PERFORMANCE-BASED ENGINEERING OF BUILDINGS FOR EXTREME EVENTS

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ABSTRACT

Blast, earthquakes, fire and hurricanes are extreme events for building construction and warrant innovative structural engineering solutions. The state-of-the-practice and new developments in performance-based earthquake engineering (PBEE) are discussed, with emphasis on hazard intensity measures, engineering demand parameters, and performance levels. The new performance-based earthquake engineering methodology is extended to performance-based blast engineering. Sample intensity measures, engineering demand parameters, and performance levels are proposed for blast engineering. Some similarities and differences between performance approaches for blast and earthquake engineering are identified.

INTRODUCTION

Performance-based engineering of buildings and infrastructure for gravity and windstorm loadings has been indirectly undertaken for more than 20 years since the introduction of strength (concrete structures) and load-and-resistance-factor (steel structures) design in the 1960s and 1970s. Such engineering of buildings and infrastructure has been based on force-based analysis and design checking of *components* using

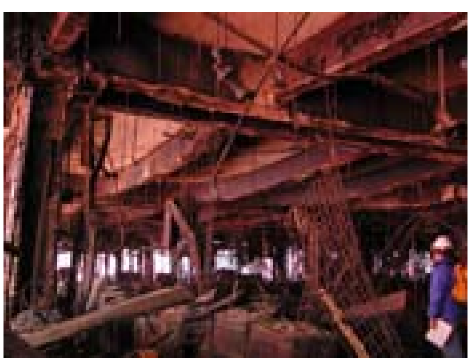
$$\sum \alpha_i L_i \le \phi C \tag{1}$$

where α_i are load factors, L_i are load effects (e.g., dead load, live load), ϕ is a capacity reduction factor for the action that is being checked (e.g., moment, axial load, shear force) and C is the component capacity that is determined using a materials standard such as the AISC Load and Resistance Factor Design Manual (AISC 2002). Factored component *force* demands are required to be less than or equal to de-rated component *force* capacities. The values of α_i and ϕ were selected to ensure that the probability of component failure, measured here as component demands greater than component capacities, is extremely low. *Global* performance of a framing system is measured by performance at the *component* level. No statements are made regarding the relationship between *component* and *system* failure.

Extreme loadings on buildings and infrastructure are produced by natural and man-made hazards including strong earthquakes, hurricane and tornado winds, blast, fire and equipment malfunctions. Setting aside equipment malfunctions for the purpose of this paper, the extreme loadings of Figure 1 should be resisted by buildings and infrastructure without collapse for sufficient time so as to allow the occupants the time required to exit the structure.

a. Earthquakes

b. Blast



c. Fire

d. Hurricanes

Figure 1. Extreme loadings and effects on building structures

Blast and earthquake loadings are short-term loadings with durations measured in milli-seconds and seconds, respectively. For such loadings, component and system ductility can be utilized to avoid system collapse. For relatively long-duration loads such as hurricane wind loadings on buildings, strength alone must be used to avoid collapse. Because component and system ductility are related to framing system displacements and deformations, and not component forces, performance-based engineering for extreme blast and earthquake loadings must be displacement or deformation-based based rather than force-based per (1).

The following sections of this paper provide summary information on the state-of-the-practice in performance-based earthquake engineering and the framework for on-going and future developments in performance-based earthquake engineering. Aspects of the performance-based earthquake engineering framework that might prove useful in the development of performance-based guidelines for blast engineering of buildings are identified. Some similarities and differences between performance approaches for blast and earthquake engineering are identified.

PERFORMANCE-BASED EARTHQUAKE ENGINEERING OF BUILDING STRUCTURES

Practice of performance-oriented earthquake engineering

The traditional prescriptive provisions for seismic design contained in U.S. building codes (e.g., ICBO 1997; FEMA 2000b) and under development since the late 1920s (ATC 1995) could be viewed as performance-oriented in that they were developed with the *intent* of achieving specific performance, that is, avoidance of collapse and protection of life safety. It was assumed by those engineers preparing the codes that buildings designed to these prescriptive provisions would (1) not collapse in very rare earthquake; (2) provide life safety for rare earthquakes; (3) suffer only limited repairable damage in moderate shaking; and (4) be undamaged in more frequent, minor earthquakes. The shortcomings of the prescriptive procedures include fuzzy definitions of performance and hazard and the fact that the procedures do not include an actual evaluation of the performance capability of a design to achieve any of these performance objectives. Further, records of earthquake damage to buildings over the past 70+ years following minor, moderate and intense earthquake shaking has demonstrated that none of the four performance objectives has been realized reliably. Deficiencies in the prescriptive provisions in terms of accomplishing the four target objectives have been identified following each significant earthquake in the United States and substantial revisions to the prescriptive provisions have then been made.

