한국지진공학회 논문집 | 제11권 제4호 (통권 제56호) | 2007년 8월, pp. 13~23

A Simplified Procedure for Performance-Based Design

Farzin Zareian[1) · Helmut Krawinkler[2)

ABSTRACT >> This paper focuses on providing a practical approach for decision making in Performance-Based Design (PBD). Satisfactory performance is defined by several performance objectives that place limits on direct (monetary) loss and on a tolerable probability of collapse. No specific limits are placed on conventional engineering parameters such as forces or deformations, although it is assumed that sound capacity design principles are followed in the design process. The proposed design procedure incorporates different performance objectives up front, before the structural system is created, and assists engineers in making informed decisions on the choice of an effective structural system and its stiffness (period), base shear strength, and other important global structural parameters. The tools needed to implement this design process are (1) hazard curves for a specific ground motion intensity measure, (2) mean loss curves for structural and nonstructural subsystems, (3) structural response curves that relate, for different structural systems, a ground motion intensity measure to the engineering demand parameter (e.g., interstory drift or floor acceleration) on which the subsystem loss depends, and (4) collapse fragility curves. Since the proposed procedure facilitates decision making in the conceptual design process, it is referred to as a Design Decision Support System, DDSS. Implementation of the DDSS is illustrated in an example to demonstrate its practicality.

Key words hazard curves, mean loss curves, structural response curves, collapse fragility curves, Design Decision Support System

1. INTRODUCTION

In the writers' opinion, structural engineering is a noble profession populated by individuals who want to serve society to the best of their ability. But structural engineers are low on the totem pole of esteem and reverence in society, at least in the US. They are respected but not envied. Their product, the design and detailing of structural systems, is considered by most owners and architects a necessary cost, but rarely as a benefit. Here is where the problem lies - structural engineers do not advocate the benefit of their services. One reason is (likely there are many others) that engineers use terminology that means nothing to stake holders who make important decisions, may they be owners, users, architects, contractors, or particularly lawyers and business or facility managers. We paint ourselves into a corner by trying to communicate "forces and deformations". Nobody cares about these esoteric quantities, except we.

The fact of life is that in the US, and likely in most other countries, decisions are based on dollars or euros or yens or Persian rials or any other currency. This applies even to life safety, because it is economically unfeasible to design and build in a manner that collapse will never occur even in the most extreme natural event, such as a catastrophic earthquake. In our world of limited resources, money needed for improvement of seismic safety competes with resources needed for cancer research and traffic safety, and for many other good causes. For these and many other reasons we engineers have to learn to communicate in term understood by those who make decisions. Forces and deformations will not do, and neither will be ambiguous terms such as moderate and severe damage. This will not sell. We have the option of being satisfied with our present role and swallow reporters' questions such as the following one posed to the second author after the damaging Northridge

1) Assistant Professor, Department of Civil and Environmental Engineering, Univer. of California, USA

2) Professor, Department of Civil and Environmental Engineering, Stanford University, USA
(대표저자: krawinkler@stanford.edu)

본 논문에 대한 토의를 2007년 10월 31일까지 학회로 보내 주시면 그 결과를 게재하겠습니다.

(논문접수일 : 2007. 7. 6 / 심사종료일 : 2007. 7. 26)

earthquake: "How come you guys still havn't gotten it right?" Or we have the option to adapt to societal terminology and express our decisions in generally understood terms, such as expected annual loss or chance (probability) of collapse.

2. PERFORMANCE-BASED EARTHQUAKE ENGINEERING

This is where performance-based earthquake engineering comes into play. This term has been around for decades, but only recently has it been brought to the level where we can express, quantitatively, direct losses, downtime losses, and casualties. Much work has been done during the last decade in quantifying performance, in many organizations and many countries. We apologize for referring in this short paper only to work done within the Pacific Earthquake Engineering Research (PEER) Center. Past work focused mostly on Performance-Based Assessment (PBA) and not Performance-Based Design (PBD). The problem with rigorous PBA methods, such as those advocated by PEER (e.g., [Cornell and Krawinkler, 2000][4], [Krawinkler, 2002][8], [Krawinkler and Miranda, 2004][9], [Deierlein, 2004][5], [Aslani, 2005][1]) is that they require elaborate formulations in order to incorporate and propagate uncertainties, many of which are difficult to quantify at this time because of lack of data.

Researchers at the PEER Center have developed a sophisticated PBA methodology that utilizes a chain of four random variables: *Intensity Measure (IM), Engineering Demand Parameter (EDP), Damage Measure (DM)*, and *Decision Variable (DV). IM* is a scalar or vector [Baker and Cornell, 2004] quantity that represents the intensity of a ground motion. Traditionally, the elastic spectral acceleration at the first mode period of the structural system, $S_a(T_1)$, is used as an *IM. EDP* is a structural response parameter (e.g., maximum story drift or maximum floor acceleration) and is estimated through nonlinear response analysis of a structural model subjected to ground motions [Medina and Krawinkler, 2003][12], [Ibarra and Krawinkler, 2005][6]. *DM* is the representation of component damage (e.g., cracks in partition walls) and is determined based on a repair strategy and many other considerations [Taghavi and Miranda, 2003][13], [Aslani, 2005].[1] *DV* is a measure of performance, with a focus on three major loss categories: direct (monetary) loss, downtime loss, and life loss.

