CAPACITY DESIGN IN SEISMIC RESISTANT STEEL BUILDINGS
A Reliability-Based Methodology to Establish Capacity-Design Factors

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INTRODUCTION

Capacity design has been employed in engineered systems for many years. The basic concept behind capacity design is to control the modes of structural behavior by protecting certain critical structural components from failing. In seismic resistant structural steel buildings, so-called "deformation controlled" or "fuse" components are designed to yield and dissipate energy, and in the process to shield other "force controlled" or "non-ductile" components from excessive force or deformation demands. Capacity design principles aim to ensure this desirable behavior by designing the members and connections adjacent to the fuse member to be stronger than the fuse itself. Capacity-design factors, such as component or system overstrength parameters, are used by structural codes to help ensure a low probability of failure for the critical component. These factors typically include a combination of "amplification" or "overstrength" factors to increase the required strength of the critical component and "resistance" factors to reduce its design strength. While the basic concept of capacity design is straight forward there are inconsistencies in the way capacity design factors are established for different seismic resisting systems. Rational development of capacity design requires consideration of many factors related to the variability in earthquake ground motions, component strengths and overall inelastic system response. The main objective of this paper is to identify key factors affecting the reliability of capacity designed components and to describe a proposed reliability-based methodology to guide selection of capacity design factors for seismic design.

1 CAPACITY DESIGN IN SEISMIC RESISTANT STEEL BUILDINGS

The AISC *Seismic Provisions for Structural Steel Buildings* (AISC 2010), and modern building codes in general, employ capacity design principles to help ensure ductile response and energy dissipation capacity in seismic resisting systems. Given the desired mode of behavior, the design provisions are devised to confine significant inelastic deformations to those structural components that have been appropriately designed. Other components, which have not been designed to sustain inelastic action, are designed with sufficient strength to remain essentially elastic. Following the terminology in FEMA 356 (2000) and ASCE 41 (2007), components that are designed to sustain inelastic displacements are referred to as "deformation controlled", and other components designed to remain essentially elastic are referred to as "strength controlled". Strictly speaking, the distinctions between deformation and strength controlled components are not absolute, but the terms are introduced for convenience in discussions of capacity design requirements.

An example of capacity design is brace connections in Special Concentrically Braced Frames (SCBFs) where the current design requirements for brace connections in SCBF imply that the connections should have sufficient strength to develop the *yield strength* of the braces. In a reliability context, this requirement could be phrased as "the probability is low that the connection fails prior to brace yielding". Mathematically, this requirement would be described by evaluating and sufficiently limiting the probability of connection failure prior to brace yielding. This assessment implies that (a) the desired limit state criterion is known, and (b) the acceptable failure rate is defined. These assumptions depend on how the component behavior relates to the overall system response. For example, as illustrated in *Fig. 1*, a simple criterion for brace connections is that the connection components be designed with sufficient strength to develop the yield strength of the brace, considering the influence of strain hardening at large deformations. Additionally, the acceptable failure probability of the strength controlled component should depend on how critical the individual component, in this case a brace connection, is to the overall system reliability.

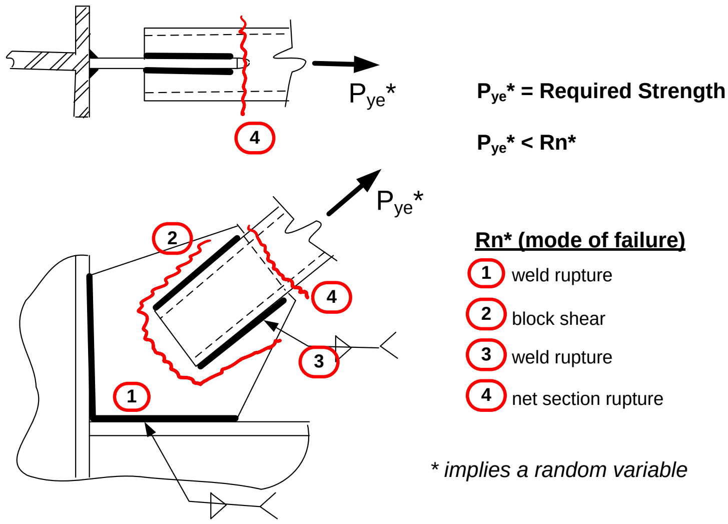
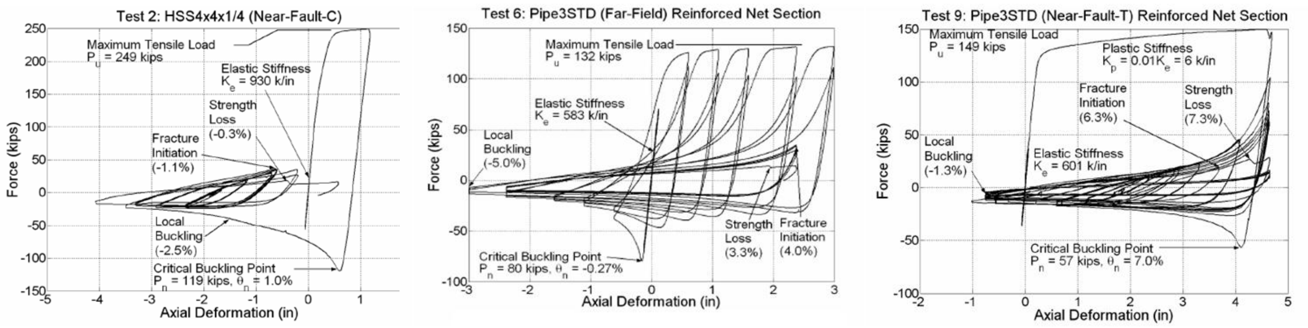