Performance expectations for mission-critical buildings began to evolve in the mid-1970s following severe damage to a number of emergency response facilities, most notably hospitals, in the 1971 San Fernando earthquake. Earthquake engineers decided that those buildings deemed to be essential for post-earthquake response and recovery (e.g., hospitals, fire stations, communications centers and similar facilities) should be designed to remain operational following severe earthquakes, and assumed that this would be achieved by boosting the required strength of such buildings by 50% compared with comparable non-essential buildings and requiring more rigorous quality assurance measures for the construction of essential facilities[1]. Since that time, the prescriptive provisions have evolved slowly but still include few direct procedures for predicting the performance of a particular building design, or for adjusting the design to affect the likely performance, other than through application of arbitrary importance factors that adjust the required strength.

Large economic losses and loss of function in mission-critical facilities following the 1989 Loma Prieta and 1994 Northridge earthquakes spurred the development of performance-based seismic design procedures with the goal of developing resilient, loss-resistant communities. In the early 1990s, experts design professionals and members of the academic community, ostensibly structural and geotechnical engineers, recognized that new and fundamentally different design approaches were needed because the prescriptive force-based procedures were a complex compendium of convoluted and sometimes contradictory requirements, were not directly tied to the performance they were intended to achieve, were not reliable in achieving the desired protection for society, were sometimes excessively costly to implement, and were not being targeted at appropriate performance goals in most cases.

[1] Although the 50% increase in strength served to reduce damage to the structural framing, there is no evidence to support the assumption that the essential facility would be operational after severe earthquake shaking.

Funding in the early to mid-1990s from the Federal Emergency Management Agency (FEMA) to the Applied Technology Council (ATC) and the Building Seismic Safety Council (BSSC) led to the development of the *NEHRP Guidelines and Commentary for Seismic Rehabilitation of Buildings* (FEMA 1997). This development effort 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 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. Another important feature of the *NEHRP Guidelines* (FEMA 273/274) was the introduction of standard performance levels, which quantified levels of structural and nonstructural damage, based on values of standard structural response parameters. The *NEHRP Guidelines* also specified a total of four linear and nonlinear analysis procedures, each of which could be used to estimate the values of predictive response parameters for a given level of shaking, and which could then be used to evaluate the building's predicted performance relative to the target performance levels contained in the performance objective. Figure 2 below illustrates the qualitative *performance levels* of FEMA 273/274 superimposed on a global force-displacement relationship for a sample building. The corresponding levels of damage are sketched in the figure. Brief descriptions of the building damage and business interruption (downtime) for the three performance levels are given in Table 1.

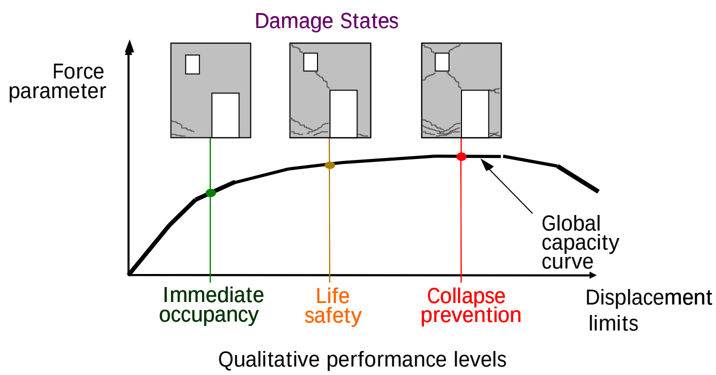


Figure 2. Qualitative performance levels of FEMA 273/274/356 (*after Comartin*)

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

Table 1. Building performance levels per FEMA 273/274/356 (*after Comartin*)

Figure 3 illustrates the FEMA 273/274 nonlinear static procedure for performance assessment. First, the earthquake hazard is characterized by one or more elastic acceleration response spectra. A nonlinear mathematical model of the building is prepared and subjected to monotonically increasing forces or displacements to create the capacity curve of Figure 3, which is generally plotted in terms of base shear (ordinate) versus roof displacement (abscissa). A maximum roof displacement is calculated for each design spectrum using an equivalent SDOF nonlinear representation of the building frame. Component deformation and force actions for performance assessment are then established for the given roof displacement using the results of the nonlinear static analysis. Component deformation and force demands are then checked against component deformation and force capacities, which are summarized for the performance levels of Figure 2 in the materials chapters of FEMA 273. If *component* demands do not exceed *component* capacities, the *building* performance objective are assumed to have been met.