Implementation of rigorous PBA in conceptual design, the stage at which most of the important design decisions are made, is rarely feasible. Firstly, in order to complete a PBA process, the building configuration and its structural components have to be predetermined in order to compute relevant *EDPs* through structural analysis. One could perform an iterative assessment that starts with a judgmental conceptual design that is refined in several PBA iterations. This is not a desirable PBD process unless targeted performance objectives already are considered in the judgmental conceptual design phase. Secondly, the PBA methodology has a probabilistic basis intended to incorporate the effect of all important aleatory and epistemic uncertainties in the evaluation of *IMs, EDPs|IMs, DMs|EDPs*, and *DVs|DMs*. This requires more effort (and data gathering) than can be justified for most practical design projects. (A comprehensive implementation of rigorous PBA is documented in a PEER testbed report, [Krawinkler, ed., 2005].[11])

The authors believe that good design should be based on concepts that incorporate performance objectives up front in the design decision process, and that PBA should be a verification and refinement process of a <u>good</u> conceptual design. A bad choice of a structural system cannot be transformed into a desirable alternative, even with the most sophisticated PBA. Also, conceptual design should be easy to implement for ordinary structures that comprise most of the structural engineering work in consulting offices. For this reason, the simplified PBD process proposed in this paper focuses on <u>expected (mean) values</u> of all random variables, and delays the uncertainty evaluation and propagation till the PBA phase. This facilitates up-front design decision making by permitting a focus on the most important global behavior aspects without having to deal with mathematical formulations that are important but obscure behavior-based decision making.

In this paper we propose a semi-graphical procedure that incorporates all the important variables of the PBA process but permits a rapid assessment of relevant design alternatives, which are represented by structural system choices (shear walls and moment-resisting frames are implemented so for) with a wide range of parameters that define all salient characteristics of the structural system. The objective is to facilitate decision making on these system choices in the conceptual design phase. For this reason the proposed approach is referred to as a Design Decision Support System (DDSS). The details of the DDSS are elaborated in the Ph.D. dissertation of the senior author [Zareian, 2006].[15]

3. THE THREE DOMAINS OF PERFORMANCE-BASED EARTHQUAKE ENGINEERING

Performance-based earthquake engineering comprises three domains; the Hazard Domain, the Loss Domain, and the Structural System Domain, as illustrated in Figure 1. In the probabilistic PBA world, the three domains are populated by probabilistic data on *IMs* (Hazard Domain), *IM-EDP* relationships (Structural System Domain) and damage (*EDP-DM* relationships) and loss data (*DM-DV* relationships) (Loss Domain). In the simplified world considered here, it is assumed that mean relationships between the variables are adequate to make important design decisions. The Structural System and Loss Domains have to be partitioned into a no-collapse (NC) sub-domain and a collapse (C) sub-domain, as collapse contributes to all kinds of losses.

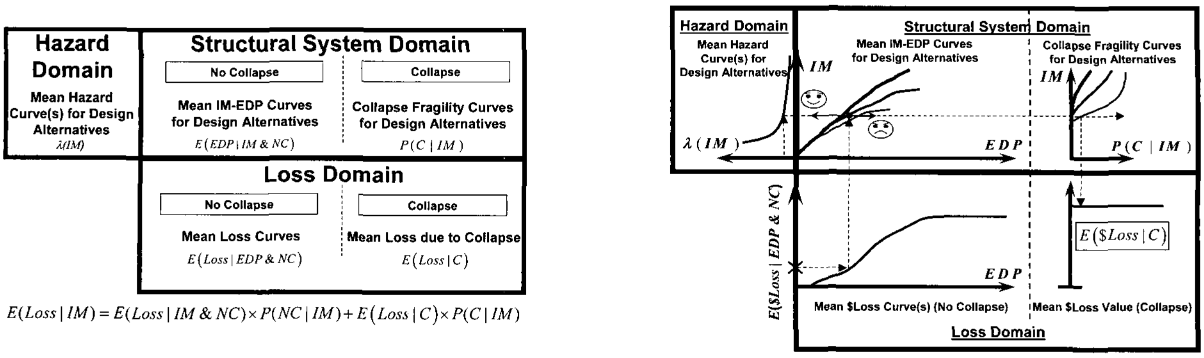
The Hazard Domain contains mean hazard curves for the *IM* selected to represent the site hazard for the building to be designed. The Loss Domain is populated by mean loss curves. The type of curves depends on the performance category being considered, i.e., direct (monetary) loss, downtime loss, and life loss. In each case, the total expected loss can be computed by the equation at the bottom of Figure 1(a) as the sum of the losses if collapse does not occur (i.e., conditioned on NC) and the losses if collapse does occur (i.e., conditioned on C). This paper is concerned only with direct loss ($loss) and probability of collapse. The latter affects direct loss but is used also as a surrogate for life loss, in which case a tolerable probability of collapse becomes a separate performance objective. Downtime loss, in concept, is described by curves similar to mean $loss curves, but is not addressed here because it covers a wide range of scenarios that is beyond the scope of this paper.

The domain that couples the Loss Domain and the Hazard Domain is the Structural System Domain. In the no-collapse sub-domain it contains mean *IM-EDP* relationships for design alternatives, and in the collapse sub-domain it contains collapse fragility curves for the same alternatives. The relationships that populate each of the three domains are discussed in more detail in the following subsections.

