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A two-dimensional framework for performance-based seismic design

Stojadinović, Božidar [1] Collins, Kevin R.[2]

ABSTRACT

Performance-based seismic design is based on balancing the risk a structure is exposed to and the performance the owners of the structure expect. This design approach is radically different from the design approaches used to-date. Instead of clear-cut capacity–demand comparisons of design quantities at the level of structural elements, a designer must compare measures of seismic hazard and measures of seismic performance of the entire structure across a continuous range of performance objectives prescribed by the codes. Several conceptual frameworks for performance-based design have already been proposed. These frameworks prescribe qualitative measures of seismic hazard and qualitative measures of seismic performance. However, development of practical performance-based design provisions is impeded by two major obstacles: 1) the lack of quantifiable measures of the influence of local design decisions on global performance; and 2) the lack of efficient probabilistic treatment of seismic hazard exposure and structural performance.

In this paper we present a new conceptual framework for performance-based design. This framework is characterized by: a two-dimensional space of design parameters that allows for direct capacity–demand comparison of the seismic performance objective and the seismic capacity of a structure; a definition of performance objectives based on probabilistic measures of seismic hazard; and a definition of the global structural capacity based on quantifiable measures of the quality of design. The proposed framework is based on an equivalent single-degree-of-freedom idealization of a structure. To illustrate the framework, a sample preliminary performance-based design of a steel moment-resisting frame will be used.

PRELIMINARY PERFORMANCE-BASED DESIGN

A performance-based design procedure begins by selecting performance objectives and identifying seismic hazards, continues with conceptual, preliminary and final designs followed by design acceptability checks and design review, and concludes with quality assurance during construction and building maintenance after construction [SEAOC 95]. The first engineering considerations of the proposed conceptual design are done in the preliminary design phase. The main goal of a preliminary design procedure is to formulate a design that is as close as possible to the desired final design of the structure.

Preliminary design is usually an iterative procedure. Fro displacement-based design, the key design information is a limiting value of the fundamental period of the structure: a structure designed to have a smaller fundamental period (i.e. a larger stiffness) will satisfy the chosen performance objective with a desired level of reliability. In broader scope, this maximum period limit is intended to insure that the capacity of the structure in an earthquake will be larger than the imposed seismic demand. To do a preliminary design a designer needs to consider: 1) the probabilistic nature of seismic hazard exposure; 2) the non-deterministic notion of acceptable behavior; and 3) the effect of local design decisions and global proportions of the structure on its performance. A demand/capacity design procedure formulated in a two-dimensional performance-based design space may make preliminary design much simpler.

[1] Assistant Professor, Department of Civil and Environmental Engineering, 2380 G. G.Brown Building, University of Michigan, Ann Arbor, MI, 48109-2125, phone: 734-764-6576, fax: 734-764-4292, e-mail: boza@engin.umich.edu

[2] Assistant Professor, Department of Civil and Environmental Engineering, 2374 G. G.Brown Building, University of Michigan, Ann Arbor, MI, 48109-2125, phone: 734-764-5617, fax: 734-764-4292, e-mail: kevinrc@engin.umich.edu

PERFORMANCE-BASED DESIGN SPACE

Preliminary performance-based design of a structure can be performed in a two-dimensional design space. One dimension of this design space is a measure of seismic hazard. The other dimension of the design space is a measure of structural performance. Design demand, defined by performance objectives, and design capacity, describing the response of the structure, can be simultaneously represented in this space. The relation between demand and capacity is established using the equivalent single-degree-of-freedom (SDOF) system concept.

Performance Objective Line

Uniform hazard spectra provide a convenient means of accounting for the probabilistic nature of seismic hazard exposure in the design process. The concept of a uniform hazard spectrum is similar to that of a typical response spectrum. The difference is that each ordinate of a uniform hazard spectrum curve has the same probability of exceedance. Examples of uniform hazard spectra are presented in [Collins 95, Collins 96].

