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Risk evaluation of steel frames with welded connections under earthquake

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Abstract. Numerous failures in welded connections in steel moment-resisting building frames (SMRF) were observed when buildings were inspected after the 1994 Northridge Earthquake. These observations raised concerns about the effectiveness of such frames for resisting strong earthquake ground motions. The behavior of SMRFs during an earthquake must be assessed using nonlinear dynamic analysis, and such assessments must permit the deterioration in connection strength to capture the behavior of the frame. The uncertainties that underlie both structural and dynamic loading also need to be included in the analysis process. This paper describes the analysis of one of approximately 200 SMRFs that suffered damage to its welded beam-to-column connections from the Northridge Earthquake is evaluated. Nonlinear static and dynamic analysis of this SMRF in the time domain is performed using ground motions representing the Northridge Earthquake. Subsequently, a detailed uncertainty analysis is conducted for the building using an ensemble of earthquake ground motions. Probability distributions for deformation-related limit states, described in terms of maximum roof displacement or interstory drift, are constructed. Building fragilities that are useful for condition assessment of damaged building structures and for performance-based design are developed from these distributions.

Key words: buildings (codes); design (buildings); connections; earthquakes; frames; loads (forces); probability; reliability; safety; strutural engineering; welding.

1. Introduction

Inspections of steel moment-resisting building frames following the Northridge Earthquake of January 17, 1994 revealed damage to a large number of welded beam-to-column connections in the form of severe weld cracking. Prior to the earthquake, it was generally believed that this structural frame system would perform well during severe earthquakes, and it continues to be a common structural system in areas of high seismicity. In the aftermath of the Northridge Earthquake, several research investigations, among them investigations by NIST (Gross 1998) and CUREe (SAC 1996), were initiated to determine the causes for the apparent poor performance of welded connections and to develop recommendations for improving building practices in areas prone to large earthquakes. The project conducted at NIST forms the basis for some of the findings reported in this paper. The surveys of connection damage following the Northridge Earthquake, improvements in structural modeling, and analysis of uncertainty in ground motion and structural response collectively provide

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an opportunity to validate and assess the limitations of current structural analysis and modeling procedures and the role of uncertainty in forecasting building performance during strong earthquakes.

2. Deterministic modeling of the building

A typical Pre-Northridge welded flange-bolted web moment-resisting connection is illustrated in Fig. 1. The detailing requires the beam flanges to be welded to the column using complete-penetration (CP) groove welds. The beam connection is made by either welding the web directly to the column or bolting the web to a shear tab which, in turn, is welded to the column. Prior to the Northridge earthquake, it was thought that this type of connection provided adequate strength and satisfactory ductility in most cases. However, about 200 SMRFs experienced various degrees of brittle connection failure during 1994 Northridge Earthquake.

A hysteretic model (Kunnath 1995, Gross 1998) was adopted for this study to capture this mode of connection behavior and its effects on building resistance. This model, illustrated in Fig. 2, incorporates the effects of damage due to weld fracture and subsequent nonlinear response in the connection region, and specifically describes connection failures that initiated at the root of the groove weld in the bottom beam flange and subsequently led to separation of bottom beam flange from the column flange. The moment at weld fracture is denoted by M_{cr}, which is specified as a fraction (β_5) of the yield moment, M_y. Following weld fracture, the primary envelope is replaced by a degraded bilinear model with reduced stiffness, reduced capacity, and modified post-yield slope. This connection model was incorporated in an inelastic dynamic analysis program, IDASS (Kunnath 1995), which was used to perform the evaluations of the frame. This program also has the capability of taking into account second-order (P-delta) effects in the frame if they occur during the analysis.

The building under consideration is a 13-story (one story is underground) office building in the San Fernando Valley, approximately 4.8 km southwest from the epicenter of the Northridge Earthquake. A 3-D view of the moment frames is shown in Fig. 3. Fig. 4 illustrates one of the moment frames at the perimeter of the building. Plan dimensions are 48.77 m by 48.77 m. Typical