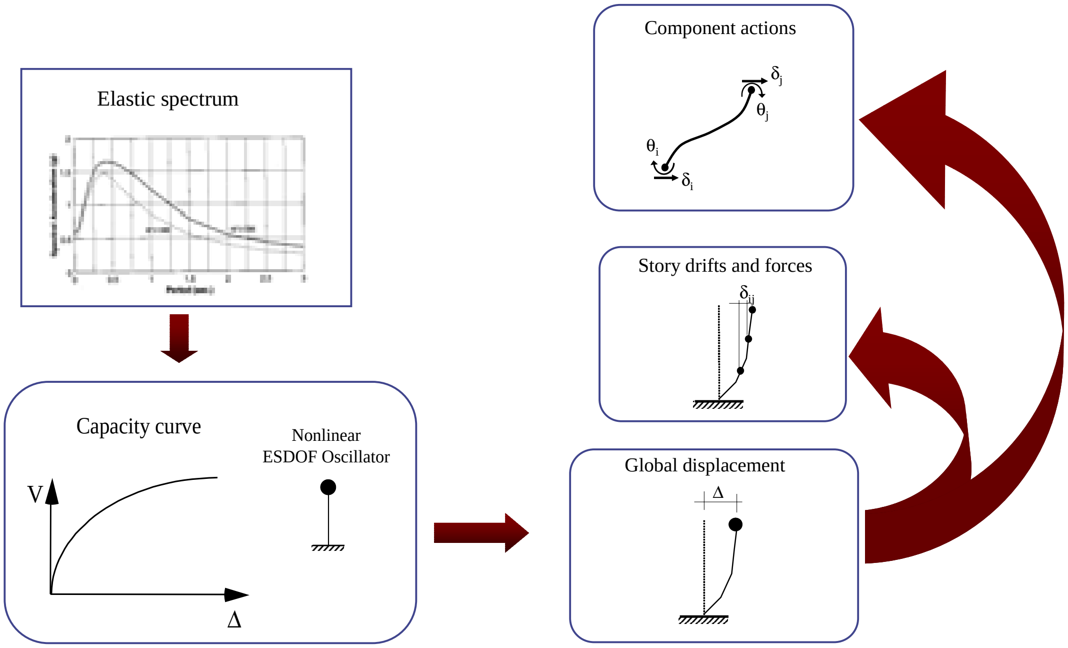


Figure 3. Performance assessment procedure of FEMA 273/274/356 (*after Comartin*)

Other projects including *ATC-40, Methodology for Evaluation and Upgrade of Concrete Buildings* and *Vision-2000 Framework for Performance-based Seismic Design Project* further developed and extended the technology developed in FEMA-273/274. These technologies were further refined by the American Society of Civil Engineers in their conversion of the FEMA-273/274 reports into the *Prestandard for Seismic Rehabilitation of Buildings, FEMA-356* (FEMA 2000a). Together, the *FEMA-356*, *ATC-40*, and *Vision-2000* documents define the current state of practice of performance-based seismic engineering.

Hamburger (2003) identified key shortcomings with the state of practice characterized by FEMA 273/356, including (1) the current procedures predict structural response and demands based on the global behavior of the structure but evaluate performance on the basis of damage sustained by individual components with the result that the poorest performing elements tend to control the prediction of structural performance, (2) much of the acceptance criteria contained in the documents, and used by engineers to evaluate the acceptability of a structure's performance is based on judgment, rather than laboratory data or other direct substantiating evidence, leading to questions regarding the reliability of the procedures, (3) many structural engineers view the guidelines as excessively conservative, when compared against designs developed using prescriptive criteria, however, the reliability of the guidelines and their ability to actually achieve the desired performance has never been established, and (4) the performance levels of FEMA 273/356 do not directly address some primary stakeholder concerns, that is probable repair costs and time of occupancy loss in the building, due to earthquake induced damage.

