EFFECTS OF RANDOM CONNECTION FRACTURES ON THE DEMANDS AND RELIABILITY FOR A 3-STORY PRE-NORTHRIDGE SMRF STRUCTURE

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ABSTRACT

Since the Northridge earthquake of 1994, the seismic reliability of new and existing steel moment-resisting frame (SMRF) buildings has been in question. The observed brittle fractures of welded beam-column connections have prompted research on how brittle connection behavior affects the seismic performance of SMRF structures. As a means for quantifying some of the effects of connection fractures, a procedure for assessing the seismic drift demand hazard for a structure, and its reliability against a particular collapse limit state, is presented. The procedure combines a conventional spectral acceleration seismic hazard curve with results of a suite of nonlinear analyses, as demonstrated for a three-story SMRF building designed according to practices prevalent before the Northridge earthquake (i.e., pre-Northridge). In the absence of a practical analytical model which can accurately predict connection fractures, and faced with the many uncertainties involved in the behavior of brittle connections, an empirical analysis model for connection fracture and random simulation are employed for the nonlinear dynamic analyses that are vital to the procedure. By comparing the results of the procedure for the example structure with brittle and with ductile connections, the effect of brittle connection behavior on the drift demands and reliability for the structure can be evaluated.

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Introduction

The procedure described in this paper for evaluating the drift demand hazard and the annual probability of reaching a collapse limit state for a model structure, has been developed as part of the SAC steel project (Cornell, 1997 and Wen, 1997). The procedure is presented generally and then demonstrated for a three-story SMRF building. Two analysis models of the building, one with all brittle connections and the other with all ductile connections, are considered. The results of the procedure for the brittle and ductile cases are compared in order to investigate the effect of connection fractures on the performance of the model structure. In addition, the "dynamic pushover" analysis is introduced as a tool for studying the nonlinear behavior of a model structure; the results of such analyses for the ductile and brittle cases are also compared.

Procedure

The procedure described in this paper can be implemented for any local or global demand parameter, but here it is presented for evaluating the drift[3] demand hazard and the annual probability that the drift demand exceeds the drift capacity (or the probability of failure, for short). The procedure combines an existing site hazard curve for spectral acceleration with drift response results from nonlinear dynamic analyses of a model structure subjected to several ground motions at different levels of intensity (as measured by spectral acceleration), to arrive at a drift demand hazard curve. More specifically, the annual probability of exceeding any specified drift demand, and the drift demand associated with a particular exceedance probability, can be computed. With estimates of the median drift capacity and the dispersion of drift capacity, the annual probability of failure (i.e., the probability that the drift demand exceeds the drift capacity when the drift capacity is regarded as a random variable), and the "design spectral acceleration" corresponding to a target probability of failure, can also be computed. A possible method for estimating the median and dispersion of drift capacity makes use of dynamic pushovers.

Spectral Acceleration Hazard

An existing site hazard curve for spectral acceleration provides the probability of exceeding any particular spectral acceleration, for a given period and damping ratio. The elastic spectral acceleration at the fundamental period of the model structure is used since it is usually an effective structure-specific measure of ground motion intensity for predicting the nonlinear response of buildings (like the three-story SMRF considered in this paper) with a period of around one second (Shome and Cornell, 1998). This and other such measures are under investigation for taller, longer period buildings. An "effective" intensity measure for earthquake records is one for which the record-to-record dispersion of the drift response given the intensity level is relatively small, and for which a hazard analysis is available. The particular choice of ground motion intensity measure, however, is not critical to the procedure.

Relationship between Spectral Acceleration and Drift

The median relationship between spectral acceleration and drift is established by performing nonlinear dynamic analyses of the model structure for numerous ground motions at different levels of intensity (as measured by spectral acceleration). The spectral acceleration (e.g., at the fundamental period of the model structure) for each ground motion is simply obtained from its

[3] For the procedure described in this paper, drift can refer to any number of drift measures, such as inter-story drift, drift ductility, or a drift-dependent damage index.

elastic response spectrum (e.g., for 5% damping). The response of the model structure subjected to each earthquake record provides the corresponding drift. For a set of spectral acceleration versus drift data points, a regression (or "least squares fit") of the form

$$\hat{\delta} = a\, S_a^{\,b} \tag{1}$$

where $\hat{\delta}$ is the median drift response and S_a is the spectral acceleration, provides the necessary relationship between spectral acceleration and median drift. The exponent b in Equation 1 is included to capture "softening" of the nonlinear relationship between spectral acceleration and median drift. Also note that a regression of the form given in Equation 1 is equivalent to a linear regression of the log of drift on the log of spectral acceleration. The dispersion of the drift response given the spectral acceleration is calculated as the mean squared deviation of the (spectral acceleration versus drift) data points from the regression fit.