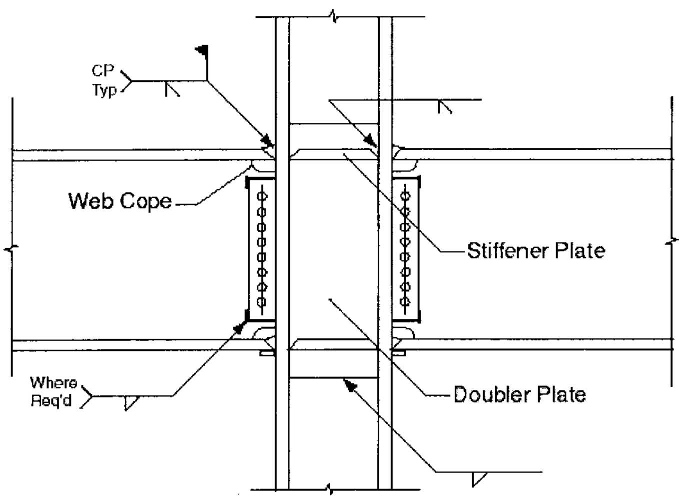

Fig. 1 Welded flange beam-to-column moment connection

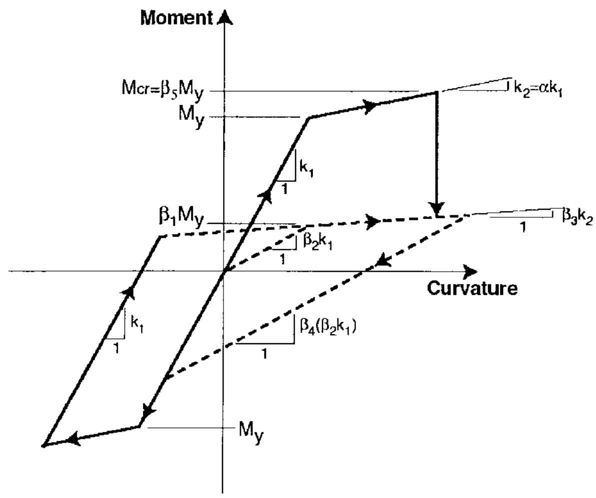

Fig. 2 Hysteresis model for damaged welded connection

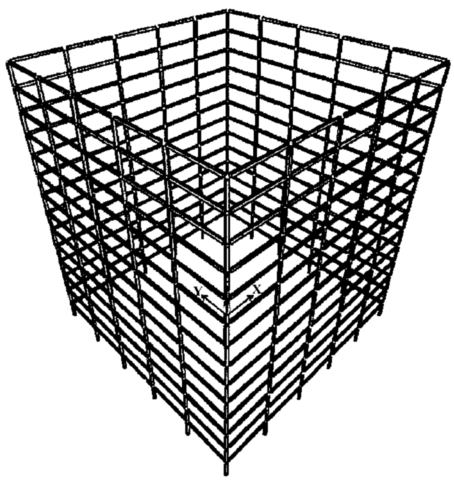

Fig. 3 3D view of the building

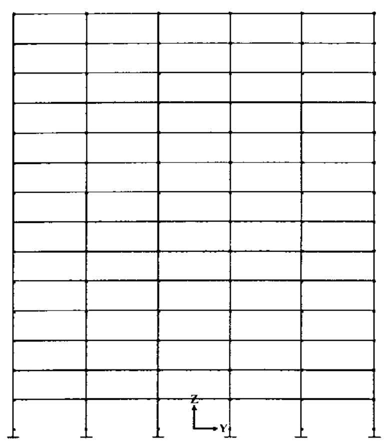

Fig. 4 N-S elevation of the building

story heights are 4.01 m and all bay widths are 9.75 m. Typical beams vary from W27×84 to W36×230. Interior columns vary from W14×167 to W14×500. The exterior columns are welded box sections varying in size from 371.5 mm to 447.7 mm. The building is almost symmetric in both directions, and thus two of the N-S perimeter moment frames, which experienced stronger ground motion than the E-W frames, were modeled as 2-D frames (Gross 1998). The interior beams and columns are only vertical-load-carrying frames and are not part of the 2-D model; nor are secondary and non-structural elements included in the model. Similar assumptions have been used by other investigators in modeling SMRFs (Luco and Cornell 2000). Both beam and column yield strengths were assumed to be 276 MPa. Floor diaphragms were assumed to be rigid in plane. Damping was assumed to be 5% of critical. The fundamental building period was determined by this frame analysis to be 3.1s.

This building was instrumented by the California Division of Mines and Geology at its basement, 6th floor, and roof levels. The fundamental period determined from the measured responses was 2.7 s in the N-S direction; the additional stiffness reflected in the measured dynamic response stems from the effects of the gravity frame and nonstructural cladding and partitions that are not included in the dynamic model of the N-S moment frame. The majority of connections in the building were inspected after the earthquake by Nabih Youssef and Associates (Uang *et al*. 1995). A static pushover analysis and a nonlinear time history analysis are performed described below.

