research at the NSF-funded PEER and MCEERCenters (www.peer.berkeley.edu and www.mceer.buffalo.edu).

2.4 Identification of Next-Generation Nonstructural Engineering Demand Parameters

2.4.1 Next Generation EDP_N Criteria for Selection

An important criteria in the selection of EDP_N*s* for next-generation performance-based engineering guidelines, is that there should be significant correlation between the EDP_N and damage states that are significant to the value of decision variables. It is also desirable that the EDP_N*s* be both useful and efficient. For an EDP_N to be useful, it must be compatible with the structural analysis or testing protocol that is used to evaluate the nonstructural component response. An EDP_N is efficient if the variability associated with prediction of response and damage tends to be small.

Based on the above discussion, there are two basic classes of EDP_N*s*. The first class of EDP_N is associated with building response motions such as interstory drift and peak floor acceleration. A second class of EDP_N*s* are those associated with calculated secondary response parameters. For example, a significant EDP_N is likely to be the inelastic rotation of a pipe joint where the input to the pipe stress analysis is floor spectra and relative displacements of the floors to which the pipe is attached.

One basic assumption in the selection criteria is that the weight and stiffness of the nonstructural component is small relative to the weight and global stiffness of the building such that behavior and response of the nonstructural component have a negligible influence on the dynamic response of the building. This assumption is valid for most nonstructural components and systems but does not hold in some cases. It is clear that simulation procedures used to predict *EDPs* and EDP_N*s* will need to account for the stiffness and mass of nonstructural components where these are significant to overall structural response.

2.4.2 Process Used for Identification of EDP_Ns

To identify the EDP_N*s* that should be considered for use in the next-generation procedures, a two step process was used. In the first step, an e-mail survey was conducted of persons who attended the February 2003 ATC-58 Project Program Design Workshop (ATC, 2003a) in San Francisco soliciting their recommendations. Survey responses were received from the following researchers and engineers:

Andre Filiatrault, MCEER, University at Buffalo

Ahmad Itani, University of Nevada, Reno

Eduardo Miranda, Stanford

Praveen Malholtra, Factory Mutual Global

Ali Memari, Penn State

Gary McGavin, AIA, Cal Poly Pomona

Robert Kennedy, RPK Structural Mechanics

As the second step, an informal literature search was conducted that included relevant guidelines, conference and seminar proceedings. The documents of most relevance were Appendix G and I of the SEAOC Bluebook (SEAOC 1999), FEMA 356 (ASCE, 2000) and the proceedings of three seminars on nonstructural components in the ATC-29 series (ATC, 1992b; ATC, 1998; ATC, 2003b).

The results of these searches and surveys were quite consistent. The EDP_N*s* identified were either associated with equivalent force coefficients, floor dynamic motions, or relative displacements.

2.4.3 Identified EDP_Ns

Table 2.3 summarizes the $EDP_N s$ that have been identified from the searches and survey for potential use in the next-generation guidelines.

Table 2.3 – Suggested EDP_Ns for Next-generation Procedures

Class 1 – EDP_Ns directly linked to building response (EDPs serve as intensity measures)	
Inertial Force Sensitive Components	Peak floor accelerations[1]
	Peak floor spectra response accelerations[1]
	Floor spectra response acceleration at period of component[1]
	Peak absolute floor velocity[1]
	Peak floor response spectra velocity[1]
	Floor response spectra displacement at period of component[1]
	Cumulative absolute velocity parameter[2]
Relative Displacement Sensitive Components	Peak interstory relative displacements[3]
	Peak horizontal relative displacements across seismic joints or isolation planes
Class 2 – EDP_Ns not directly linked to building response (EDP_Ns determined by structural analysis of component, using building response motion as input)	
	Inelastic rotation or deformation in component
	Axial, flexural and shear force in attachments
	Peak stresses

1- 3 axes, including 2 horizontal axes, and vertical axis considered
2- See Section 2.4.4
3- Two horizontal directions considered