Fig. 1. Braced Frame Connection Detail



a) b) c)

Fig. 2. Brace response during a) Near-fault compression loading b) Far-field loading c) Near-fault tension loading (Images from Fell et al. (2006))

The loading demand in capacity-based design is unique to other design concepts in the sense that the demand on the strength controlled components originates mainly from other components within the system, i.e. from the deformation controlled components as they undergo inelastic deformation during seismic events. *Figure 2* shows brace responses when braces are subjected cyclic loading under various loading protocols. Regardless of the loading protocol, i.e. near-field compression, far-field or near-field tension, the responses all share a common characteristic, i.e. the braces yield in tension at relatively low deformations. This results in brace connections experiencing demands close to their maximum tensile demands at relatively low deformations, deformations that are likely to occur under low to moderate earthquake intensities. *Figure 3* demonstrates this point further, where the incremental dynamic analysis results are shown for a single story SCBF. The maximum brace forces and story drifts are plotted versus earthquake intensity in *Figs. 3b* and *3c*, respectively. The probability of connection failure is calculated by comparing the brace force demands in *Fig. 3b* to the connection strength. As shown in *Fig. 3d*, the failure probability is negligible before brace yielding and then saturates as the braces reach their maximum capacities. Predicting the ground motion intensity causing initiation of brace yielding is, therefore, pivotal in establishing the connection reliability as it will indicate the probability of connections experiencing large demands.