A static pushover analysis, or collapse mode analysis, is a simple and efficient technique to study the nonlinear response of a building and, in particular, its distribution of forces subsequent to yielding (e.g., Gupta and Krawinkler 2000). The result from the pushover analysis is presented in a plot of base shear versus roof displacement in Fig. 5. The shape of the load-deformation curve is influenced by the static lateral force distribution used to load the structure. The force distribution can be approximated by the recommended design distribution (BSSC 1998):

$$F_i = V_B \frac{W_i h_i^k}{\Sigma_i W_i h_i^k} \tag{1}$$

where V_B is the base shear, h_i is the story height, W_i is the story weight, and exponent k simulates the distribution of forces associated with the first mode or more complex behavior. The apparent compliance of the building frame increases slightly with increasing k. Deviation from linearity occurs at an overall deformation that is approximately 1% of the building height. Deformations of this order are sufficient to cause damage to nonstructural components and cladding.

Limit states of performance during earthquakes are measured most simply in terms of deformation (Song and Ellingwood, I 1999). In this study we use roof displacement angle, defined as the maximum roof displacement normalized by the building height,

$$\mathrm{RDA} = \frac{\delta}{H} \tag{2}$$

where δ is the maximum roof displacement from dynamic analysis and H is the overall building height, and maximum interstory drift angle, defined as,

$$\mathrm{ISDA} = \max_{i=1}^{14} \left(\frac{\delta_i}{h_i} \right) \tag{3}$$

where δ_i is the maximum interstory drift for story i and h_i is the story. It is possible to map such global measures of structural response to various qualitatively stated conditions of building performance – nonstructural damage, life safety, and so forth (Song and Ellingwood, II 1999). Such measures also are consistent with research carried out elsewhere as part of the SAC Joint Venture (Luco and Cornell 1997, Wen and Foutch 1997).

Using the ground motion recorded in the basement by the California Division of Mines and Geology (CDMG), a nonlinear time history analysis was performed. The results of this analysis are presented in Gross (1998). The RDA was determined to be 0.55% from inelastic MDOF time history analyses, as compared to 0.49% from the CDMG record on the roof of the building. The 5%-damped spectral acceleration at the fundamental period was 0.23 g. The period and shape of the response, from both the analysis and the record on the roof, were also found to be close (Song 1998). Similar connection damage patterns, most of which concentrated in mid-to-lower levels, were observed in the predicted and the surveyed results (Gross 1998).

However, predictions of nonlinear response of steel frames subjected to strong ground motion may not match what is observed particularly well, even when advanced nonlinear dynamic analysis tools are used (Song 1998). This is due, in part, to the fact that ground motions and building properties are random in nature. Assuming only mean values of properties and one best-estimated ground motion may lead to results that deviate significantly from the actual response of the building. Moreover, results from one analysis do not provide insight on the likely resistance of the building to future earthquakes. A probabilistic rather than deterministic analysis of building response to earthquake ground motion can place such comparisons in better perspective by indicating the agreement between predicted and observed damage that might be expected, given the level of uncertainty in the problem.

3. Stochastic modeling of the building

The structural parameters that are treated as random include beam and column yield strength, elastic modulus, damping, and connection hysteresis parameters β_1 and β_5 defined in Fig. 2. β_1 is the remaining strength after the connection fractures; β_5 indicates the level at which the connection fractures. A sensitivity study in Song (1998) revealed that these two parameters are most critical of the five connection parameters in determining RDA and ISDA in nonlinear dynamic analysis. Their probability distributions can be found in Table 1; the basis for these probability models is provided elsewhere (Song 1998).