Drift Demand Hazard

Once the median relationship between spectral acceleration and drift (i.e., the median drift given spectral acceleration), and the dispersion of drift given spectral acceleration are known, the spectral acceleration hazard curve can be used to create a drift demand hazard curve. Under certain simplifying assumptions (Cornell, 1996), the probability of exceeding any specified drift demand, δ', can be calculated in closed analytical form as

$$H_\delta(\delta') \equiv P[\delta > \delta'] = H_{S_a}\left(S_a^{\delta'}\right) \cdot C_{f_1} \tag{2a}$$

where $H_{S_a}(x)$ is the spectral acceleration hazard (or mean annual frequency of exceeding x), $S_a^{\delta'}$ is the spectral acceleration corresponding to δ' (i.e., the inverse of the median relationship between spectral acceleration and drift), and C_{f_1} is a correction factor which accounts for the dispersion in drift given spectral acceleration. $S_a^{\delta'}$ and C_{f_1} are calculated as

$$S_a^{\delta'} = \left(\frac{\delta'}{a}\right)^{\frac{1}{b}} \tag{2b}$$

$$C_{f_1} = e^{\frac{1}{2} k^2 \cdot \sigma^2_{\ln(\delta)|S_a} / b^2} \tag{2c}$$

where a and b are the regression coefficients from Equation 1, k is the log-log slope of the spectral acceleration hazard curve (fit near the spectral acceleration of interest), and $\sigma_{\ln(\delta)|S_a}$ is the "COV" [4] of drift given spectral acceleration.

[4] In this paper, the "COV" (also referred to as the dispersion) is defined as the standard deviation of the natural logarithms of the data, which is approximately equal to the conventional coefficient of variation (i.e., the standard deviation divided by the mean) for values less than 0.3. Correspondingly, the term median is used in this paper to refer to the geometric mean, which is calculated as the exponential of the average of the natural logarithms of the data. The geometric mean is a logical estimator of the true median, especially if the data are at least approximately lognormally distributed.

The drift hazard curve can also be read to determine the drift demand corresponding to a prescribed probability level. Alternatively, the drift demand associated with a particular annual probability of exceedance, P_0, can be calculated explicitly using the formula

$$\delta^{P_0} = \delta^{S_a^{P_0}} \cdot C_{f_3} \tag{3a}$$

where $\delta^{S_a^{P_0}}$ is the median drift corresponding to $S_a^{P_0}$, which is the spectral acceleration associated with the prescribed annual probability of exceedance. As in Equation 2a, C_{f_3} is a correction factor which accounts for the dispersion in drift given spectral acceleration, and is calculated as

$$C_{f_3} = e^{\frac{1}{2} k \cdot \sigma^2_{\ln(\delta)|S_a} / b} \tag{3b}$$

Equation 3a provides, in effect, the "load" factor which should be applied to the spectral acceleration at a given exceedance probability level in order to find the drift demand at that probability level, recognizing the dispersion in nonlinear structural responses given the ground motion intensity (i.e., spectral acceleration).

Collapse Limit State Probability

If the drift capacity for a model structure is regarded as a random variable, the annual probability that the drift demand exceeds the drift capacity (i.e., the probability of failure) can be calculated using the equation

$$P_f \equiv P\left[\delta^{demand} > \delta^{capacity}\right] = H_{S_a}\left(S_a^{\hat{\delta}^{capacity}}\right) \cdot C_{f_2} \tag{4a}$$

where $S_a^{\hat{\delta}^{capacity}}$ is the spectral acceleration corresponding to the median drift capacity, and C_{f_2} is a correction factor which accounts for both the dispersion in drift demand given spectral acceleration and the dispersion in drift capacity. $S_a^{\hat{\delta}^{capacity}}$ and C_{f_2} are calculated as

$$S_a^{\hat{\delta}^{capacity}} = \left(\frac{\hat{\delta}^{capacity}}{a}\right)^{\frac{1}{b}} \tag{4b}$$

$$C_{f_2} = e^{\frac{1}{2} k^2 \cdot \left(\sigma^2_{\ln(\delta)|S_a} + \sigma^2_{\ln(\delta^{capacity})}\right) / b^2} \tag{4c}$$

where $\sigma_{\ln(\delta^{capacity})}$ is the "COV" of drift capacity. Clearly, in order to calculate the probability of failure, the median and dispersion of the drift capacity must be estimated; this issue is discussed in the following subsection.