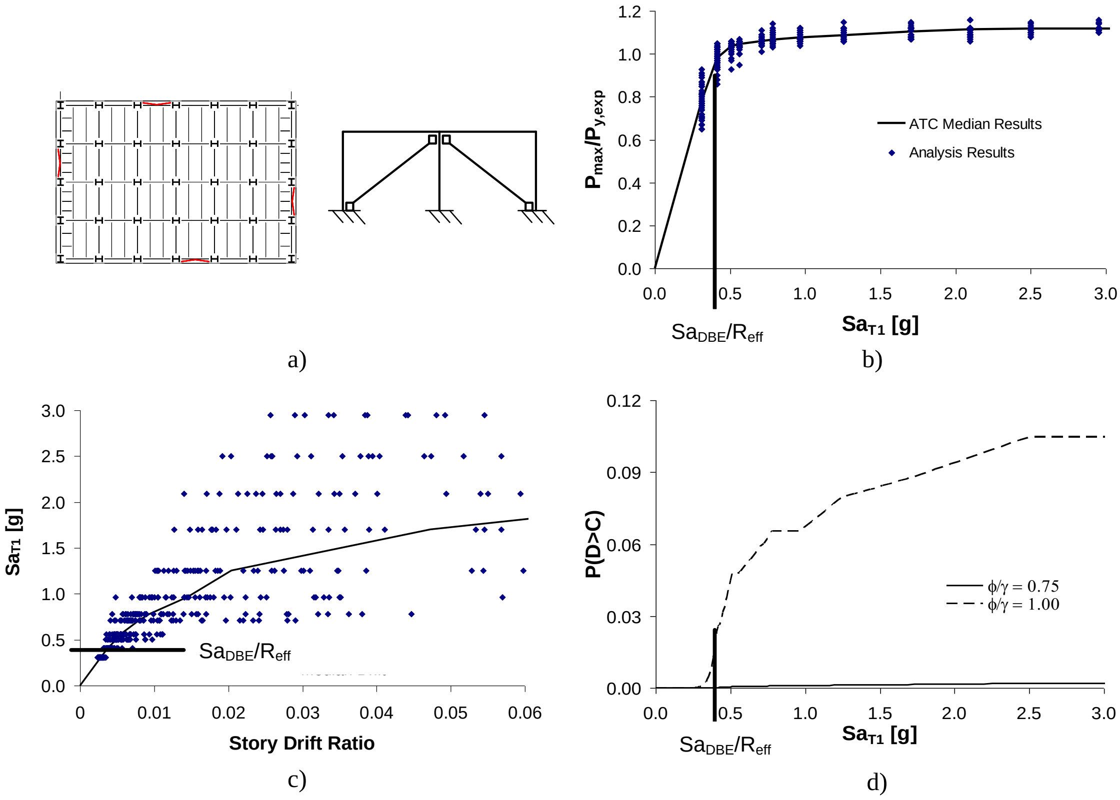


Fig. 3. Results from IDA study on a single story SCBF a) Plan and elevation of frame analyzed. b) Maximum brace forces, P_{max}, recorded in each analysis normalized by their expected yield strength, $P_{y,exp}$. c) Maximum story drift ratio recorded in each analysis. d) Probability of connection failure vs. spectral acceleration for a given connection capacity and dispersion

When designing deformation controlled components, their required elastic strength is based on the seismic design forces of the overall frame. Following the *ASCE 7* loading standard, the seismic design forces are based on the spectral acceleration for the maximum considered earthquake (MCE) at the fundamental period of the structure, Sa_{T1}. This spectral intensity is then (a) multiplied by two-thirds to the design bases earthquake (DBE) intensity and (b) divided by the frame's response modification coefficient, or R-factor (equal to R=6 for SCBFs). Thus, the initiation of yielding in the deformation controlled braces typically occurs at earthquake intensities significantly lower than the design basis earthquake. Due to the use of capacity-design factors (γ and ϕ), the ratio between nominal specified versus median material and member strengths, and member oversizing, the effective R (R_{eff}) is smaller than the code specified R-factors (see *Fig. 4*). This implies that the spectral acceleration at which yielding initiatves, $Sa_{y,exp}$, will always be larger than the Sa_{design}. Knowing R_{eff} , then $Sa_{y,exp}$ can be predicted with *Eq. 1*.

$$Sa_{y,\exp} = \frac{{}^{2}\!/_{3}\, Sa_{MCE}}{R_{eff}} \geq \frac{{}^{2}\!/_{3}\, Sa_{MCE}}{R} = Sa_{design} \qquad (1)$$

Knowing $Sa_{y,exp}$, the site ground motion hazard curve can be used to predict the probability (frequency) that the strength controlled components will experience large forces and cause the deformation controlled components to yield (see *Fig. 5*).

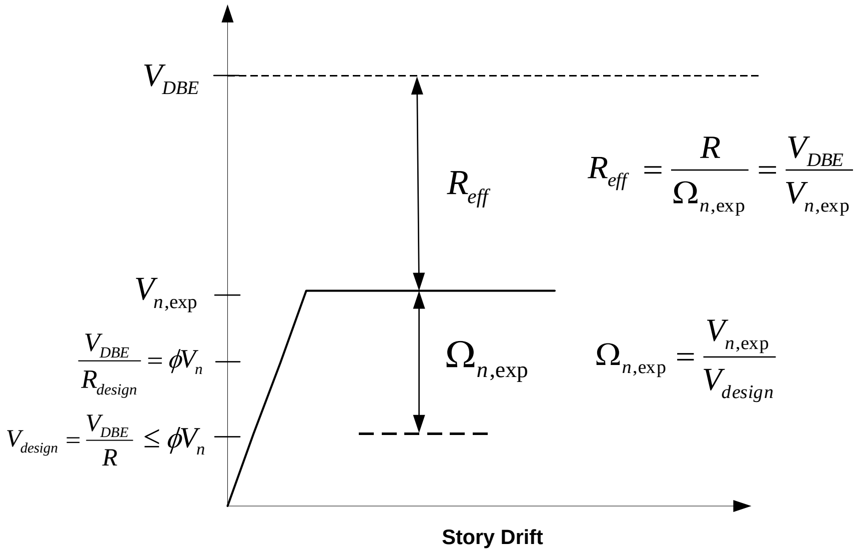


Fig. 4. Idealized static nonlinear response (pushover curve) comparing the design base shear V_{design} to the expected base shear strength $V_{n,exp}$ and the ratios $\Omega_{n,exp}$ and R_{eff} between design, expected and elastic design basis earthquake base shears.