In recent years numerous methods have been proposed to construct ensembles of ground motion for design or reliability assessment. In this study, the approach taken in the SAC Project is followed. Recent NEHRP Recommendation (NEHRP 1998) specify the seismic hazard in terms of spectral acceleration rather than peak ground acceleration. Thus, in this study, spectral acceleration (S_a) was chosen to characterize the ground motion intensities so that the building resistance can be keyed to this measure. An ensemble of ground motions is required for stochastic analysis of building response (e.g., Shome and Cornell 1998). A total of nine historic earthquake accelerograms with magnitudes from 5.3 to 6.7 and epicentral distances from 5 km to 24 km were used for this purpose (Table 2). The mean spectral acceleration of these nine records was determined to be 0.09 g at T=3.1 s. In order to compare the scaling effect on building response, three series of experimental designs for the building were carried out: (1) unscaled ground motions; (2) ground motions scaled to S_a=0.09 g; and (3) ground motions scaled to S_a=0.23 g, which, as noted previously, is the spectral acceleration associated with the CDMG record at the site. In other words, in the second and third experiments, the ground motions were scaled so that the same roof acceleration would be achieved

Table 1 Random material strength parameters

Parameter	Mean	COV	CDF
$F_{y,col}$ (Mpa)	276	0.12	Lognormal
$F_{y,beam}$ (Mpa)	276	0.12	Lognormal
β_1	0.4	0.29	Uniform
β_5	0.95	0.09	Uniform
E (Gpa)	200	0.06	Uniform
ξ	5%	0.29	Uniform

Table 2 Earthquakes and records in stochastic analysis

Earthquake	Date	Recording site	CIT number	Magnitude	Approx. Dist. (km)	Component
1. Imperial Valley	5/18/40	El Centro	A001	6.7	9	S00E
2. Northern California	3/9/49	Hollister Public Library	U301	5.3	21	S01W
3. Eureka	12/21/54	Eureka Federal Building	A008	6.6	24	N11W
4. Hollister	4/8/61	Hollister City Hall	A018	5.6	21	S01W
5. Parkfield (Array 5)	6/27/66	Cholame Shandon	B034	5.3	5	N05W
6. San Fernando	2/9/71	8244 Orion Blvd.	C048	6.4	20	N00W
7. San Fernando	2/9/71	Hollywood Storage Lot	D058	6.4	21	S00W
8. San Fernando	2/9/71	Palmdale Fire Station	G114	6.4	15	S30W
9. San Jose	9/4/55	Bank of America Basement	A010	5.8	10	N31W

Table 3 Stochastic analysis result for RDA

Experiment	Unscaled ground motion	Scaled to S_a=0.09g	Scaled to S_a=0.23g
1	0.57%	0.48%	0.59%
2	0.76%	0.35%	0.88%
3	0.46%	0.29%	0.65%
4	0.12%	0.42%	0.69%
5	0.23%	0.41%	1.00%
6	0.45%	0.35%	0.97%
7	0.28%	0.37%	0.93%
8	0.14%	0.34%	0.85%
9	0.15%	0.45%	0.64%
mean	0.35%	0.38%	0.80%
COV	63.28%	15.61%	19.71%

from a SDOF elastic analysis of a deterministic model of the structure.

The uncertainties in ground motion and in the remaining structural parameters are treated using a Latin Hypercube sampling plan (Song 1998, Imam and Conover 1980, O'Connor and Ellingwood 1987). Sensitivity studies (Song 1998) have shown that ensembles of nine accelerograms are sufficient to define the fragilities presented in this paper.

The RDAs predicted from nine samples are presented in Table 3. In this illustration, scaling the ground motions to produce the same S_a has little effect on the mean RDA (cf Columns 1 and 2 of Table 3). Note, however, that scaling reduces the coefficient of variation (COV) in response by 74%. This finding is consistent with the results from concurrent SAC Joint Venture studies (e.g., Shome and Cornell 1998). The mean RDA of 0.8%, from the stochastic analysis, which coincides with the displacement at which the building starts to yield from static pushover analysis (k=1 in Fig. 5), is about 30% higher than the RDA of 0.6% from the deterministic analysis of the building. If inelastic structural displacements due to dynamic forces are considered to be good indicators of the damage-causing potential of an earthquake characterized by a given response spectrum, then an ensemble of ground motions for purposes of prediction and evaluation can be created by scaling all ground motions to yield a common S_a at the fundamental period of the building. This modeling