Following the discovery of unanticipated brittle fracture damage to welded moment-resisting steel frame buildings following the 1994 Northridge earthquake, FEMA sponsored a large project (widely known as the SAC Steel Project) to develop seismic evaluation and design criteria for that class of buildings. Key products of the project included a series of recommended design criteria documents [FEMA-350 (FEMA 2000c), FEMA-351 and FEMA-352], which incorporated performance-based design methodologies that addressed some of the issues associated with the state of practice per FEMA 273/356. The FEMA technical reports provided a large database of research data on the structural performance of this one structural system, which permitted the development of less subjective acceptance criteria for use in design, developed a methodology for evaluating the structural performance of a building based on its global response and behavior rather than solely on the amount of damage sustained by individual structural components, and developed a methodology for characterizing a level of confidence associated with a design's ability to meet a performance objective, addressing in part, concerns related to designer warranties of building performance (Hamburger 2003). Although the SAC performance methodology has not seen widespread acceptance, the prescriptive procedures contained in *FEMA-350* and *FEMA-351* that were validated using the performance-based methodology have been widely accepted and incorporated into national design standards and building codes.

Towards performance-based earthquake engineering

FEMA has contracted with the Applied Technology Council (ATC) to develop a *next generation* of performance-based seismic design guidelines for buildings, a project known as ATC-58. The guidelines are to be applicable to new and retrofit building construction and will address structural and non-structural components. Although focused primarily on design to resist earthquake effects, the next generation performance guidelines will be compatible with performance-based procedures being developed at this time for other hazards including fire and blast.

The ATC-58 project will utilize performance objectives that are both predictable (for design professionals) and meaningful and useful for decision makers. Preliminary project work tasks have revealed that these decision makers (or stakeholders) are a disparate group, representing many constituencies and levels of sophistication (Hamburger 2003). Decision makers include building developers, corporate facilities managers, corporate risk managers, institutional managers, lenders, insurers, public agencies and regulators. Each type of decision maker views performance from a different perspective and select performance goals using different decision making processes. The performance-based design methodology will include procedures for estimating risk on a design-specific basis, where risk will be expressed on either a deterministic (scenario basis or event) or a probabilistic basis. Risk will be expressed in terms of specific losses (e.g., cost of restoration of a facility to service once it is damaged, deaths and downtime) rather than through the use of traditional metrics (e.g., life safety in a design-basis earthquake).

The performance prediction process is similar to that utilized in the HAZUS national loss estimation software, although the individual steps in the process will be implemented differently. Figure 4 from Hamburger (2003) is the flow chart for the ATC-58 performance prediction methodology. Much of the methodology is based on procedures currently under development by the Pacific Earthquake Engineering Research (PEER) Center (Moehle 2003) with funding from the U.S. National Science Foundation.

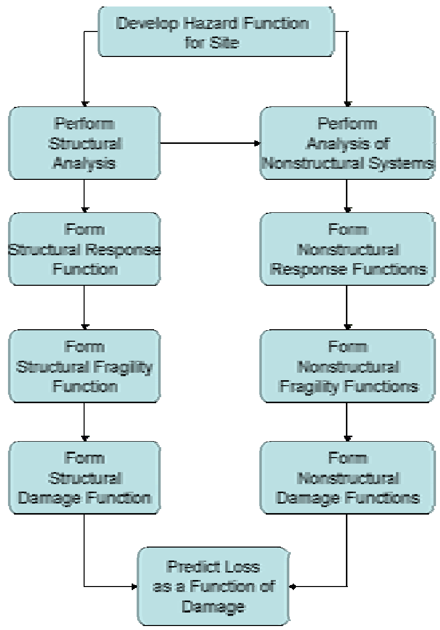


Figure 4. ATC-58 performance prediction flowchart (Hamburger 2003)

The PEER performance-based methodology is formalized on a probabilistic basis and is composed of four sequential steps: hazard assessment, structural/nonstructural component analysis, damage evaluation, and loss analysis or risk assessment. The product from each of these four steps is characterized by a generalized variable: Intensity Measure (*IM*), Engineering Demand Parameter (*EDP*), Damage Measure (*DM*), and Decision Variable (*DV*), for each of the steps, respectively. Figure 5 illustrates the methodology and its probabilistic underpinnings. The variables are expressed in terms of conditional probabilities of exceedance (e.g., $p(EDP\,|\,IM)$) and the approach of Figure 5 assumes that the conditional probabilities between the parameters are independent. Moehle (2003) and Hamburger (2003) describes the performance-based methodology that has been adopted for the ATC-58 project. Key features of the methodology as presented by Moehle and Hamburger are summarized below for a building of a given geometry and design (termed *D* in the figure) and location (termed *O* in the figure). As such, the building and the hazard are fully defined.