Provided that all these relationships have been defined, the design decision process follows the procedure illustrated in Figure 1(b). Owners (or regulatory bodies) define performance objectives, such as limitation of $loss to an acceptable value at a specific hazard level (e.g., the 50/50 level) and a tolerable probability of collapse at another hazard level. In the Loss Domain the mean loss curve is entered with a target value, and a vertical line is drawn towards the Structural System Domain. In the Hazard Domain the mean hazard curve is entered with the specific hazard value and a horizontal line is drawn towards the Structural System Domain. The intersection of the two lines defines a "design target" point, which divides the space of design alternatives into a feasible and unfeasible subspace (mean IM-EDP curve intersects the horizontal line to the left or the right of the design target point, respectively). The intersection of the horizontal line with the collapse fragility curve in the collapse sub-domain shows the probability of collapse for the specific hazard level and quantifies the contribution of collapse to the expected loss [E(Loss|C)xP(C|*IM*)]. Furthermore, the collapse fragility curve also quantifies the probability of collapse at the hazard levels at which a tolerable probability of collapse is specified, and serves to eliminate design alternatives that violate the stated objective of a tolerable probability of collapse. Later in this paper this process is illustrated on hand of a comprehensive example.

3.1 Information Provided in the Hazard Domain

The hazard domain contains mean hazard curves for an *IM* for which the hazard is (or can be) quantified and



〈Figure 1〉 The three domains of PBEE, represented by mean data [Krawinkler et al., 2005]
(a) symbolic representation, (b) domain representation by mean relationships, and design target point

that can be related to building response. The spectral acceleration at the first mode period of the structure, $S_a(T_1)$, is a widely used intensity measure for this purpose. For many countries, maps of S_a associated with specific hazard levels (usually 50/50 and 10/50, and maybe 2/50) are available at a few specific periods. It is assumed here that sufficient information is available to generate hazard curves for the fundamental period of the design alternatives.

3.2 Information Provided in the Loss Domain

The construction of mean loss curves is a major effort. A process for accomplishing it is discussed in [Zareian, 2006].(15) Loss is incurred in individual structural and nonstructural components of a building, and the value of components and component assemblies, and the associated losses, depend strongly on the function of the building (office building, hotel, hospital, etc.). In [Krawinkler et al., 2005](10) is it recommended to aggregate component losses into subsystem losses, where a subsystem is defined either at the story level or the building level and contains all components that can be correlated with the same *EDP*. For typical buildings it is appropriate to define three types of subsystems, i.e., nonstructural drift sensitive subsystems (NSDSS), nonstructural acceleration sensitive subsystems (NSASS), and structural subsystems (SS). In the first type (NSDSS) maximum interstory drift is used as the *EDP*, in the second type (NSASS) peak floor acceleration is used as the *EDP*, and in the third type (SS) the appropriate *EDP* depends on the type of structural system (e.g., moment-resisting frame or shear wall).

The challenge is to quantify mean loss curves for each subsystem. Fragility curves have to be developed for each component j of a subsystem, which define, as a function of the appropriate *EDP*, the probability of being in or exceeding specific damage states (DS_{jk}). Presuming that the expected cost of repair for each damage state is known, the total loss in the subsystem, given *EDP* and no-collapse, is computed by summing the expected losses for all components of the subsystem as follows:

$$E[loss \mid EDP, NC] = \sum_{j=1}^{n}\sum_{k=1}^{m_j} E\left[loss_j \mid DS_{jk}, NC\right] \cdot P\left(DS_{jk} \mid EDP, NC\right) \quad (1)$$

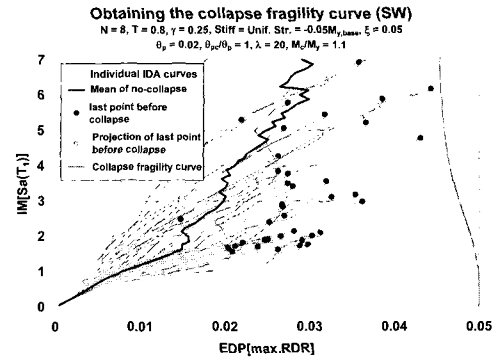
The result will be a subsystem mean loss curve whose shape will be occupancy specific. Work is in progress to quantify generic loss curves for specific building occupancies such as office and hotel buildings, and to express the total value of the subsystem as a multiple of a value per unit area (m^2 or ft^2). In the example demonstrated later the shape of the mean loss curves is assumed to be linear as shown in Figures 4 and 5.