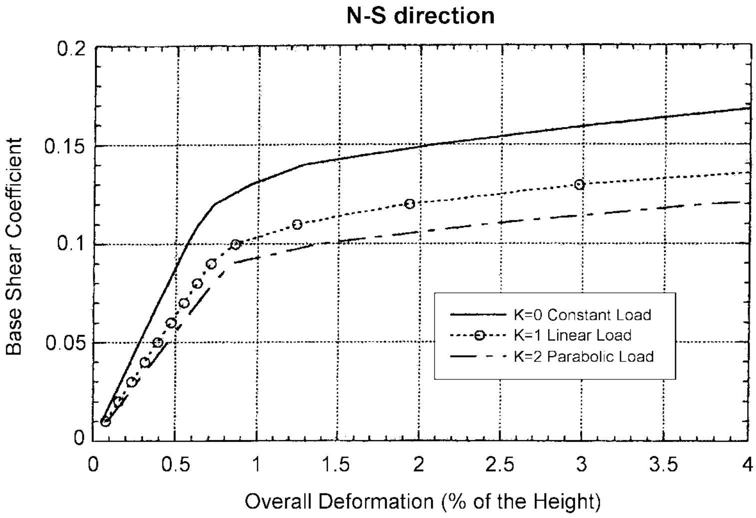

Fig. 5 Pushover analysis

procedure reduces the contribution of uncertainty in the ground motion intensity to the variance in the response of the system, leaving uncertainties in soil conditions and attenuation laws reflected in random phasing of the record and duration of strong motion as the main contributors to overall response variance. Statistics of the ISDAs can be estimated similarly from the stochastic analysis.

4. Fragility modeling of building resistance

The resistance of a building as a system can be described probabilistically by its fragility, $F_R(x)$. The fragility is defined as the probability of a limit state (described in terms of RDA or ISDA), conditioned on spectral acceleration that is consistent with the specification of the seismic hazard;

$$F_R(x)=P[LS|S_a=x] \tag{4}$$

where LS represents the limit state and spectral acceleration, S_a, at the fundamental period of the building, is the control variable. The limit states pertain to the structural frame as a system rather than to any one beam or column. The fragility for any limit state in this paper is obtained from the cumulative distribution function (CDF) of the RDA. For example, the (qualitatively stated) performance objective “building function should not be impaired” might be associated with (or mapped to) a (quantitatively defined) structural limit state defined as not exceeding 2% RDA; thus, the fragility would be,

$$F_R(x)=P[RDA>2\%|S_a=x] \tag{5}$$

The fragility in Eq. (5) often is modeled by a lognormal distribution (Kennedy and Ravindra 1984).

To determine these conditional probabilities, the ground motion ensembles were scaled so that S_a at the fundamental period of the building increased over the range of interest, the corresponding dynamic responses of the frame to these ensembles were determined, the responses were rank-ordered on lognormal probability plots, and Eq. (5) was used to determine the fragilities for increasing levels of RDA (Song and Ellingwood, II 1999). Three levels of performance and their corresponding hypothesized limit states are presented here: RDA=1% (onset of nonstructural damage), 2% (impaired function); and 5% (severe damage) (Ellingwood 1998). Fig. 6 presents the fragility for the three deformation limits identified above. It can be seen from the figure that lognormal CDF is a reasonable approximation to the simulated results. The median (50th percentile) spectral accelerations at which "failure/nonfailure" is equally likely are 0.33*g* for RDA=1%, 0.59 g for 2%, and 1.45 g for 5%, or at progressively more severe limit states. The fragility variability is considerably larger at the more severe RDA limit states. Similar results can be found for performance limits defined by the ISDA in Fig. 7. The fragility of *ISDA* for another damaged building in Northridge Earthquake was presented in detail elsewhere (Song and Ellingwood, II 1999).

The NEHRP Recommended Provisions (NEHRP 1998) are based on a maximum considered earthquake (MCE), defined by a spectral acceleration having a probability of 2% of being exceeded in 50 years. The design spectral response acceleration is two-thirds the MCE. From a seismic hazard analysis performed independently (Song 1998), this corresponds to a spectral acceleration of