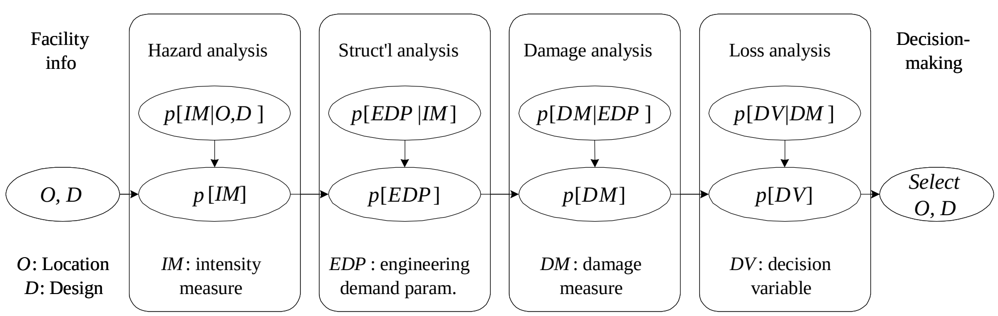


Figure 5. Probabilistic underpinnings of the PEER and ATC-58 performance methodologies (Moehle 2003)

Hazard analysis is the first of the four steps and produces ground motion Intensity Measures (*IM*s). The traditional IMs are peak ground acceleration and spectral acceleration at selected periods. Values for the *IM*s are obtained by probabilistic seismic hazard assessment at the location of the site (*O*). *IM*s are typically described as a mean annual probability of exceedance of the IM ($p\langle IM \rangle$ in Figure 5). The second step in the process is to use the *IM*s and the corresponding earthquake histories for simulation of the building response and the estimation of Engineering

Demand Parameters (*EDP*s). *EDP*s, which traditionally have included component forces and deformations and story drifts, are calculated by linear or nonlinear methods of analysis. The products of the analysis are a conditional probability, $p(EDP \mid IM)$, for each *EDP*, which are integrated over the $p\langle IM\rangle$ to estimate the mean annual probability of exceedance of each *EDP*, $p\langle EDP\rangle$. The third step in the process is to relate the *EDP*s to Damage Measures (*DM*s) that describe the physical state of the building. The outcome of this step are conditional probabilities, $p(DM \mid EDP)$, which can be integrated over the $p\langle EDP\rangle$ to calculate the mean annual probability of exceedance for the *DM*, $p\langle DM\rangle$. The fourth and final step in the PEER_ATC-58 methodology is to calculate Decision Variables (*DV*s). The mean annual probability of exceeding a *DV*, $p\langle DV\rangle$, is calculated by integrating the conditional probability $p(DV \mid DM)$ (or loss function) over the $p\langle DM\rangle$. The PEER_ATC-58 methodology can be expressed in terms of a triple integral of (2) based on the total probability theorem, namely,

$$v(DV) = \iiint G\langle DV \big| DM\rangle \mid dG\langle DM \big| EDP\rangle \mid dG\langle EDP \big| IM\rangle \mid d\lambda(IM) \tag{2}$$

where all terms have been defined previously. Column 2 of Table 2 below lists *IM*s, *EDP*s, *DM*s and *DV*s that could be adopted by the ATC-58 project for steel moment-frame construction. Column 3 lists similar measures that could be applied in the case of blast engineering.

	Earthquake engineering	Blast engineering
Intensity Measures	Peak ground acceleration Spectral acceleration at T_1 Spectral acceleration at T_1 and T_2	Charge weight and standoff Charge weight and location
Engineering Demand Parameters[1]	Demand-to-capacity ratios Beam plastic rotation Beam shear Column axial load, moment, shear Column plastic rotation Inter-story drift	Demand-to-capacity ratios Beam plastic rotation Beam shear, axial load Column axial load, moment, shear Column plastic rotation Floor vertical displacement
Damage Measures	Deaths Dollars Downtime	Deaths Dollars Downtime
Decision Variables	Annualized loss Performance objective	Performance objective