Figure 1 shows the general trend of uniform hazard spectra for displacement response of a bi-linear SDOF system. Given a period of the SDOF oscillator T_1, the uniform hazard spectral ordinates determine the intervals of displacement values whose exceedance can be subjectively described as "frequent", "occasional", "rare", or "very rare". The length of exceedance intervals depends on the period of the SDOF system. Figure 1 compares the displacement exceedance intervals for a SDOF system with a period T_2 larger than T_1. Evidently, an increase in period makes exceedance intervals longer. The length of displacement exceedance intervals may, also, depend on the hysteresis properties of the SDOF system. However, considering the results presented by Fajfar [Fajfar 94], variation of hysteresis properties should not have a significant effect on the shape of uniform hazard spectra and the length of exceedance intervals.

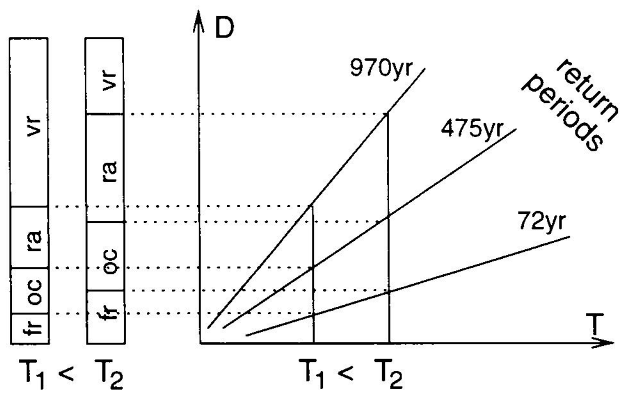

Figure 1: Relation between uniform hazard spectra and subjective description of hazard exposure. Legend: **fr**–frequent; **oc**–occasional; **ra**–rare; **vr**–very rare.

Measures of performance are intended to inform a user or an owner of the building about the consequences of an earthquake. As such, performance measures have to be defined for the structural system, the non-structural elements, and the content of the structure [Krawinkler 97]. Structural system performance can be quantified using a combined displacement ductility and hysteretic energy index, such as the well-known Park-Ang damage index. Performance of non-structural elements can be measured using damage indices related to interstory drift for shear buildings, or to column elongation for cantilever buildings [Krawinkler 97]. Performance of the content can be quantified using a damage index based on floor acceleration levels [Krawinkler 97].

The use of a damage index concept provides an opportunity to define the notion of acceptable structural performance. A set of graduated levels of performance, qualitatively described as "fully operational", "operational", "life safe", or "near collapse", can be associated with intervals of damage index values [SEAOC 95]. The lengths of damage index intervals depend on both the fundamental period and the hysteresis properties of the structure. Although this dependence has not been explicitly investigated, Mazzolani and co-workers [Mazzolani 97] suggest that the influence of the period may be stronger than the influence of the hysteresis properties.

The acceptable combinations of seismic hazard exposure and structural performance are defined in codes. These combinations constitute a set of performance objectives. This set of performance objectives can be represented in two-dimensional performance-based design space (Figure 2). The discrete nature of recurrence intervals and performance levels results in a checkerboard structure of the design space. A step-wise line, the performance objective line, herein referred to as the PO-line, divides the design space in two regions. A point representing an acceptable design has to be at or below the PO-line. Thus, the PO-line defines the performance-based design demand.

Location of the PO-line strongly depends on the fundamental period of the structure. Figure 2 shows how the PO-line shifts with the increase of period. This shift occurs because of the elongation of the displacement exceedance intervals. Note that the increase of the fundamental period reduces the acceptable design region, shifting the PO lines to the right. Intuitively, this is what a designer expects: it is more difficult to satisfy displacement-based performance criteria for a longer period (softer) structure.