Analogous to calculating the drift demand corresponding to a particular annual probability of exceedance, the "design spectral acceleration" associated with a particular probability of failure (i.e., probability that the drift demand demand exceeds the drift capacity), P'_f , can be computed using the equation

$$S_a^{design} = S_a^{P'_f} \cdot C_{f_4} \tag{5a}$$

where $S_a^{P'_f}$ is the spectral acceleration corresponding to an annual exceedance probability of P'_f (from an elastic spectral acceleration hazard curve), and C_{f_4} is a correction factor which accounts for both the dispersion in drift demand given spectral acceleration and the dispersion in drift capacity. The correction factor C_{f_4} is calculated as

$$C_{f_4} = e^{\frac{1}{2} k \cdot \left(\sigma^2_{\ln(\delta)|S_a} + \sigma^2_{\ln(\delta^{capacity})} \right) \Big/ b^2} \tag{5b}$$

If a proposed structural design is "deterministically" analyzed for this design spectral acceleration (Wen, 1997), and the resulting drift demand does not exceed the median drift capacity, then the failure probability for the structural design does not exceed the target failure probability.

Dynamic Pushovers

For some drift parameters, the drift capacity for a model structure may be difficult to identify. Such is the case for maximum story drift angle[5] (over all stories), which is the basic demand parameter employed by SAC and used for the numerical example in this paper. SAC is investigating the prospect of using "dynamic pushover" analyses of a model structure, subjected to several earthquake records, to characterize the maximum story drift angle capacity against collapse. A single dynamic pushover analysis entails performing multiple nonlinear dynamic analyses for a model structure subjected to an earthquake record which is incrementally scaled. The result is a dynamic pushover curve which relates the scale factor for the earthquake record and the drift response of the model structure. From the dynamic pushover curve, the maximum story drift angle limit corresponding to the transition point when the analytical response of the model structure becomes "unstable" (i.e., when the dynamic drift response increases drastically for a relatively small increase in ground motion intensity), or when the apparent "stiffness" (i.e., the slope of the dynamic pushover curve) decreases radically, may be used as a measure of the maximum story drift angle capacity. It is important to note that this "dynamic capacity" is different, in concept, than a static story drift angle capacity. With several estimates (from dynamic pushover curves for several earthquake records) of the maximum story drift angle capacity, the median and dispersion of the maximum story drift angle capacity, and hence the probability of failure for the model structure, can be calculated.

[5] Story drift angle is defined as inter-story drift normalized by story height.

Numerical Example

The procedure presented above for determining the drift demand hazard and the annual probability of reaching a collapse limit state is now demonstrated for a three-story SMRF model structure. In order to quantify the effects of brittle connection behavior on the demands and the reliability, the procedure is carried out for the model structure with brittle connections and with ductile connections. Dynamic pushover analyses for both the ductile and brittle cases are also performed in an attempt to characterize the drift capacity, and as another basis for evaluating the effect of connection fractures on structural performance. For this example, the basic drift demand parameter considered is the maximum story drift angle over all stories, denoted as $\bullet_{max}$. Also, the elastic spectral acceleration at the fundamental period of the model structure (1.03 seconds) for a damping ratio of 2% (the damping ratio used for dynamic analysis) is used as the structure-specific measure of ground motion intensity, and is denoted as S_a.

Ground Motions

Thirty of the SAC Phase II ground motions for Los Angeles at the 10% in 50 years and 2% in 50 years probability levels are used for analysis. The twenty 10% in 50 years earthquake records (LA01-LA20) and the ten 2% in 50 years earthquake records (LA21-LA30) are recorded ground motions which have been scaled to match, in a minimum weighted least squares residual sense, the 1997 USGS mapped spectral values at four periods, namely 0.3, 1.0, 2.0, and 4.0 seconds (Somerville, 1997).

Model Structure

The structure evaluated in this example is the SAC Phase II three-story building designed according to pre-Northridge practices for Los Angeles conditions. A two-dimensional centerline model of one of the building's perimeter moment-resisting frames is used for analysis. Brittle connections are incorporated into the model with a "fracture element" implemented in DRAIN-2DX by (Foutch and Shi, 1996). The element mimics the behavior seen in full-scale laboratory tests of moment-resisting beam-column connections which experience top and/or bottom beam flange fracture. As for the common inelastic, but ductile element, the fracture element is a rotational spring that is placed at the ends of an elastic beam element in order to emulate point plasticity, and in this case, fracture.