2.4.4 Cumulative Absolute Velocity Parameter

With the exception of the Cumulative Absolute Parameter, the $EDP_N s$ listed in Table 2.3 are generally well known. The Cumulative Absolute Velocity (CAV) was originally proposed by the Electric Power Research Institute (EPRI) in EPRI NP-5930 (EPRI, 1988) as a parameter for determining the damage threshold for engineered structures and anchored industrial grade equipment subjected to earthquake ground motion. Originally, the CAV was defined by:

$$\mathrm{CAV} = \int_0^{t_{\max}} \left|a(t)\right| d(t) \tag{6}$$

where:

$a(t)$ = acceleration time history

$t_{\max}$ = duration of record

With this definition, the CAV damage threshold was set at:

$$\text{CAV} \leq 0.30\text{g-sec (116 inch/sec)} \tag{7}$$

which corresponds to the highest CAV for which Modified Mercalli Intensity MMI VII damage has never been reported.

From Equation 1 CAV can be seen to be the sum of the consecutive peak-to-valley distances in the velocity time history. Another interpretation of the CAV is as the area under the acceleration versus duration curve. In this way the CAV is a measure both of the amplitude and the duration of motion. The CAV was subsequently refined in EPRI TR-100082 (EPRI, 1991).

CAV was developed to serve as a conservative threshold on the potential for damaging engineered structures and anchored industrial grade equipment. This CAV threshold is about a factor of five lower than the lowest CAV value associated with documented damage to an industrial/power facility. It is about a factor of three lower than the lowest CAV values associated with documented damage to buildings of good design and construction.

The CAV threshold has been used to determine whether after an earthquake has affected a facility, detailed investigations should be performed to assess whether any damage has occurred. It has not been used as a predictive measure of the amount of damage or severity of damage that may occur, other than in this sense.

It is currently understood that CAV has only been computed for ground motions. However, it could also be computed for floor-motion response histories. It is not clear as to how this quantity could be used to predict damage as no studies have yet been performed to determine if such calibration occurs. The developer of CAV, Robert Kennedy, does not recommend the use of this parameter as an engineering demand parameter for architectural, mechanical, and electrical equipment without extensive further study.

2.4.5 Discussion of EDP_Ns

In addition to the peak displacements and accelerations captured by the EDP_Ns described above, another important factor that can affect the performance of a nonstructural component is the duration of shaking or the number of cycles of significant demand. Other than in the case of *CAV*, and damage indices, duration is not captured in EDP_Ns in current use, but is a factor that should be considered in the adoption of EDP_Ns for the next generation guidelines. The consideration of which EDP_Ns to adopt is dependent on several factors including how the component is designed or tested.

Fragility is a relationship between the value of an EDP_N and the probability that damage of certain types is experienced or exceeded. The fragility of many types of nonstructural components may be determined by shake table testing. Table motions used to determine fragility should emulate the floor motions found in actual buildings in which the components will be mounted. This means that table motion itself could be an intensity measure. It should be noted that because building-response floor motions can be several times larger than ground motions, many contemporary shake table facilities may not be able to generate sufficient motion to replicate conditions experienced in real buildings. To illustrate this point, Figure 2.3 presents a plot of a 5 % damped floor response spectra derived from response motions recorded at the roof of a a 6-story hospital building during the Northridge earthquake. Also shown in the figure is the corresponding ground motion response spectra derived from a field instrument present at the same site.