1. For steel moment-frame construction only

Table 2. Sample *IM*s, *EDP*s, *DM*s and *DV*s for performance-based engineering

PERFORMANCE-BASED BLAST ENGINEERING

Introduction

Prior to the mid-1990s, analysis and design of building structures in the United States to resist the effects of blast loading and progressive collapse was undertaken by a relatively small group of specialty design professional consultants for a limited number of clients that managed high-exposure facilities such as government buildings, courthouses, and defense- and energy-related structures. Mainstream structural-engineering consultancies were not involved in blast engineering. The terrorist attacks on the World Trade Center in 1993 and 2001, the Murrah Building in 1995, and the Pentagon in 2001 altered substantially the attitude of the structural engineering community, building owners and insurers regarding blast design of commercial building construction, and there is renewed design-professional interest in blast engineering. However, because the blast-engineering design-professional community is smaller than the earthquake community and blast considerations in *commercial* building design were the exception rather than the norm, there has been no national effort, on the scale of the FEMA-BSSC effort for earthquake engineering (FEMA 2000b), to produce guidelines and commentary for the analysis and design of blast- and progressive-collapse-resistant buildings.

The General Services Administration (GSA) has developed guidelines for progressive collapse analysis and design for new federal office buildings and major modernization projects (GSA 2003) but these guidelines are for limited distribution at the time of this writing. The GSA guidelines represent the state-of-the-practice in blast engineering of buildings but, similar to current building codes for seismic design, make use of indirect methods of analysis and prescriptive procedures of unknown reliability (Hamburger and Whittaker 2003). Resource documents for blast engineering are being developed currently by FEMA (FEMA 2004a, 2004b) but these documents will not provide explicit guidelines for analysis and design.

Performance-based blast engineering should make possible a process that permits design and construction of buildings with a realistic and reliable understanding of the risk of loss (physical, direct economic and indirect economic) that might occur as a result of future attack. This process could be used to (a) predict the global response, degree of damage (and perhaps economic loss) to a building subjected to a scenario blast event (Figure 6a) or physical attack (Figure 6b); (b) design individual buildings that are more loss-resistant than typical buildings designed using prescriptive criteria of a documents similar to GSA (2003); (c) design individual buildings with a higher confidence that they will actually be able to perform as intended for a blast attack; (d) design individual buildings that are capable of meeting the performance intent of the prescriptive criteria, but at lower construction cost than would be possible using the prescriptive criteria; (e) design individual buildings that are capable of meeting the performance intent of the prescriptive criteria, but which do not comply with all of the limitations of the prescriptive criteria with regard to configuration, materials and systems; (f) investigate the performance of typical buildings designed using prescriptive provisions and develop judgments as to the adequacy of this performance; and (g) formulate improvements to the prescriptive provisions so that more consistent and reliable performance is attained by buildings designed using prescriptive provisions.

Towards performance-based blast engineering

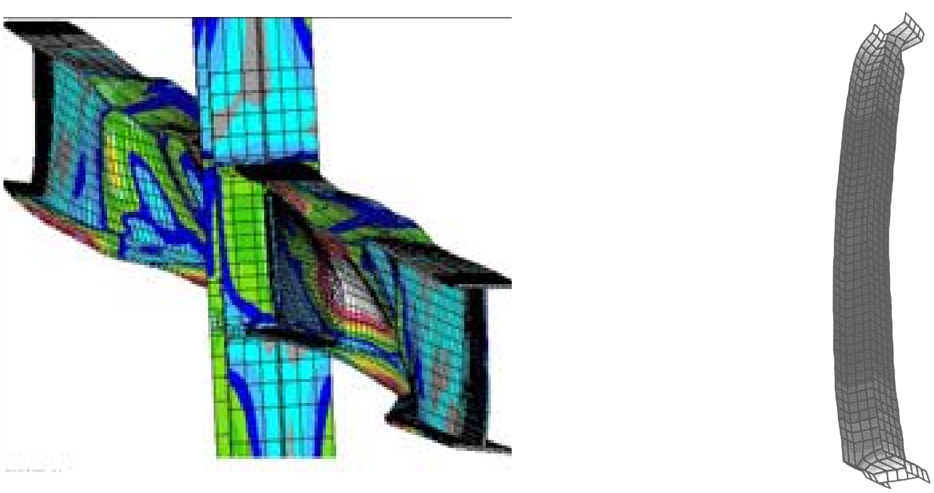
Components of equation (2) are broadly applicable to performance-based engineering for all loading conditions, normal and extreme. Significant overlaps should exist for extreme blast and earthquake loadings because inelastic response of the framing system is anticipated in both cases. That said, there are significant differences between blast and earthquake engineering in the loading environment (hazard or *IM*s) and important differences in simulation procedures and component response (*EDP*) assessment.