3.3 Information provided in the Structural System Domain

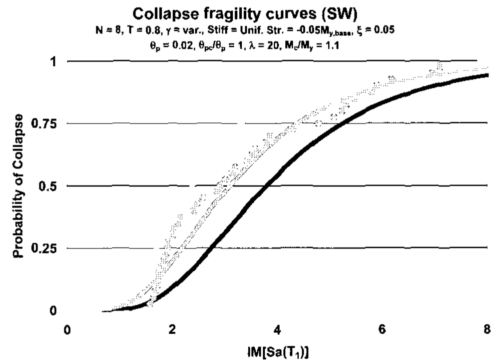
The structural system domain contains system-specific information that relates the *EDPs* needed do quantify losses to the *IMs* in the hazard domain. This domain is populated by mean *IM-EDP* relationships obtained by subjecting a given structural system to a series of ground motions of increasing intensity corresponding to the *IM* used in the hazard domain. There are many ways to obtain such mean *IM-EDP* relationships, the most common one being the use of incremental dynamic analysis, IDA [Vamvatsikos and Cornell, 2002].(14) In the study summarized in [Zareian, 2006](15) a comprehensive set of generic moment-resisting frames and shear walls is studied in detail to obtain a database of response parameters (*EDPs* and collapse fragility curves) for regular steel and reinforced concrete structures. Structures from 4 to 18 stories are analysed, with periods equal to 0.1N, 0.15N, and 0.2N for moment resisting frames, and 0.1N, 0.075N, and 0.5N for shear wall structures. The yield strength of the structures is defined by a yield base shear coefficient $\gamma = V_y/W$. Three variations of strength and stiffness along the height are specified, and for the moment-resisting frames three cases of relative strength of columns versus beams are evaluated (to assess the strong column - weak girder effect). The components of the structures are modeled by elastic elements and rotational springs that describe the inelastic moment-rotation behavior at plastic hinge locations. The hysteretic properties of these rotational springs incorporate stiffness and strength deterioration properties of the type discussed in [Ibarra et al. 2005].[7]

Utilizing deteriorating component properties permits the execution of IDAs till the level of structural collapse, which in an IDA is defined as the largest intensity at which the dynamic analysis converges. The implication is that for the next increment of *IM* the structure no longer finds a state of equilibrium, which is synonymous with attaining a state of dynamic instability, i.e., collapse. Typical results, using an 8-story shear wall structure subjected to a set of 40 ground motions, are shown in Figure 2. In this example, the *EDP* is the maximum roof drift ratio (RDR), and the intensity measure is the spectral intensity at the first mode period of the structure, $S_a(T_1)$. The black line shows the mean of "survivors", meaning the mean of *EDP|IM* and no collapse, which is the information used in Figures 4 and 5 to represent the mean *IM-EDP* curves. It can be seen that the mean curve changes slope rather abruptly once collapse is observed for individual ground motions.

The structural system domain includes also collapse fragility curves. Such curves represent the cumulative distribution function for P(C|IM). These curves are obtained by projecting the last points of the IDA curves on the vertical axis as illustrated in the right part of Figure 2. In the standard CDF domain the fragility