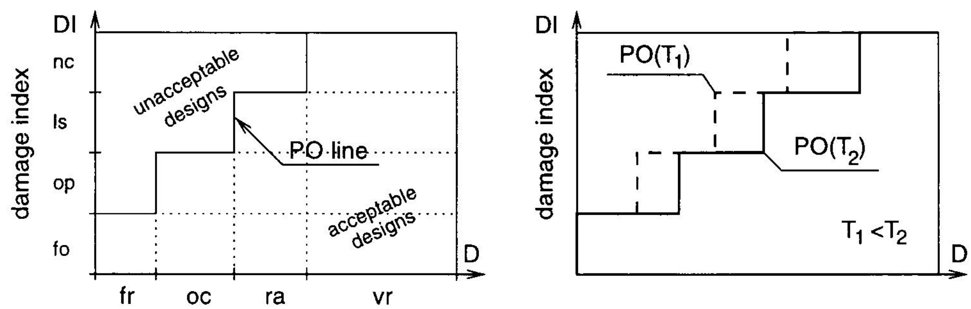

Figure 2: Performance objective lines in the design space. Legend: **fo**–fully operational; **op**–operational; **ls**–life safe; **nc**–near collapse.

Response Curve

A structure responds to seismic excitation in a complex manner. An equivalent SDOF system can be used to simplify the response of the structure for design purposes [Collins 95]. Properties of the equivalent SDOF system are derived from the response of the MDOF cantilever-type structure subjected to horizontal base motion and information about the deformed shape of the structure, obtained from inelastic static push-over analyses. Similar kinematic transformations can be used to relate a damage index measure of local (element or connection) performance to a global measure of performance of an equivalent SDOF system.

The response of a structure can be represented in the two-dimensional performance-based design space. A response point is defined by the magnitude of the equivalent SDOF system displacement which corresponds to a level of seismic hazard exposure and a damage index value that measures the performance of the system. A set of such points (assuming sufficient smoothness) form a curve in the two-dimensional design space. This is the response curve for a structure. The shape and the location of the response curve (R-curve hereafter) strongly depends on the quality of local detailing of structural elements and connections, and on the quality of global proportioning of the structure. This is demonstrated for a generic steel moment-resisting frame shown in Figure 3.

Response Curve for a Steel Moment-Resisting Frame

The 1994 Northridge and 1995 Hoygo-Ken Nanbu (Kobe) earthquakes conclusively demonstrated that the behavior of the beam-to-column connection governs the response of steel moment-resisting frames. The performance of a typical beam-to-column connection can be described using a simple damage index (Figure 3). This damage index is defined using the connection moment-rotation response. Damage is considered negligible as long as the actual connection rotation is smaller than its yield rotation ϕ_y. The connection may fail in a ductile manner when its rotation exceeds the ultimate rotation ϕ_u, or in a brittle manner at fracture rotation ϕ_f. Damage index values are assumed to vary linearly in proportion to the connection rotation ϕ between ϕ_y and ϕ_u. Even though this is a very simple deformation-based local damage index, it quantifies the local performance of the connection very well.

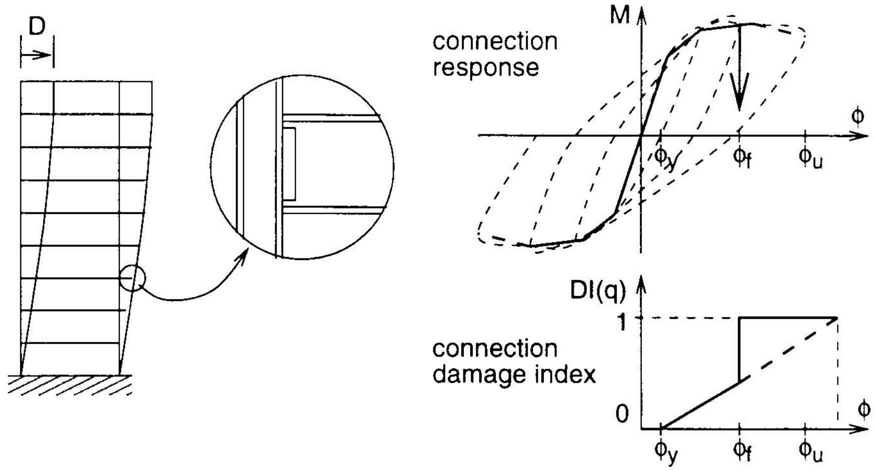

Figure 3: Connection damage index.