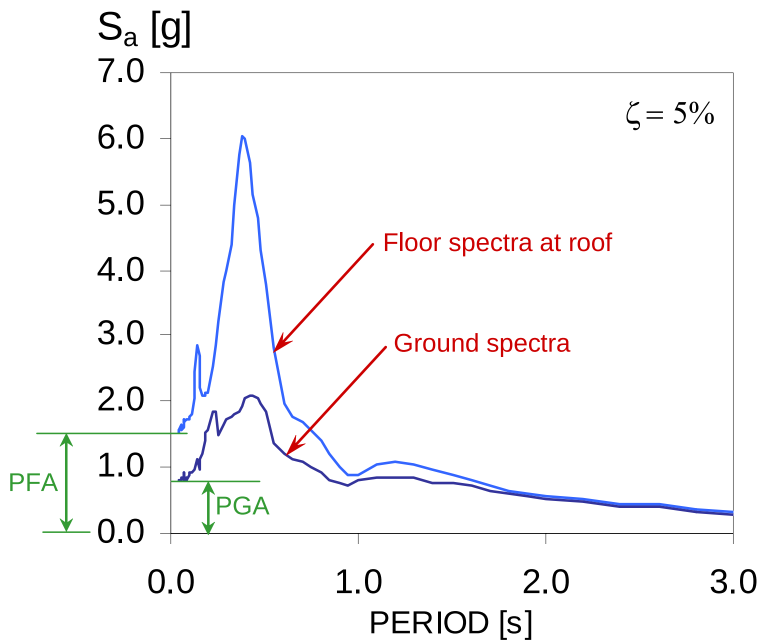

Fig. 2.3 Response spectra and floor spectra computed from motions recorded at a 6-story hospital in Sylmar, California during the 1994 Northridge earthquake.

The performance of most nonstructural component types can be found to be adequately predicted by one or more of the EDP_Ns described in Section 2.4.3. For example components that are inherently rugged and rigid and attached at only one horizontal plane in a structure are most sensitive to peak floor accelerations. Performance of such components can best be predicted by this EDP_N. Cladding, glazing and partitions are most sensitive to interstory displacements in plane, while out of plane, they are most sensitive to peak floor acceleration. Some flexible components such as motor control centers or vibration isolated equipment are probably most sensitive to peak spectral acceleration. Others, with well defined fundamental periods may be most sensitive to the spectral acceleration at the fundamental period of the item under consideration. The performance of distributed systems such as piping is probably predicted best by calculated EDP_Ns such as peak inelastic rotations at a connection or peak stress in an elbow. It is expected that the performance of unanchored contents is sensitive to a complex combinations of several EDP_Ns including acceleration and velocity in both horizontal and vertical directions.

2.4.6 Available Sources of Testing Motions and Protocols

Table 2.4 summarizes the sources of test and qualification criteria and protocols currently in use for determining the fragility of nonstructural components, as revealed in the survey and literature search previously described. It is expected that many other test motions and protocols for nonstructural components are available.

2.4.7 Other Noteworthy References

Filiatrault and Chrisopolous (2002) presents a rich list of references on almost all subject areas in nonstructural systems related research conducted in the past. Chapters 3 and 5 of the report present the performance of nonstructural building components during past earthquakes and briefly summarize past analytical and experimental work.

Table 2.4 Standard Motions and Protocols for Fragility Evaluation of Nonstructural Components

Protocol/Criteria Type	*Description*
Floor Response Histories	A floor acceleration database for 3- and 6-story reinforced concrete buildings has been generated for Eastern and Western Canada and is available at the following web site: http://www.struc.polymtl.ca/pwgsc/
Floor Test Spectra and Testing Protocols	AC-156, *Acceptance Criteria for Seismic Qualification Testing of Nonstructural Components*, (ICC, 2000a).This document provides procedures for developing generic floor test spectra and protocols that are derived from current code requirements.
	"Network equipment-building system (NEBS) requirements: Physical protection," *Generic Requirements, GR-63-CORE*, (BellCore 1995). This document provides generic spectra and testing procedures intended for telecommunication equipment.
Component Cyclic Loading	FM Global Procedure found in the following paper "Testing Sprinkler-Pipe Seismic-Brace Components," (Malhotra et al., 2003). Given a design force level, this paper develops a procedure to determine the number of test cycles needed to establish a component's capacity.
	ATC-24, *Guidelines for Cyclic Testing of Components of Steel Structures*, (ATC, 1992a). This document, while intended specifically for testing steel connections, has been used for a variety of component testing, including testing of nonstructural components.
Interstory Drift Racking Tests	AMMA 501.6-01, *Recommended Dynamic Test Method for Determining the Seismic Drift Causing Glass Fallout from a Wall System*, (AMMA, 2001) American Architectural Manufacturers Association, Schaumburg, Il. This document provides a dynamic racking crescendo testing procedure intended for glass panels but is also usable for other drift sensitive components and systems.