Laboratory test results (SAC, 1996), as well as field inspections of connections damaged by the Northridge earthquake, suggest that a moment-resisting connection may fracture before reaching its nominal plastic moment, M_p. To account for this possibility, in this example it is assumed that the probability of any moment-resisting beam-column connection fracturing before reaching M_p (i.e. "early") is 25%. The connections which fracture "early" are set to fracture at 75% of M_p. The remaining connections fracture when the maximum plastic rotation reaches 0.015 radians. Once fracture occurs, the bending strength of the connection when the fractured flange is "in tension" drops to 30% of M_p. As frequently observed in the field, only bottom flange fracture is considered for this example.

For each earthquake record used for dynamic analysis, a different, random spatial distribution of "early" fracturing connections is simulated assuming mutual independence of the connections. Thus, for the thirty ground motions considered in this example, thirty different model structures (or realizations of the model structure) are analyzed. This simulation technique is utilized in lieu of simulating several different model structures for each record in order to minimize the number of analyses. A check verifies that the analysis of a different spatial distribution of "early" fracturing connections for each earthquake does not (a) bias the median θ_{max} response, (b) significantly alter the estimate of the dispersion in response (i.e., the "COV" of θ_{max} given S_a), or (c) significantly change the regression of θ_{max} on S_a (Luco and Cornell, 1997).

Spectral Acceleration Hazard

For this example, a spectral acceleration hazard curve of the form

$$H_{S_a}(S_a) = k_0\, S_a^{-k} \tag{6}$$

is obtained simply by fitting a line (in log-log scale) to the points defined by the two annual exceedance probabilities and the corresponding median spectral accelerations, for the two sets of SAC ground motions. In this case, the log-log slope of the spectral acceleration hazard curve is simply ***-k*** **(**= -3.03 for this example). The hazard curve utilized in this example, as well as the two points used to obtain it, are shown in Figure 1. Note that since the two median spectral accelerations (at the fundamental period of the model structure) are for a damping ratio of 2%, the simple hazard curve used here is for a damping ratio of 2% rather than for the 5% value typically reported by USGS. Also, the SAC earthquake records, and hence the simple hazard curve created for this example, were modified to reflect a firm soil site rather than the soft rock site used as a basis by USGS (Somerville, 1997).

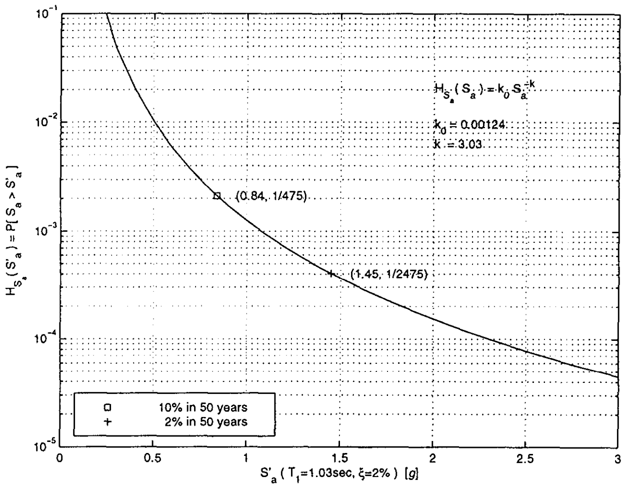

Figure 1. Annual hazard curve for spectral acceleration.

Relationship between Spectral Acceleration and Drift

Plots of S_a versus θ_{max} from nonlinear dynamic analyses using the twenty 10% in 50 years and the ten 2% in 50 years ground motions are presented in Figure 2 for the model structure with ductile and with brittle connections. The regression analyses results, including the "COV" of θ_{max} given S_a, are also shown on Figure 2. Note the increase in the dispersion of θ_{max} given S_a from the ductile to the brittle case (0.217 to 0.300). Closer inspection reveals that the majority of this increase is due to fundamental differences in the dynamics of the ductile and brittle model structures, rather than differences in the random locations of "early" fracturing connections, which only accounts for about 5% of the total dispersion. It is also interesting to note that the value of the regression coefficient *b* for the ductile case is significantly smaller (by more than two times the standard error of estimation of *b)* than one, indicating a "hardening" (i.e., increase in slope) of the median S_a versus θ_{max} curve.[6]

[6] The same phenomenon is seen in several of the dynamic pushover curves presented later in Figure 6. In many cases, the increase in S_a without a proportional increase in θ_{max} coincides with a change in the direction of maximum drift reponse, or a shift in the story in which θ_{max} occurs. Further investigation into the subject is still required.