⟨Figure 2⟩ Example of IDAs and mean *IM-EDP* curve

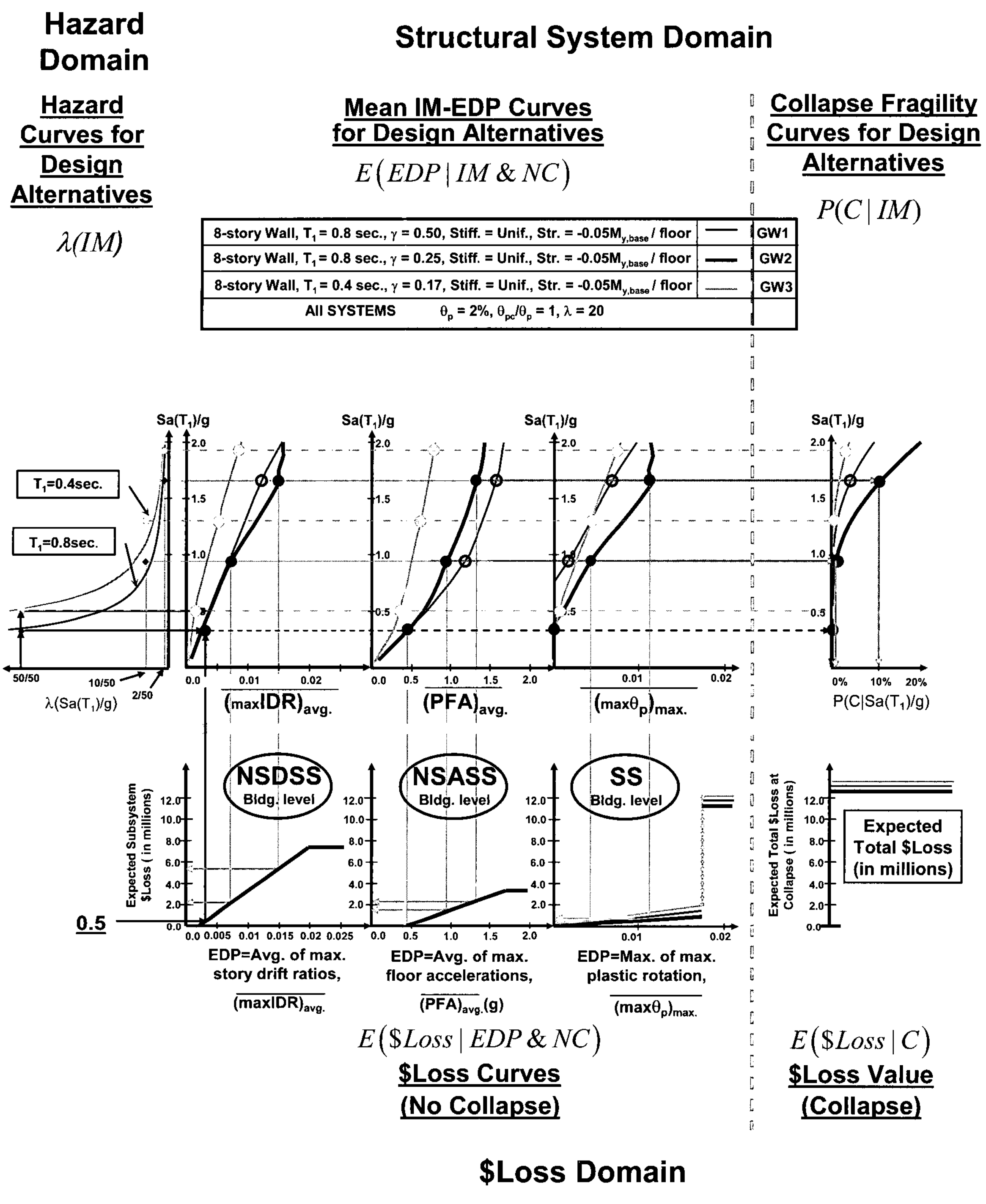


⟨Figure 3⟩ Examples of collapse fragility curves

curves are as shown in Figure 3. The middle curve is a lognormal distribution fitted to the data point of Figure 2, i.e., for a yield base shear coefficient of $\gamma = 0.25$. The other two collapse fragility curves are for base shear coefficients of $\gamma = 0.5$ and 0.17, respectively. The information presented here illustrates how the mean *IM-EDP* curves and the collapse fragility curves utilized in Figures 4 and 5 are obtained.

4. IMPLEMENTATION EXAMPLE

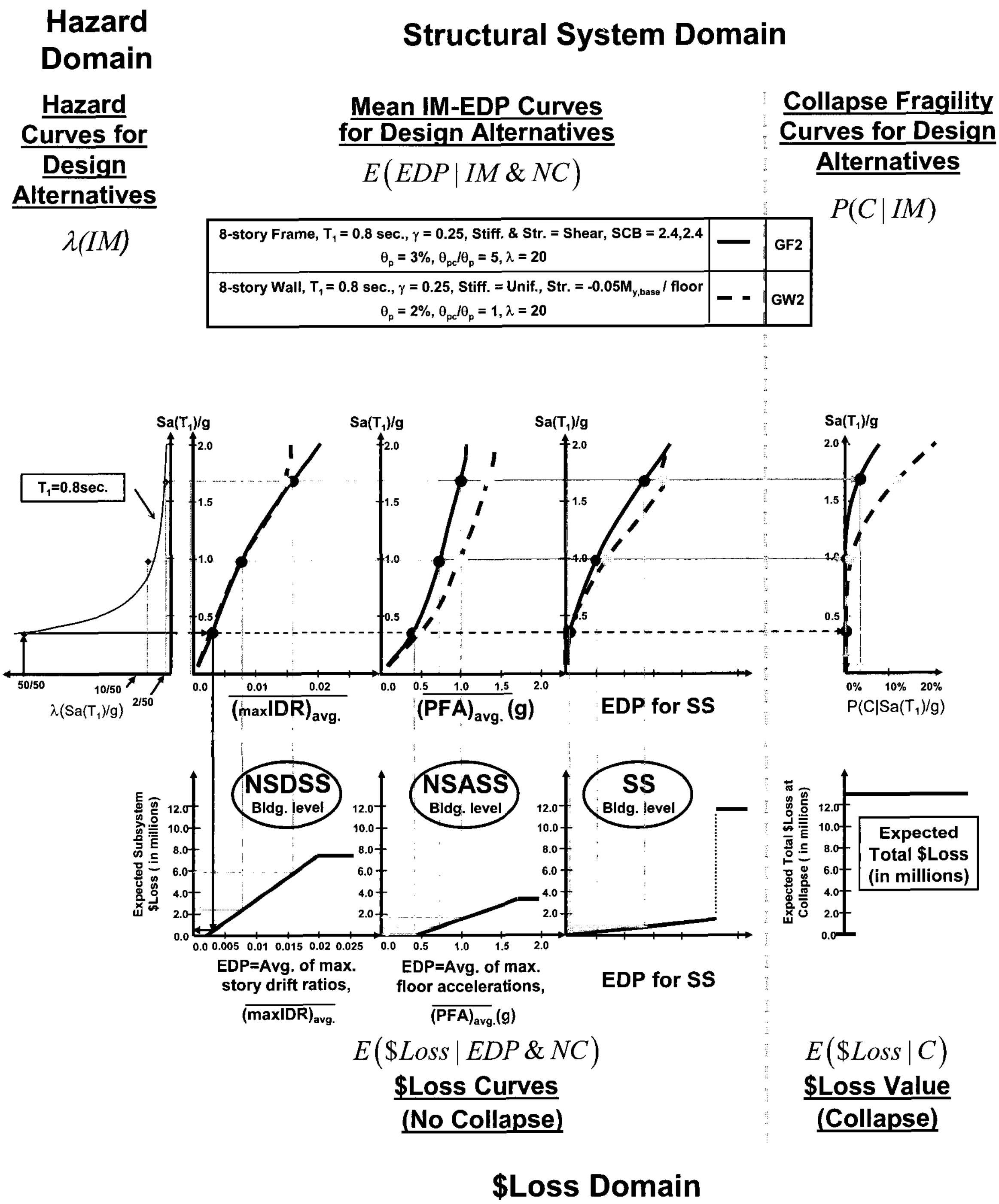
The design decision scenario is set as follows. The building for which a structural system is to be designed is an 8-story office building. The site is in Los Angeles, and the site hazard, in terms of spectral accelerations at the 50/50, 10/50, and 2/50 hazard levels is provided on a web site. The owner's performance targets are to limit expected monetary loss to $500,000 in case of a 50/50 event, and to limit the tolerable probability of collapse to 0.1 in case of a 2/50 event. In addition, the owner wants to limit the mean annual frequency (MAF) of collapse to 0.0002. Design alternatives for structural systems are reinforced concrete shear walls and moment resisting



〈Figure 4〉 Example of DDSS implementation in conceptual design with performance targets at discrete hazard levels, using building-level subsystems and exploring three shear wall design alternatives.

frames. The design decisions to be made are concerned with the type of system and the strength and stiffness of the most desirable design alternative. The structural system is regular in plan and elevation, thus, data of the type discussed earlier can be utilized for graphical representations of mean *IM-EDP* relationships.