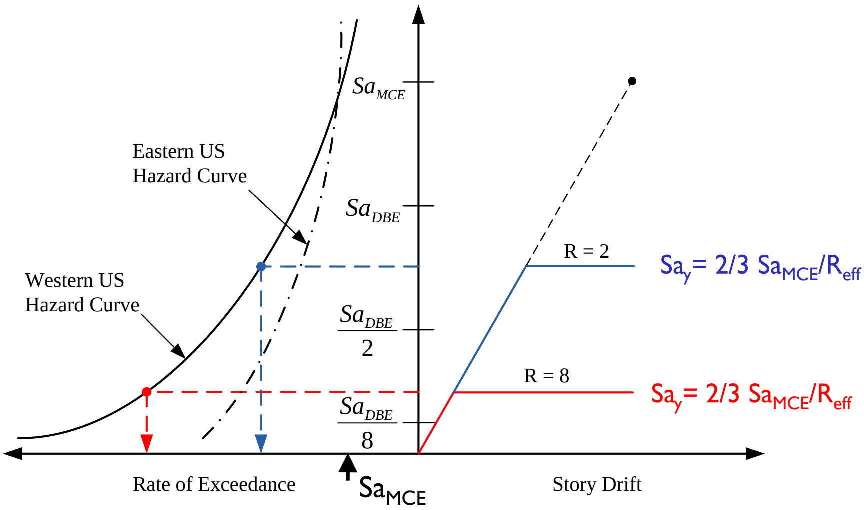


Fig. 5. Relationship of the site ground motion hazard curve (left) to the static nonlinear response curves (right) to illustrate the rate of exceedence of the spectral acceleration corresponding to yield in the structure. Characteristic hazard curves are shown for the eastern and western United States, and response curves are shown for structures designed with two R-values (2 and 8).

2 RELIABILITY-BASED METHOD TO ESTABLISH CAPACITY-DESIGN FACTORS

The proposed reliability-based methodology for establishing capacity design factors for components in seismic resistant structural steel buildings utilizes the well-established LRFD component reliability methodology with apt adjustments to address the issues specific to capacity-based design. The basic equation to determine capacity design factors is:

$$\frac{\gamma}{\phi} = \frac{D_m}{D_n} \frac{C_n}{C_m} \exp\left(\beta_{R,Ha} \sqrt{V_C^2 + V_D^2 - \rho V_C V_D}\right) \tag{2}$$

where D_m and C_m, and D_n and C_n are the median and nominal values of the demand and capacity probability distributions, respectively, V_D and V_C their lognormal standard deviations, and ρ the correlation between demand and capacity. The reliability index, $\beta_{R,Ha}$, provides a measure of probability of demand exceeding capacity of capacity-designed components, for a specified pair of γ and ϕ. The key differences between the this methodology and LRFD is in the way the those two factors, i.e. reliability index, $\beta_{R,Ha}$, and the demand parameters, D_m and V_D, are selected. The demand in capacity design is based on the capacity of the deformation controlled components, as opposed to more traditional loading such as dead, live and wind loads, and it will vary significantly depending on the deformation demands in the structure. The reliability index in the LRFD methodology was originally calibrated with pre-LRFD design equations and then used as a comparative value for different failure modes. The reliability index, $\beta_{R,Ha}$, in *Eq. 2* serves a similar purpose but takes system effects, i.e. the frame's response modification coefficient, or R-factor, member overstrength and the frame's site ground motion hazard curve, into consideration such that different failure modes in different systems have a consistent reliability.

To determine an appropriate value of $\beta_{R,Ha}$, advantage is taken of results such as in *Fig. 3d* where the probability of demand exceeding capacity is a function of the spectral acceleration, Sa. The site ground motion hazard curve (which provides frequencies of exceedance of each *Sa*) can be combined with the probability of demand exceeding capacity for a given *Sa*, to compute the mean annual frequency, MAF, of demand on strength controlled components exceeding their capacity.