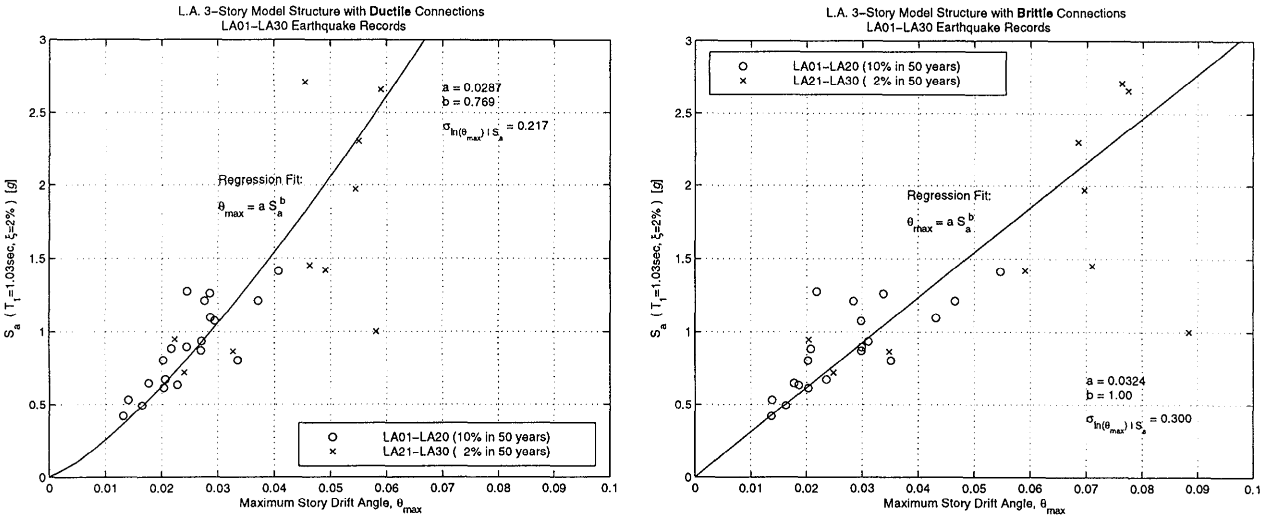

Figure 2. S_a versus θ_{max} and regression analysis results for the model structure with (a) ductile connections, and (b) brittle connections.

Median values of θ_{max} and S_a, and the "COV" of θ_{max} given S_a for the 10% in 50 years and 2% in 50 years earthquake records are listed in Table 1 for both the ductile and brittle cases. Note that the increase in the median θ_{max} from the ductile to the brittle case is significantly larger for the 2% in 50 years probability level (26% increase) than for the 10% in 50 years probability level (7% increase). This result agrees with intuition, as one would expect the effect of connection fractures to be more pronounced for ground motions of larger intensity. In either case, the increase in the median θ_{max} response due to connection fractures is smaller than one might have anticipated. Also note that while only a single value for the "COV" of θ_{max} given S_a is used for the procedure described in this paper, the results in Table 1 and other studies (Shome and Cornell, 1998) suggest that the "COV" of θ_{max} given S_a also increases with the drift level (or ground motion intensity) in the nonlinear range.

Table 1. Median and "COV" values of θ_{max} and S_a for 10% in 50 years and 2% in 50 years ground motions.

	10% in 50 years		2% in 50 years	
	Ductile	Brittle	Ductile	Brittle
median θ_{max}	0.0238	0.0255	0.0424	0.0533
median S_a $[g]$	0.84	0.84	1.45	1.45
$\sigma_{\ln(\theta max)\mid Sa}$	0.165	0.239	0.321	0.433

Drift Demand Hazard

With the regression coefficients a and b of Equation 1, the "COV" of θ_{max} given S_a, and the log-log slope of the spectral acceleration hazard curve (equal to $-k$ for this example), the probability of exceeding any particular maximum story drift angle demand, is computed according to Equation 2a. The resulting annual hazard curve for θ_{max} demand is presented in Figure 3 for the model structure with ductile and with brittle connections. Note that the θ_{max} demands corresponding to exceedance probabilities of 1/475 (10% in 50 years) and 1/2475 (2% in 50 years), also shown in Figure 3, are calculated explicitly using Equation 3a.