Another important source of information is the MCEER Database for nonstructural damage that can be downloaded from the web site: https://mceer.buffalo.edu/publications/reports/docs/99-0014/default.asp. This is a unique database that provides damage information on nonstructural components collected from earthquakes as early as the Alaska Earthquake of 1964 to the present. Information in this database has been gathered from various available publications, such as books, reports, and periodicals. This is information not easily retrieved, as most earthquake reconnaissance reports and other publications concentrate on structural or geologic effects.

2.4.8 Nonstructural Engineering Demand Parameters for Blast Engineering

Performance-based design for blast engineering is in its infancy. Currently, the only nonstructural component that is routinely designed for blast resistance, and for which standardized design procedures have been developed, is glazing. However, the design of glazing systems for blast resistance follows a performance-based procedure that is quite similar to first-generation performance-based seismic design approaches. A brief description of this procedure is presented below, followed by a discussion of engineering demand parameters used in the process.

The primary concern in performance-based design for blast hazards is protection of personnel safety. Personnel safety can be jeopardized by glazing response to blast loading when the glazing or its supports fail, and either entire panels of glazing or shards from the fractured panel are propelled at high velocity into an occupied space. Rather than calculating the probability of life endangerment, resulting from glazing response to blast, in current performance-based design procedures, a series of discrete performance levels are identified, some of which are considered acceptable and some unacceptable with regard to protecting life safety, based on qualitative consideration of the potential risk to personnel, should the damage state occur. These include the following:

a) Glazing panel and its supports are undamaged

b) Glazing panel is fractured; however, the panel remains within its support frame

c) Glazing panel is fractured and ejected from its frame; however, it is not propelled a sufficient distance into occupied space to present a significant hazard

d) Glazing panel is fractured or supports fail and portions of the glazing or the entire panel are propelled into occupied space

Typically, just as with first-generation performance-based seismic design approaches, performance objectives are stated by coupling a specific blast event with a desired performance level. Blast events may be quantified either as the detonation of a specific charge of explosive material, typically expressed in equivalent tons of TNT at a specified location relative to the glazing, or in terms of a specific blast impulse pressure wave that would be generated by such a detonation. In some cases, a standard blast impulse pressure wave is assumed without direct association with a specific explosive charge or distance. The performance objective is to not exceed one of the damage states described above, should the design detonation or blast impulse wave be experienced.

For purposes of design for blast resistance, glazing is treated as a structural element and is subjected to nonlinear dynamic analysis for its response to the design blast impulse wave. The glazing is typically modeled as a flat plate, or a laminated assemblage of plates, depending on the glazing type, with consideration given to the support condition. Nonlinear finite element methods are used to calculate the distribution of flexural and shear stresses and strains in the glazing and should the glazing fracture, the distance that fragments are propelled beyond the frame into occupied space. The damage state or performance level is then determined by evaluating the values of the predicted *EDPs*. For this analysis, the following *EDP's* are used:

- Peak flexural stress in the glazing
- Peak shear stress in glazing
- Peak flexural strain in glazing
- Curvature of glazing panel
- Velocity of glazing pieces, should fracture occur

On the basis of the above *EDP's*, determination is made as to whether a specific glazing design is capable of achieving the various performance levels. Specialized software, for example, WINGARD Limited Edition (ARA, 2004) has been developed to perform these analyses in an efficient manner and the predicted results from this software have been calibrated against tests in which glazing panels of different designs have been subjected to blast-impulse waves.

3. PRELIMINARY CATEGORIZATION OF SIGNIFICANT NONSTRUCTURAL COMPONENTS AND SYSTEMS

As noted earlier, nonstructural components include all items attached to or contained within a building other than the primary structural system. In a typical building there are countless types of nonstructural systems and components and it would be impractical to develop a performance prediction methodology that explicitly considers all the components that exist in any one building, let alone the entire inventory of buildings that must be addressed. However, it should be possible to identify certain components and systems that have particularly important and significant consequences with regard to the critical decision variables (life loss or serious injury, repair costs and downtime) and to categorize them into several broad groups that have similar performance characteristics and engineering demand parameters. Similarly, it should be possible to identify components that have a lesser impact and to similarly categorize them. Appropriate EDP_N's will then be developed for each of these broad categories.