Blast loads on building structures (Biggs 1964; Mays and Smith 1995; Conrath et al. 1999) produce fundamentally different component responses than earthquake shaking. Further, blast loads are characterized deterministically at this time using scenario events (α charge weight at β distance) and not probabilistically using a hazard curve as described in the previous section. Using the terminology associated with equation (2), sample *IM*s for blast loading are listed in Table 2 above for explosives placed outside and inside a building. Similar to the translation of earthquake *IM*s into earthquake histories for the purpose of simulation, blast *IM*s must be transformed into loading functions, including pressure-impulse curves for assessing component integrity (response) to direct air-blast and the likelihood of component loss; and loading functions associated with component elimination due to air blast.

a. Murrah Federal Building b. Deutsche Bank building (Berman et al. 2002)

Figure 6. Global response of buildings to blast loadings and physical attack

Simulation of building response to earthquake shaking involves subjecting a (nonlinear) mathematical model of the building frame to one or more earthquake histories. Nonlinear component models for such simulation should be based on experimental test data similar to that collected systematically in the SAC Steel Project (FEMA 2000c). Two assumptions made in the SAC Steel testing program and for the development of beam-component models were that beams are deformed primarily about their strong axis and twisting and distortion of the gross section is avoided. Both assumptions are reasonable for components in building frames subjected to earthquake shaking. However, neither assumption is valid for components in the immediate vicinity of air blast because such pressure loadings can produce gross damage and failure as shown in the numerical simulations of Figure 7. Further, the component models for earthquake simulation are based primarily on cyclic testing in the absence of significant axial load: testing conditions that are clearly inappropriate for components resisting progressive collapse.



a. W-shape beam cross section (*after Karns*) b. W-shape beam web (*after Crawford*)

Figure 7. Gross distortion of steel components due to direct air blast

The *EDP*s of Table 2 for performance-based blast engineering are virtually identical to those for earthquake engineering. Demand-to-capacity (D/C) ratios are useful when linear methods of analysis are employed but calibration of D/C ratios to Damage Measures (*DM*s) using nonlinear response-history simulation is required. Values for the remaining *EDP*s could be output by nonlinear response simulations to develop *DM*s. Different conditional probabilities $p(DM \mid EDP)$ will result from air-blast and progressive collapse type loadings. Much full-scale experimental testing will be required to both facilitate such calculations of conditional probabilities and calibrate existing component models for blast and progressive-collapse analysis (Crawford et al. 2001).

Decision Variables (*DV*s) in the form of performance objectives have been identified for use in performance-based earthquake engineering. Details are provided in Table 1. Table 3 provides similar information for performance-based blast engineering. The proposed performance levels, damage descriptions and downtime estimates are preliminary and mutable, and are presented only to kindle discussion on *DV*s for performance-based blast engineering.

Performance level	*Damage description*	*Downtime*
Immediate occupancy	Negligible structural and nonstructural damage	24 hours
Life safety	Nonstructural and glazing damage; probable structural damage to beams and columns over a limited area; no collapse; adequate emergency egress; no loss of life due to structural damage	Several months to a year
Collapse prevention	Severe structural and nonstructural damage; structural damage over a wide area; incipient collapse; possible restricted egress; minimal loss of life due to structural damage	Possible total loss

Table 3. Possible building performance levels for blast-type loadings

SUMMARY REMARKS

Extreme events such as blast loadings and severe earthquake shaking will generally induce nonlinear behavior in building frames and produce substantial nonstructural damage. Although the current prescriptive procedures for design against blast and earthquake loadings might produce buildings of acceptable safety, the procedures are indirect, of unknown reliability, and might result in inefficient and costly construction. Performance-based engineering should facilitate design and construction of buildings with a realistic and reliable understanding of the risk of loss (physical, direct economic and indirect economic) that might occur as a result of future blast attack, earthquake shaking (or both).

The second-generation performance-based earthquake engineering methodology, which is being adopted for the ATC-58 project, is applicable conceptually to performance-based blast engineering. Sample intensity measures, engineering demand parameters, and performance levels for use in performance-based blast engineering were presented to foster discussion. Key similarities and differences between performance approaches for blast and earthquake engineering were identified.

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