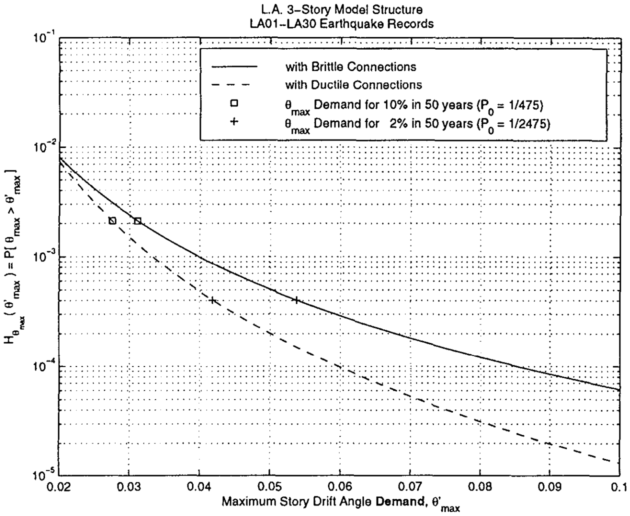

Figure 3. Annual hazard curve for maximum story drift angle demand.

As expected, brittle connection behavior causes an increase (over the ductile case) in the probability of exceedance for a given θ_{max} demand, or alternatively, an increase in the θ_{max} demand for a given hazard level. This increase is a consequence of both the larger median and the larger "COV" of θ_{max} given S_a for the model structure with brittle connections. As already demonstrated for the median θ_{max} demands, the difference in the probability of exceedance between the ductile and brittle cases is greater at larger levels of demand.

Collapse Limit State Probability

In order to calculate the annual probability that the θ_{max} demand exceeds the θ_{max} capacity for the model structure (i.e., the probability of failure), the median θ_{max} capacity and the "COV" of the θ_{max} capacity must be estimated. An attempt to identify the maximum story drift angle capacity against collapse using dynamic pushover analyses is detailed in the following subsection, As an alternative, the probability of failure is calculated according to Equation 4a for a range of median θ_{max} capacities (0.025 to 0.10) and two values of θ_{max} capacity "COV" (0.10 and 0.40). A value of 0.025 for the median θ_{max} capacity may be regarded as a lower bound which approximates the *static* story drift angle capacity; that is, an elastic drift angle of approximately 0.01 and an inelastic drift angle of 0.015 (assuming that the inelastic drift angle is equal to the plastic rotation in the beam-column connections *before* fracture). The upper bound median θ_{max} capacity of 0.10 merely corresponds to the story drift angle after which the credibility of the analysis model is likely undependable. Likewise, the θ_{max} capacity "COV" values of 0.10 and 0.40 are presumably extreme lower and upper values.

The annual probabilities of failure calculated for the three-story model structure with ductile and with brittle connections are shown in Figure 4. It is important to recognize that the median θ_{max} capacity, as well as the "COV" of θ_{max} capacity, are likely different for the ductile and the brittle cases. Thus, a comparison of the probabilities of failure for the ductile and brittle cases cannot be made by simply comparing the probabilities for a single value of median θ_{max} capacity (or θ_{max} capacity "COV").

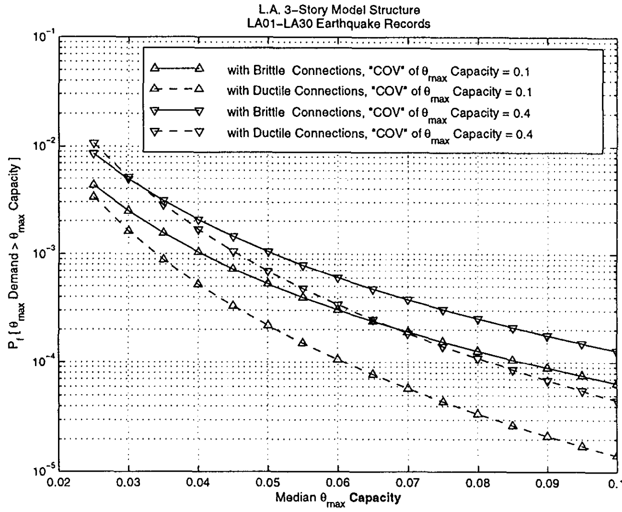

Figure 4. "Probability of failure" versus median θ_{max} capacity.