The link between the global and the local behavior of the frame is derived using a kinematic relation between the connection rotation ϕ and the roof displacement D. In its ultimate, pre-collapse state, the frame forms a plastic mechanism described by parameters α and β (Figure 4). Parameter α measures the portion of the structure involved in the mechanism. A value of α close to unity means that the structure is forming a complete weak-beam/strong-column plastic mechanism. Conversely, formation of a soft-story-like mechanism implies α is small. Parameter β describes the amplification of inter-story drifts caused by higher mode effects. A value of β close to unity means the deformed shape is linear. Practical values of β rarely exceed two [Collins 95]. This simple, yet realistic, deformation mechanism results in a linear kinematic relation between roof displacement D and connection rotation ϕ (Figure 4).

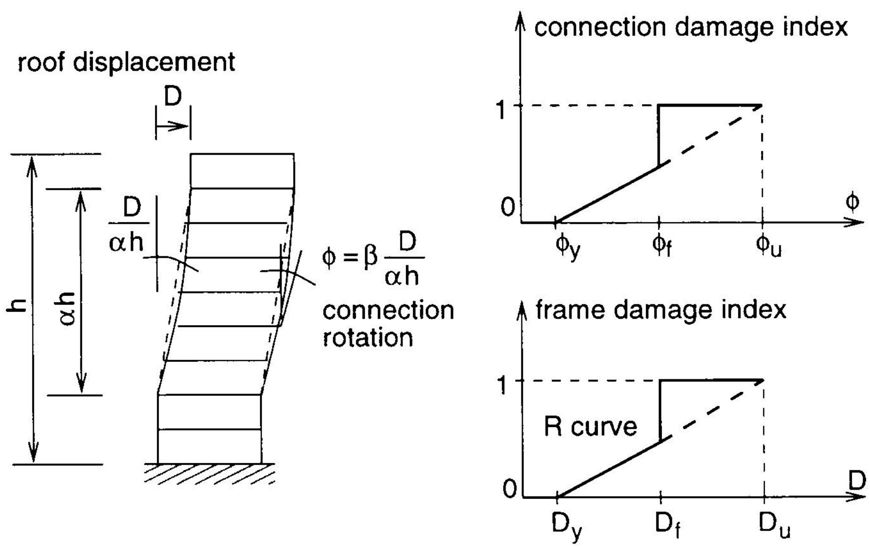

Figure 4: R-curve for the steel moment-resisting frame.

Derivation of the R-curve for the steel moment frame is now simple. The kinematic relation is used to compute the yield (D_y), fracture (D_f) and ultimate (D_u) frame roof displacements, corresponding to the yield, fracture and ultimate rotation of the most exposed connection. Global damage index of the frame is assumed to be the same as the local damage index of the most exposed connection. The frame R-curve describes the response of the frame in terms of the degree of damage the frame incurs at various levels of seismic hazard exposure. If there are no connection fractures, the R-curve is a straight line. If a connection fractures, the R-curve becomes piece-wise linear.

The values of parameters α and β depend strongly on the global proportions of the structure. Proportions of the structure broadly describe its geometry, distribution of masses and stiffnesses, number of stories and bays, ratio of story height and bay width, and ratio of beam and column strengths. The values of yield, ultimate and fracture rotations (ϕ_y, ϕ_u and ϕ_u) depend strongly on the detailing of the beam-to-column connection. Detailing data comprises parameters such as the welding procedure, the type of electrode, the relative thicknesses of beam and column flanges, the slenderness of the beam flanges and the beam web, and lateral bracing provided for the beam bottom flange in the plastic hinge zone. Therefore, the response of the frame depends on the quality of local detailing and global proportioning of the frame. Figure 5(a) shows how the R-curve for the frame shifts with worsening quality of global proportioning. Formation of the soft story reduces α and increases the connection rotation demand to cause a quicker deterioration of the structural system response. Similarly, poor connection details reduce the fracture rotation to a value slightly larger than the yield rotation, and seriously compromise the performance of the structure (Figure 5(b)). Note that deterioration of design quality shifts the R-curves to the left side of the design space.

