Recent Developments on the Design and Evaluation of Steel Structures for Seismic Loads: An NSF Perspective

by

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ABSTRACT

There are many sources of uncertainty when designing or evaluating new buildings for seismic loads. Current codes account for these uncertainties in an ad hoc fashion. A new procedure has been developed for design and evaluation of steel moment frame buildings in the U.S. Randomness and uncertainty are accounted for in a rational way. The procedure is performance based and considers Immediate Occupancy and Collapse Prevention performance limits. The goal is to achieve a 90% confidence that the Performance Objective will be achieved under the design hazard level. A large number of design and evaluation examples were investigated. The results indicated that new buildings designed in accordance with the latest IBC specifications are conservative when collapse is considered. The reliability of older buildings, however, is not expected to have the same level of resistance against collapse.

KEYWORDS: Steel Moment Frame Buildings; Reliability-Based Design; Seismic Loads; Performance-Based Design

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1.0 INTRODUCTION

Prediction of seismic response of a new or existing structure is complex, due not only to the large number of factors that need to be considered and the complexity of seismic response, but also due to the large inherent uncertainty associated with making these predictions. Clearly the characteristics of future earthquakes can only be approximated leading to very large uncertainties in the loads acting on the structure. Structural properties may differ from those intended or assumed by the designer, or may change substantially during the earthquake (e.g. local fracture of connections). Analysis methods may not accurately capture the actual behavior due to simplifications in the analysis procedure (linear vs. nonlinear for instance) and modeling of the structure. Our knowledge of the behavior of structures during earthquakes is not complete which introduces other uncertainties. Consequently, seismic performance prediction must consider these uncertainties.

Many of these issues are covered to a greater or lesser extent in current codes through the use of load and resistance factors, adjustment of various design parameters following major earthquakes and introduction of new analytical and design procedures as they are developed and verified. In responding to the problems observed in steel moment frame buildings after the Northridge and other earthquakes, the SAC steel project has attempted to develop a comprehensive understanding of the capacity of various moment resisting connections and the demands on these connections. To achieve satisfactory building performance through design, or to evaluate an existing building, one needs to reconcile expected seismic demands with acceptable performance levels while recognizing the uncertainties involved.

Seismic design regulations for new buildings in the United States rely on a very simplified approach for achieving acceptable performance.

This approach uses several coefficients that may be used in different combinations to satisfy performance goals and objectives. These are the *R* factors, C_d values, drift limits and importance, or occupancy factors. By varying these coefficients several of the factors described above may be accounted for in a crude way. One problem with the current approach is that there has been no rational or quantifiable way to determine these coefficients.

During the 1994 Northridge Earthquake, fractured or cracked beam-column connections were discovered in over 200 steel moment frame buildings. This was a shock to many in the design profession, since steel frame buildings were considered to be the best seismic resisting system. As a result, the Federal Emergency Management Agency (FEMA) sponsored a large research-development project (SAC) that resulted in three reports (FEMA 350, (2000); FEMA 351 (2000); FEMA 351, (2000)). These documents presented a new paradigm for the design and evaluation of steel moment frame buildings. The new procedures allowed for the explicit evaluations of randomness and uncertainties to be included in the design process in a rational manner. The procedure forms the basis for a performance based engineering methodology that recognizes two performance levels, Immediate Occupancy and Collapse Prevention. One main feature of the procedure is that the design professional may calculate the confidence level is satisfying the design objective.

2.0 DEVELOPMENT

The new procedure can be used for new design, evaluation of existing buildings and evaluation of damaged buildings after an earthquake. It is a performance based procedure with two limit states considered, Collapse Prevention and Immediate Occupancy. This paper will deal only with Collapse Prevention. The design object is to have 90% confidence that the chance of not satisfying limit state is less than 2% in 50 (2/50) years. The seismic hazard level for the performance limit is also chosen to be 2/50.

The acceptance criterion is based on a confidence factor, λ, that is used to determine the confidence level. This factor is the ratio of the factored demand over factored capacity. In equation form, this is expressed as:

$$\lambda = \frac{\gamma \cdot \gamma_a \cdot D}{\phi \cdot C} \qquad (1)$$

where

D	=	estimate of median drift demand
C	=	estimate of median drift capacity
ϕ	=	resistance factor
γ	=	demand factor
γ_a	=	analysis demand factor

The factors, ϕ, γ and γ_a in Equation 15 are based on the reliability work developed by Jalayer and Cornell (1998) for the SAC project. A more detailed derivation of these equations is given by Cornell et al (2002). Equation 1 is essentially the ratio of factored demand divided by factored capacity. The demand, D, is the expected median drift resulting from a series of accelerograms sampled from the chosen hazard level. Details on how to calculate all of the variables for this procedure is given in Yun and Foutch (2000).

The resistance factor, ϕ, accounts for the fact that the estimate of *C* is affected by randomness and uncertainty in the estimation process. The capacity of the building against global collapse is a function of the earthquake accelerograms used in the IDA analyses (Vamvatsikos et. al., 2001). These accelerograms are part of a random process. The capacity is also affected by the uncertainty in the load-deformation behavior of the system determined from tests. The local collapse value is also affected by uncertainties in the response of the components due to variable material properties and fabrication.