On a preliminary basis, the following general broad categories for nonstructural systems and components have been selected. Components will initially be categorized, based on whether earthquake damage could result in:

1. leaks,
2. fire ignition,
3. prevention of safe building occupancy,
4. significant repair costs,
5. serious falling hazards,
6. prevention of critical functionality, or
7. serious business losses.

These broad categories will be subdivided into EDP_N sensitivity. For example, the following five categories of EDP_N*s* could be identified.

1. relative displacement between floors (drift)
2. peak floor acceleration
3. floor spectra acceleration ordinate at fundamental period of component
4. peak velocity of floor
5. peak stress in an individual component (e.g. tank or pipe)

It is planned to identify and categorize individual components and systems that have significant potential impact on building performance into one of the above categories and subcategories. For example, a drywall partition might be categorized as something that results in significant repair cost and is subcategorized as having an EDP_N of drift. An uninteruptable power supply system might be categorized as having critical impact on postearthquake functionality and be subcategorized as having an EDP_N of peak spectral acceleration at the fundamental period of the component. It is intended to perform a comprehensive and systematic categorization of all components within the system judged to be significant to either to life loss or injury, repair cost, or downtime. Remaining components would not be categorized as individual contributors to building performance, but instead lumped into a few general categories. At this point, the term "bin" is being used as an identifier for the broad categories. A component would be categorized into only one bin.

During the next phase of the project, generalized fragility functions will be developed for each bin identified in the previous task. The fragility functions would utilize the $EDP_N(s)$ identified for the bins. The fragility functions would initially be developed based on available resources and expert opinion.

Nonstructural fragilities are functions that relate the probability that a nonstructural component will experience or exceed a certain level of damage, given that it is driven to a certain level of response, as measured by the EDP_N. As is the case with building response functions, fragilities are expressed as probability distributions, rather than deterministic relationships in order to account for the variability and uncertainty inherent in the process of predicting nonstructural damage as a function of nonstructural response. The variability is associated with such factors as the random character of the primary structural and associated nonstructural response to individual ground motion records, and the inability of simple engineering demand parameters to distinguish between this response variation and the damage it causes. For example, two different ground motions may each produce peak interstory drift demands of 4 inches in a structure; however, one of these ground motions may cycle the structure to this drift level one time, then restore the structure to small oscillations about its original position, while the second ground motion may cycle the structure to this drift level several times and leave the structure displaced nearly to this level. Clearly, the latter motion will be more damaging to the structure than the first motion, though the value of the engineering demand parameter is the same. Such effects are not predictable unless the precise ground motion and structural response are known. Uncertainty is introduced through such factors as lack of precise definition of material strength and construction quality.

In order to form fragility functions, it is first necessary to define damage states. A variety of such damage states or measures of damage are possible. Damage states that may be meaningful for nonstructural components and systems could include “no damage,” “leakage,” “loss of function,” “loss of structural integrity” and “toppling”. In general, each category of nonstructural component or system will have different fragility functions, perhaps tied to several different EDP_Ns.

While initially the fragility functions for the broad categories will be established by expert opinion, over time they can be determined more rigorously through collection of earthquake performance data on damage sustained by actual installations, through laboratory testing programs and in some cases, through structural analysis, just as would be done for the building structure itself. For critical equipment that must function, the fragility data may come from seismic qualification testing. However, it should be noted that the purpose of qualification testing is to demonstrate that a component is able to survive a certain test and failure is not usually observed. Fragility functions, on the other hand, require that the various damages states be observed during the test, so standard qualification testing does not usually provide enough information to fully develop fragility functions. Typically, the fragility level is associated with some mean design level. For example, under systems that have high repair cost, there may be a subcategory of components that are sensitive to peak floor accelerations. A component that is designed for twice the force might have a fragility that is twice as high.

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