The total expected building loss is disaggregated into expected loss in the NSDSS, NSASS, and SS subsystems. Such subsystems can be defined at the story level or the building level. The latter implies the most coarse loss assessment because no specific attention is paid to individual stories, which may have different absolute values and different relative values (e.g., NSDSS versus NSASS). The decision to use building subsystems



⟨Figure 5⟩ Example of DDSS implementation in conceptual design with performance targets at discrete hazard levels, exploring a shear and a moment-resisting frame alternative.

is made here only for brevity; it is equally feasible to use story level subsystems [Zareian, 2006][15] but then the discussion becomes longer and more figures are needed. The non-structural subsystems (NSDSS and NSASS) are assumed to be independent of the choice of the structural system (SS), which implies that they are equal for all design alternatives. The *EDP* that correlates best with the building level NSDSS subsystem loss is the average of the maximum interstory drift ratios over the height, *(max IDR)*$_{avg}$, and the *EDP* that correlates best with the building level NSASS loss is the average of peak floor accelerations over the height, *(PFA)*$_{max}$. Thus, mean SSDSS and NSASS loss curves are of the type shown in Figure 4, where the horizontal axis is the mean of *(max*

$IDR)_{avg}$ and $(PFA)_{max}$, respectively. The shape of the loss curves is to be determined through more refined loss estimation research and data gathering, and here is assumed by judgment to be linear as shown in the figure.

The building level SS subsystem value and loss depend on the selected structural system. For shear wall structures it is assumed that the *EDP* that correlates best with the SS loss is the maximum plastic hinge rotation at the base (only shear walls that fail in a relatively ductile failure mode are considered in this example). For moment frame alternatives it is assumed that the appropriate *EDP* is the average of the maximum plastic story drift angles over the height of the structure. Since the cost of the SS subsystem is relatively small for most buildings, the associated mean loss curve is rather flat and the SS loss is not dominant at all - except if the *EDP* exceeds a certain threshold value. This threshold value of *EDP* is associated with a drastic jump in the SS loss. This is the *EDP* at which the owner decides that post-earthquake repair of the building is ineffective and declares the building a total loss. This jump is evident in the SS loss curves in the Loss Domain of Figures 4 and 5.

Given the mean subsystem loss curves in the Loss Domain, and the period dependent mean hazard curves in the Hazard Domain, various structural system alternatives can be evaluated by populating the Structural System Domain with appropriate mean IM-EDP relationships and collapse fragility curves of design alternatives. As discussed earlier such curves have been developed for shear wall and moment frame structural systems with a wide range of system parameters. Figure 4 illustrates the process of design decision making on hand of the evaluation of three selected shear wall alternatives.

The target at the 50/50 hazard level is to limit the total building loss to about $500,000. As Figure 4 shows, the largest contributor to loss is the NSDSS subsystem. Thus, as a first attempt, it is justifiable to postulate that all the loss is in the NSDSS subsystem. Entering the NSDSS mean loss curve with a value of $500,000 and entering available hazard curves for various values of T_1 at the 50/50 hazard level, it becomes evident that structural systems with $T_1 > 0.8$ sec. will not satisfy the performance objective of monetary loss < $500,000 (the design point for $loss < $500,000 in the NSDSS sub-domain and $T_1 = 0.8$ sec. in the hazard domain is barely to the right of wall systems with $T_1 = 0.8$ sec. in the Structural System Domain, i.e. structures with a longer period would exceed the acceptable $loss performance objective). This places a clear limitation of the first mode period of the structural system, and therefore on the required stiffness of the system.

Using 0.8 sec. as an acceptable upper bound on T_1, the following three shear wall design alternatives are explored:

GW1: $T_1 = 0.8$ sec., $\gamma = V_y/W = 0.5$

GW2: $T_1 = 0.8$ sec., $\gamma = V_y/W = 0.25$, and

GW3: $T_1 = 0.4$ sec., $\gamma = V_y/W = 0.17$

In all three alternatives the wall dimensions along the height are constant, and the wall bending strength decreases at the rate of $0.05M_{y,base}$ per floor. The plastic rotation capacity at the base of the wall is 2% for all three alternatives, and the deterioration properties (θ_{pc}/θ_p and λ) for the three alternatives are also the same.