The equation for calculating the resistance factor, ϕ is given by (Cornell et al, 2002):

$$\phi = \phi_{RC} \cdot \phi_{UC} \qquad (2)$$

$$\phi_{RC} = e^{\frac{-k\,\beta_{RC}^2}{2b}} \qquad (3)$$

$$\phi_{UC} = e^{\frac{-k\,\beta_{UC}^2}{2b}} \qquad (4)$$

where

ϕ_{RC} = Contribution to ϕ from randomness of the earthquake accelerograms

ϕ_{UC} = Contribution to ϕ from uncertainties in measured component capacity

The demand factor, γ, is calculated as:

$$\gamma = e^{\frac{k\,\beta_{RD}^2}{2b}} \qquad (5)$$

where

β_{RD} = $\sqrt{\sum \beta_i^2}$ where β_i^2 is the variance of the natural log of the drifts for each element of uncertainty.

The confidence factor, λ depends on the slope of the hazard curve, *k* and the uncertainty, but not randomness, associated with the natural log of the drifts. The equation for λ is (Jalayer and Cornell, 1999)

$$\lambda = e^{-\beta_{UT}\left(K_x - k\,\beta_{UT}/2b\right)} \qquad (6)$$

where

β_{UT}^2 = $\Sigma\sigma_i^2$ where σ_i is for uncertainties in the demand and capacity but not randomness

k = slope of the hazard curve

K_x = standard Gaussian variate associated with probability x of not being exceeded (found in standard probability tables)

3.0 EXAMPLES

3.1 New Buildings

Twenty steel moment frame buildings were designed in accordance with the 1997 NEHRP provisions. These included eight 3-story, eight 9-story and four 20-story buildings. It was assumed that the buildings were constructed using the beam-column connections that were pre-qualified by the SAC program. All buildings had a greater confidence level than 90% for both Local Collapse and Collapse. Local Collapse occurs when the connections at both end of a beam fracture and the beam falls to the floor below.

3.2 Existing Buildings

A series of 3- , 9- , and 20-story buildings were designed in accordance with the 1973, 1985, 1992 and 1997 Uniform Building Codes. It was also assumed that the connections were the pre-Northridge brittle type. These were then evaluated using the new performance-based procedure. The results are shown in Table 1 for global collapse. The results indicate that the newer that the building is, the higher is the confidence of satisfying the performance objective. The confidence levels range from 99% for buildings built after 1997 to 28% for 20-story buildings built in 1973. The confidence level for avoiding Local Collapse drop as low as 2% for buildings designed and constructed in 1972 or earlier.

4.0 CONCLUSIONS

This paper describes a new performance-based design and evaluation procedure for steel moment frame buildings designed for seismic loads. The procedure allows the designer to estimate the confidence level that the building will satisfy the design objective. It also is based on a rational method for accounting for randomness and uncertainty.

1 The new performance based procedure for designing new steel moment frame buildings is a powerful method for ensuring that the building will satisfy the performance objectives.

2 The new procedure provides a rational way to evaluate existing buildings for seismic loads.

3 Buildings designed by current codes and constructed with the new ductile connections are expected to perform very well during future earthquakes.

4 Older buildings designed before 1997 are more likely to collapse during future quakes.

5.0 REFERENCES

Cornell, C.A., Jalayer, F., Hamburger, R.O. and Foutch, D.A., 2002, “The Probabilistic Basis for the 2000FEMA/SAC Steel Moment Frame Design Guidelines,” Journal of Structural Engineering, ASCE, Vol. 128, No. 4, April.

FEMA-350, 2000, Recommended Seismic Design Procedures for New Steel Moment Frame Buildings, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington DC.

FEMA-351, 2000, Recommended Seismic Evaluation and Upgrade Criteria for Existing Steel Moment Frame Buildings, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, D.C.

FEMA-352 (2000), Recommended Post-earthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington DC.

Jalayer, F. and Cornell, C. A. (2003). “A Technical Framework for Probability-Based Demand and Capacity Factor Design (DCFD) seismic formats.” *PEER Report 2003/8*, Pacific Earthquake Engineering Center, University of California at Berkeley, Berkeley, Calif.

Yun, S. Y. and Foutch, D. A., 2000, “Performance Prediction and Evaluation of Low Ductility Steel Moment Frames for Seismic Loads.” SAC Background Report SAC/BD-00/26

Table 1 Collapse capacity, demand and confidence level for different building designs

Year	1997			1992			1985			1973		
Stories	C	D	Conf	C	D	Conf	C	D	Conf	C	D	Conf
3-story	0.10	0.027	99%	0.10	0.047	88	0.10	0.058	72%	0.10	0.056	75%
9-story	0.10	0.034	99%	0.078	0.043	57	0.094	0.048	64%	0.077	0.046	50%
20-story	0.085	0.024	96%	0.072	0.041	57	0.070	0.030	58%	0.069	0.045	28%