$$MAF(D > C) = \sum_{Sa} P(D > C | Sa = x) * MAF(Sa = x) \quad (3)$$

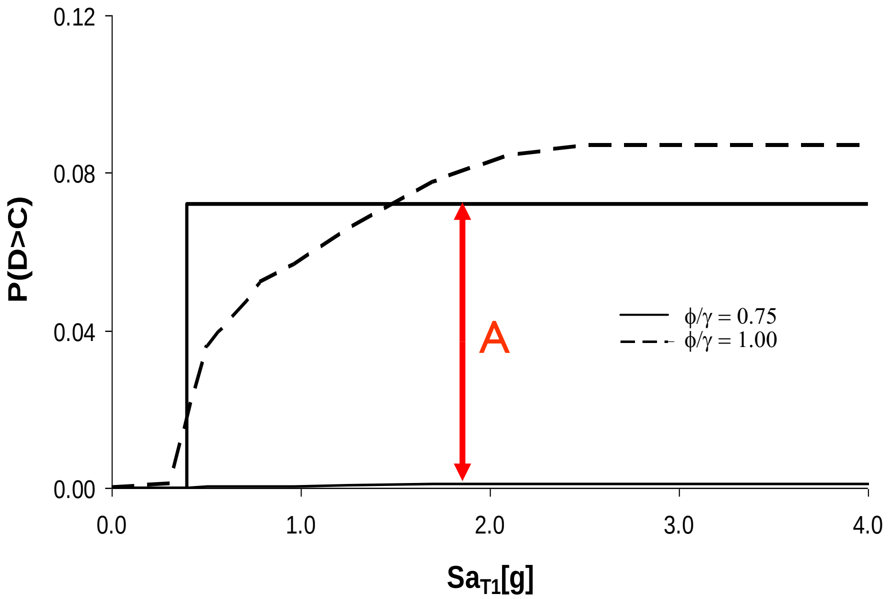


Fig. 4. Probability of imposed demand on a component exceeding its capacity as a function of ground motion intensity. The curvilinear $P(D>C|Sa)$ function is approximated by the step function with probability A.

The summation in *Eq. 3* can be avoided by simplifying the probability of demand exceeding capacity function to a step function where the probability is zero when $Sa < Sa_{y,exp}$ and constant (equal to A in *Fig. 4*) when $Sa > Sa_{y,exp}$. This approximation allows for simply multiplying the non-zero constant with the mean annual frequency of $Sa_{y,exp}$ being exceeded, $MAF(Sa > Sa_{y,exp})$, to calculate the mean annual frequency of demand exceeding capacity (obtained from the seismic hazard curve). Thus, *Eq. 3* reduces to the following,

$$MAF(D > C) \cong P(D > C | Sa > Sa_{y,\exp}) * MAF\left(Sa > Sa_{y,\exp}\right) \quad (4)$$

where

$$P(D > C|Sa) = \begin{cases} 0 & if \quad Sa \le Sa_{y,\exp} \\ A & if \quad Sa > Sa_{y,\exp} \end{cases} \quad (5)$$

For a tolerable mean annual frequencies of demand exceeding capacity and the mean annual frequency of $Sa > Sa_{y,exp}$, *Eq. 4* can be rearranged to give the probability of post-yielding demand exceeding capacity can be calculated as follows,

$$P(D > C|Sa > Sa_{y,\exp}) \cong \frac{MAF(D > C)}{MAF(Sa > Sa_{y,\exp})} = \Phi(\beta_{R,Ha}) \quad (6)$$

From *Eq. 6,* the reliability index, $\beta_{R,Ha}$, necessary for *Eq. 2* can then be calculated using the inverse standard normal cumulative distribution function, i.e.,

$$\beta_{R,Ha} = \Phi^{-1}\left(\frac{MAF(D > C)}{MAF(Sa > Sa_{y,\exp})}\right) \quad (7)$$

For a specified mean annual frequency of demand exceeding capacity, along with a hazard curve and system yield strength, the $\beta_{R,Ha}$ from *Eq. 7* can be used together with the statistical demand and capacity factors in *Eq. 2* to calculate the required ϕ/γ ratio for capacity design. In ongoing research, the authors are investigating appropriate (target) mean annual frequencies of component failure, *MAF(D>C),* that are consistent with the overall seismic system reliability implied in building codes.

3 SUMMARY AND ACKNOWLEDGMENT

The proposed reliability-based methodology to establish capacity design requirements incorporates the main factors believed to influence the reliability of capacity-designed components. The methodology takes into consideration, the response modification factors, R-factors, member overstrengths and site seismic hazard curves. Ultimately, the methodology will enable the calculation of risk consistent capacity-designed components for different structural components and systems. An important aspect of the methodology is that it allows for relaxing requirements on capacity-designed components in buildings where deformation demands are low, i.e. R_{eff} is low, and/or for buildings where the probability of experiencing earthquakes large enough to cause inelastic deformation is low.

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