The mean *IM-EDP* curves for the three alternatives are taken from the database provided in [Zareian, 2006].[15]

From Figure 4 the monetary loss for each design alternative and for each subsystem can be evaluated at the 50/50 hazard level, and also at the 10/50 and 2/50 hazard levels. All three alternatives accomplish the performance objective of limiting the total monetary loss at the 50/50 hazard level to $500,000, as well as limiting the tolerable probability of collapse at the 2/50 hazard level to 10% (see right part of the Structural System Domain shown in Figure 4). Considering construction costs, there is no good reason to pursue GW1 further, as it is a more expensive alternative ($\gamma = V_y/W = 0.5$ as compared to 0.25 for GW2) and causes larger NSASS losses at the 10/50 and 2/50 hazard levels. The choice between GW2 and GW3 depends on cost versus loss reduction considerations, as GW3 is more expensive ($T_1 = 0.4$ sec. versus 0.8 sec.) but causes much less loss than GW2 at all hazard levels. Note that the hazard curves for GW2 and GW3 are very different. In the following discussion only alternative GW2 is pursued further.

In Figure 5 the pros and cons of the just selected wall alternative GW2 are contrasted with those of a moment resisting frame alternative of the same period (T_1 = 0.8 sec). Based on loss considerations alone, the frame alternative GF2 (γ = V_y/W = 0.25, member design controlled by stiffness [straight line deflected shape under code shear force pattern], member strength controlled by lateral loading alone [gravity loading has no effect on member strength], columns much stronger than beams, and plastic hinge rotation capacity at beam ends = 0.03 before deterioration) comes out the winner. GF2 and GW2 will experience about the same loss at the 50/50 hazard levels, but at higher hazard levels (10/50 and 2/50) the wall structure GW2 will experience larger loss in the NSASS subsystem. Furthermore, the probability of collapse at the 2/50 hazard level clearly is smaller for GF2. A final decision will have to incorporate construction costs, which are expected to be significantly smaller for the wall structure than for a very stiff frame structure with T = 0.1N.

A final decision could (or better should) be based on a more comprehensive loss evaluation rather than only a visual inspection of mean loss at discrete hazard levels. Since all relationships (hazard curves, *IM-EDP* curves, and mean loss curves) are continuous, it is a relatively small effort to find the mean annual loss by integrating the mean \$loss-*IM* curve over the hazard curve. Specific points on the mean \$loss-*IM* curve are obtained by summing the mean subsystem losses at various *IM* levels, and then numerical integration is utilized to integrate over the hazard curve. For the GF2 and GW2 design alternatives the resulting mean annual loss so obtained is \$17,000 and \$22,000, respectively.

A final decision should also incorporate a more thorough evaluation of the probability of collapse. At the 2/50 hazard level it is evident from the collapse sub-domain of the Structural System domain that the probability of collapse for the GF2 alternative is clearly smaller than that of the GW2 alternative. The mean annual frequency (MAF) of collapse can be obtained by integrating the collapse fragility curve over the hazard curve, which for these two alternatives results in a MAF of collapse of 0.00014 and 0.00023 for the GF2 and GW2 alternatives, respectively. In view of the up front stated performance objective of limiting the MAF of collapse to 0.0002, these values make the GW2 structural system alternative an undesirable one.

As a final observation, the presented scenario does not imply that GF2 is the "best" alternative in an absolute sense. It is the most desirable one of the alternatives evaluated here. But if shear wall alternatives with a somewhat shorter period than GW2 (and same strength and deterioration properties) are evaluated, it is found that a shear wall structure with T_1 = 0.6 sec. causes smaller monetary loss and collapse probability than the GF2 alternative. Considering that such a shear wall alternative is likely cheaper to build than a very stiff moment frame, it can be regarded as the most effective design alternative. For space limitations, this last design iteration is not presented here.

5. CONCLUDING REMARKS

In this paper we try to illustrate the potential of a proposed Design Decision Support System (DDSS). The ingredients needed for implementation are:

- Mean hazard curves that define the site hazard in terms of a specific *IM* ($S_a(T_1)$ *is used in this implementation)* and for the period (or period range) that defines the IM for a potentially effective structural system alternative,
- Mean loss curves for building or story subsystems, which relate the mean (expected) loss in a subsystem to an *EDP* that (1) correlates well with the loss in the subsystem, and (2) can be evaluated in a systematic manner for potentially effective structural system alternatives,
- Mean IM-EDP relationships for all *EDPs* in the loss domain for potentially effective structural system alternatives, and
- Collapse fragility curves for potentially effective structural system alternatives.

In the proposed DDSS the aforementioned ingredients are represented graphically in the three-domain charts illustrated in Figures 4 and 5. There are great advantages to the graphical three-domain representation. It permits, for a potential design alternative, an instantaneous/simultaneous inspection of consequences (\$loss and collapse probability) at various hazard levels and an evaluation of the loss incurred in the individual subsystems. It permits, for a set of potential design alternatives, an instantaneous/simultaneous evaluation of the relative advantages and disadvantages of the alternatives, and in this manner greatly facilitates design decision making. The frequently asked and much debated questions about stiffness versus flexibility, and strength versus ductility can be addressed in the context of expected loss at various hazard levels and in the context of building occupancy, which will dictate the relative value of the nonstructural drift sensitive subsystem (NSDSS), the nonstructural acceleration sensitive subsystem (NSASS), and the structural subsystem (SS). The DDSS will facilitate the decision making for suitable structural systems based on subsystem values and functionality, which will be very different, for instance, for hospitals, office building, residential buildings, and museums.

It must be emphasized that the DDSS provides only mean estimates of losses and does not account for dispersions, except in the assessment of the probability of collapse. In the latter context, aleatory as well as epistemic uncertainties can be considered.

The process illustrated in Figures 4 and 5 can be used also for a quick performance assessment, avoiding the usually large analytical effort involved in accounting for uncertainties and their propagation from hazard and ground motion modeling all the way up to loss estimation and decision making.