Finally, the "design spectral acceleration" necessary to yield a specified (low) probability of failure is computed according to Equation 5a. The design spectral acceleration values for a range of failure probabilities (including 10% in 50 years and 2% in 50 years), and for two values of θ_{max} capacity "COV" (0.10 and 0.40), are presented in Figure 5 for both the ductile and brittle cases. Once again, since the "COV" of θ_{max} capacity is likely different for the model structure with ductile versus that with brittle connections, a direct comparison of the results for the ductile and brittle cases cannot be made. Nevertheless, for a single value of the θ_{max} capacity "COV", the design spectral acceleration corresponding to a target failure probability, or alternatively, the failure probability associated with a particular design spectral acceleration, is larger for the model structure with ductile connections since it is expected to be more reliable.

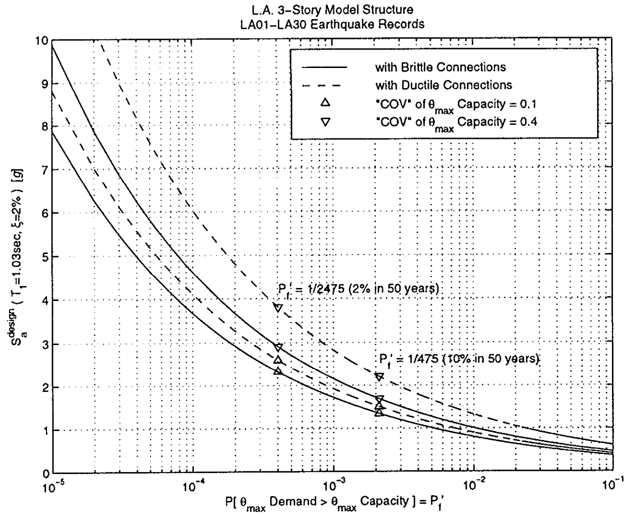

Figure 5. "Design spectral acceleration" versus target "probability of failure".

Dynamic Pushovers

As suggested for characterizing the maximum story drift angle capacity against collapse, dynamic pushover analyses are carried out for the three-story model structure, with ductile and with brittle connections, subjected to the ten 2% in 50 years ground motions. The resulting dynamic pushover curves are presented in Figure 6. Note that the dynamic pushover curves are reported in terms of S_a so as to facilitate comparison across different earthquake records.

For all but a few of the dynamic pushovers performed, the dynamic θ_{max} response of the model structure (with either ductile or brittle connections) remains "stable" up to values of θ_{max} beyond 10%, the limit corresponding to undependable analysis results. In this case, the median θ_{max} capacity may be estimated by taking the minimum of (a) the θ_{max} capacity obtained from each dynamic pushover curve, and (b) some fixed maximum value of θ_{max} capacity (e.g. 10%). However, if (as for this example) only a few of the θ_{max} capacity values estimated from the dynamic pushover curves are less than the prescribed maximum, the resulting "COV" of the capacity will be unrealistically small. Thus in this example, the probability of failure must simply be reported parametrically, for a range of median θ_{max} capacities and θ_{max} capacity "COV" values. It is possible that by improving the analysis model in the large deformation range (i.e., near collapse), dynamic pushovers may become an effective method for estimating the "dynamic θ_{max} capacity". Further studies are necessary to confirm this possibility.

The dynamic pushovers can still be used to study the effect of brittle connection behavior on the nonlinear θ_{max} response of the model structure. A comparison of the dynamic pushover curves for the model structure with ductile and with brittle connections illustrates that the increase in θ_{max} response is more pronounced for larger intensity ground motions (i.e., larger S,). As already noted, the increase in response from the ductile case to the brittle case (for the model structure under consideration) is relatively small for the original, unscaled earthquake records.