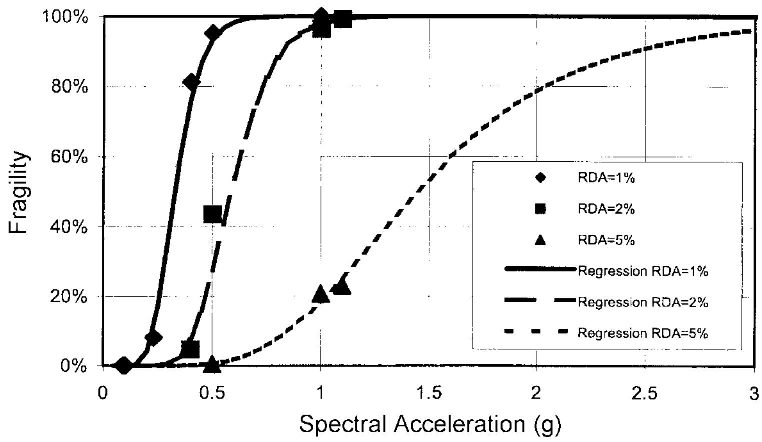

Fig. 6 Comparison of fragility for RDA limit states

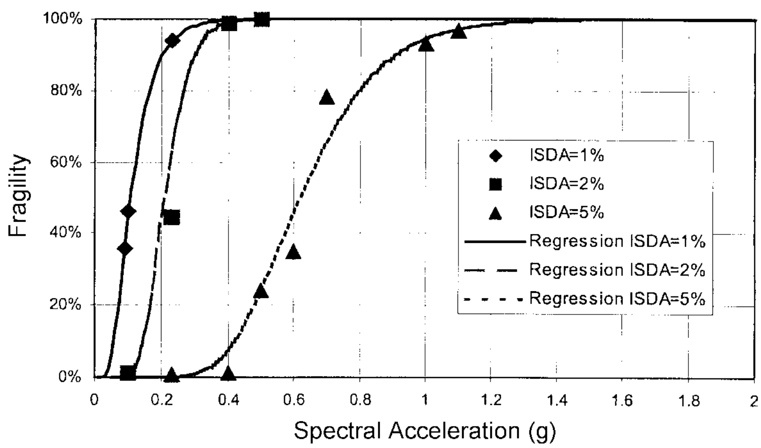

Fig. 7 Comparison of fragility for ISDA limit states

approximately 0.6 g at the site of the building. In an evaluation of a building with regard to public safety, building resistance might be identified with the 10 percentile of the “severe damage” (5% RDA) in order to be on the conservative side. Fig. 6 shows that the 10-percentile fragility of the 13-story building considered herein is 0.85 g. Therefore, this building does conform to the most recent NEHRP Recommended Provisions for new buildings (with 90% confidence).

It is interesting to note that for the same damage limit state, S_a for ISDA is lower than for RDA. In other words, the required force to cause the overall building deformation to reach 5% of building height is larger than the force required to cause an individual story to reach the same deformation limit of 5% of story height. These differences are larger in this 13-story frame than in the low-rise frames considered previously (Song and Ellingwood, II 1999). Of course, the fragility curves based on ISDA and RDA would be identical for a single story building.

The first two median values of 0.33 g for RDA=1% and 0.59 g for 2% are less than the review earthquake level of 0.6 g; the median of 1.45 g for RDA = 5% exceeds it by a comfortable margin. At 0.6 g earthquake review level, the 2% RDA (impaired function limit state) would be reached with 54% probability. On the other hand, at S_a=0.23 g, measured during the Northridge earthquake, limit states of impaired function (2% RDA) or severe damage (5% RDA) have negligible probability of occurrence. Indeed, this is consistent with the actual experience of this building, which did not require strengthening following the Northridge Earthquake.

5. Conclusions and recommendations

Deterministic nonlinear pushover static and dynamic time history analysis using a degraded connection model can predict displacement and connection damage pattern with reasonable accuracy. However, using only mean values of structural parameters and only one ground motion might not capture the whole picture of nonlinear building response. The earthquake loading and structural resistance are both random in nature and therefore, a probabilistic rather than deterministic analysis offers a better view on the possible building behavior in response to earthquake.

Assessment of building response through a range of potential earthquake magnitudes requires a series of nonlinear dynamic analyses in the time domain. Scaling the actual earthquake ground motions to the same spectral acceleration at the fundamental period of the structure is a suitable way to create a ground motion ensemble for time-domain analysis purposes. It models the contribution of nonstationarity and uncertainty in amplitude and frequency content of the ground motion in a natural way. Near-field ground motions are known to have fundamentally different characteristics, and should not be included in these ensembles.

A fragility curve provides a simple depiction of the likelihood of unacceptable building frame performance. It can be constructed with sufficient accuracy for preliminary condition assessment purposes using a relatively simple random sampling procedure. Indeed, recent studies have suggested that only a few nonlinear analyses of a building frame are required to anchor the fragility curve for progressively severe limit states, suggesting that investigators focus their attention on refining the structural modeling process rather than on developing highly efficient Monte Carlo sampling procedures. Such studies are still in progress. A properly constructed fragility curve offers a broad perspective on the likely performance of existing buildings during earthquakes of various severities, and can be used as one of several tools for improving earthquake-resistant building practices.

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