The example illustrated here focuses on building level subsystems. This is done because of space limitations. The proposed DDSS can be implemented in the illustrated manner also for story level subsystems, which is a more accurate approach as it (a) permits consideration of different value of stories, and (b) utilizes story level *EDPs* in the Structural System sub-domain, which correlate much better with the damage in each story than do global *EDPs* that represent the system demands in an average sense. As always, more accuracy comes at the expense of more complexity, which is generated by looking at the building at a story-by-story level. But this more accurate approach may be the preferred one, as it points out that decisions on structural properties made at the system level may not be the best ones if a building is scrutinized more carefully at the story subsystem level [Zareian, 2006].[15]

The implementation illustrated here focuses on monetary loss and collapse safety. In concept it is equally applicable to downtime loss and casualties. The challenge will be to develop loss curves that apply to performance objectives based on these decision variables. Downtime loss was found to be probably the largest contributor to total loss in recent earthquakes in California, such as the Northridge earthquake. This type of loss will greatly impact large corporations and institutional owners concerned with a portfolio of buildings [Comerio, 2005].[3] Again, here it hardly will be possible to develop accurate loss curves, but approximate mean loss curves together with generic *IM-EDP* curves and site-specific hazard curves will greatly facilitate seismic risk management and design decision making.

Last but not least, advocates of innovative technologies, such as base isolation and other passive control systems, need a simple performance-based design/assessment tool to evaluate merits in terms of loss reduction versus increase in construction cost. The former can be achieved by means of the three-domain charts presented here, provided that advocates are willing to develop mean IM-EDP relationships and collapse fragility curves for structural systems that incorporate such innovative technologies. This is not a major effort.

ACKNOWLEDGEMENTS

This research was carried out as part of a comprehensive effort at Stanford's John A. Blume Earthquake Engineering Center to develop basic concepts for performance-based earthquake engineering and supporting data on seismic demands and capacities. This effort is supported by the NSF sponsored Pacific Earthquake Engineering Research (PEER) Center.

REFERENCES

1. Aslani, H., "Probabilistic earthquake loss estimation and loss deaggregation in buildings," *PhD. Dissertation* Department of Civil & Env. Engineering, Stanford University, Stanford, CA., 2005.
2. Baker, J, Cornell, CA., "Choice of a vector of ground motion

intensity measures for seismic demand hazard analysis," *Proc. 13th WCEE*, Vancouver, Canada, 2004, pp. 3384.

3. Comerio, M.C., "The influence of performance engineering on disaster recovery: priorities for limiting downtime," *International Symposium on Earthquake Engineering*, Japan Association for Earthquake Engineering, Awaji Island, January 13-16, 2005.
4. Cornell, CA, and Krawinkler, H., "Progress and challenges in seismic performance assessment," PEER News, April 2000.
5. Deierlein, G., "Overview of a comprehensive framework for earthquake performance assessment," *Proc. International Workshop on Performance-Based Seismic Design - Concepts and Implementation*, Bled, Slovenia, 2004.
6. Ibarra, L.F., and Krawinkler, H., "Global collapse of frame structures under seismic excitations," *John A. Blume Earthquake Engineering Center Report No. TR 152*, Department of Civil Engineering, Stanford University, and PEER Report 2005/06, 2005.
7. Ibarra, L.F., Medina, R.A., and Krawinkler, H., "Hysteretic models that incorporate strength and stiffness deterioration," *International Journal for Earthquake Engineering and Structural Dynamics*, Vol. 34, No. 12, 2005, pp. 1489-1511.
8. Krawinkler, H., "A general approach to seismic performance assessment," *Proc. International Conference on Advances and New Challenges in Earthquake Engineering Research*, HongKong, ICANCEER 2002, Vol. 3, 2002, pp. 173-180.
9. Krawinkler, H, and Miranda, E., "Performance-based earthquake engineering," *Earthquake Engineering: from engineering seismology to performance-based engineering*, Bozorgnia Y, Bertero VV (eds); CRC Press: Boca Raton; 2004, Chapter 9-1 to 9-59.
10. Krawinkler, H., Zareian, F., Medina, RA., and Ibarra, LF., "Decision support for conceptual performance-based design," *Earthquake Engineering and Structural Dynamics*, Vol. 35, No. 1, 2005, pp. 115-133.
11. Krawinkler, H., Editor, "Van Nuys hotel building testbed report: exercising seismic performance assessment", PEER Report 2005/11, 2005.
12. Medina, R., and Krawinkler, H., "Seismic demands for nondeteriorating frame structures and their dependence on ground motions," *John A. Blume Earthquake Engineering Center Report No. TR 144*, Department of Civil Engineering, Stanford University, 2003, and PEER Report 2003/15, May 2004.
13. Taghavi, S., and Miranda, E., "Response assessment of nonstructural building elements," *Pacific Earthquake Engineering Research Center*, Report No. 2003/05, UC Berkeley, California, 2003.
14. Vamvatsikon, D., and Cornell, CA., "Incremental Dynamic Analysis," *Earthquake Engineering and Structural Dynamics*, Vol. 31, no. 3, 2002, pp. 491-514.
15. Zareian, F., "Simplified performance-based earthquake engineering," *Ph.D. dissertation*, Department of Civil and Environmental Engineering, Stanford University, Stanford, CA, USA., 2006.