The fundamental period of the building also effects the values of the system proportioning parameters α and β. However, research conducted by Miranda [Miranda 97] and Mazzolani [Mazzolani 97] suggests this effect is not very strong. This is because the effect of the building period on the response of the beam-to-column connection may be related to the strain rate in the connection. Tests conducted in Japan [Kinki 97] show that the effect of the strain rate on connection rotation capacity is small.

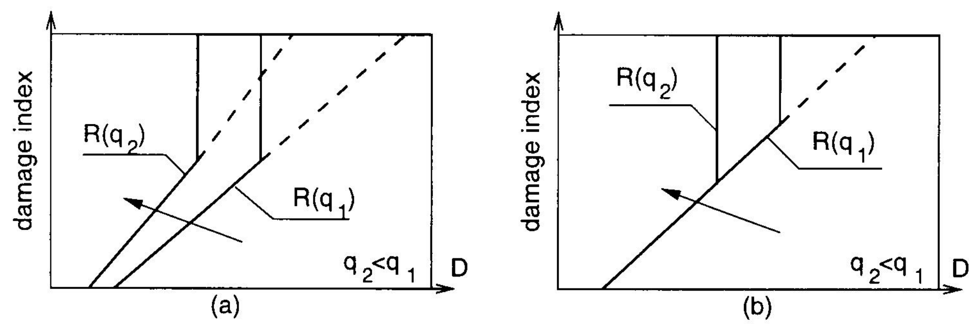

Figure 5: Shift of the R-curve with decrease in design quality.

Design Procedure

The goal of performance-based seismic design is to produce a structure that satisfies the performance objectives. This design goal has a simple geometric interpretation in the two-dimensional performance-based design space: an acceptable design is one whose response (R) curve is entirely inside the acceptable design region defined by the performance objective (PO) line. The intersection of the PO-line and the R-curve represents a design point (Figure 6(a)). This point directly defines both the largest fundamental period (T_{max}) and the necessary design quality (q) for the structure. A structure with such characteristics will perform in a desired manner across the entire range of seismic hazards it is exposed to.

The structure of the performance-based design space provides additional design information. The directions of shifts of the PO lines and R-curves give designers a clear picture of how changes in fundamental period and design quality influence the performance (Figure 6(b)). For example, in order to change an unacceptable design of the frame, a designer may: 1) increase the acceptable design region by making the structure stiffer, thereby reducing its fundamental period T; 2) shift the response curve towards the acceptable design region by improving the design quality, thereby making the structure tougher; or 3) do both simultaneously. A simple graphical interpretation of these design options will reduce the number of preliminary design iterations.

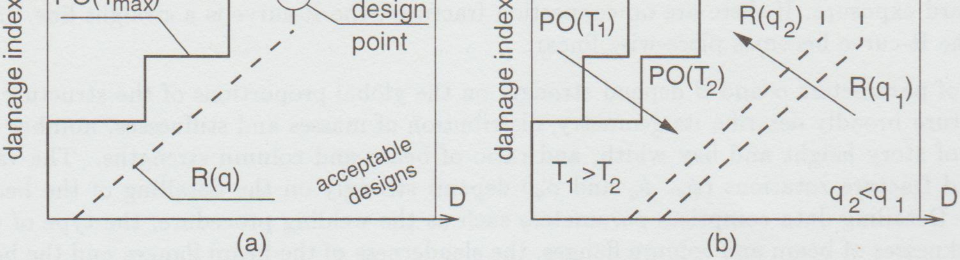

Figure 6: PO-lines and R-curves interact in the same design space.

CONCLUSION

structure of the two-dimensional design space we developed is similar to the two-dimensional performance ctive definition presented in the Vision 2000 document. The main advantage of the proposed two-ensional design framework is a clear, graphical, method that allows designers to apply the customary city–demand concepts in a performance-based design setting. In addition, our performance-based design ework advances three steps forward: 1) it directly quantifies the effect of design quality on seismic ormance; 2) it seamlessly integrates probabilistic measures of seismic hazard exposure; and 3) it allows se of reliability-based design methods to ensure that a structure has at least a minimum level of reliability.

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