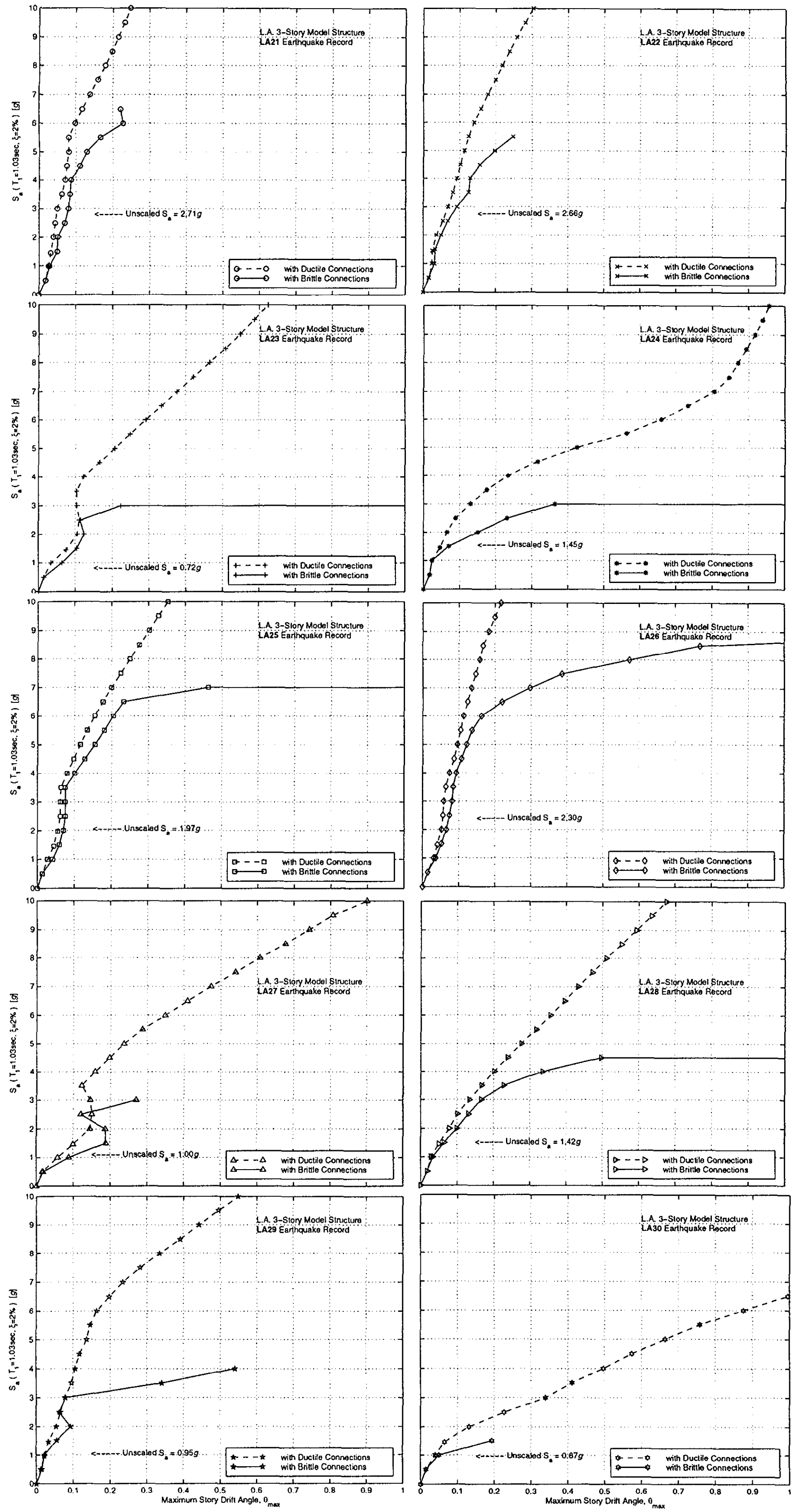

Figure 6. Dynamic pushover curves for the model structure with ductile and with brittle connections.

Conclusions

The procedure presented and demonstrated in this paper can be used not only to evaluate the seismic reliability of a model structure, but also to quantify the effects of brittle connection behavior on the performance of a structure. For the pre-Northridge three-story SMRF structure considered as an example, connection fractures appear to have a modest effect on the drift demand and the drift demand hazard for larger intensity ground motions (as measured by spectral acceleration), when in general the drift demand is larger. The effect of connection fractures is less pronounced for smaller intensity ground motions and drift demands. A comparison of the probabilities of failure for the ductile and brittle cases awaits assessment of the dynamic story drift capacity. Obviously, many more structures, fracture parameter values, and ground motions must be, and are in the progress of being, considered before these results can be generalized.

Acknowledgements

The work forming the basis for this paper was conducted pursuant to a contract with the Federal Emergency Management Agency (FEMA) through the SAC Joint Venture, and with the partial support of the National Science Foundation (NSF contract CMS-94-23596). SAC is a partnership of the Structural Engineers Association of California, the Applied Technology Council, and the California Universities for Research in Earthquake Engineering. The collaboration of Akshay Gupta and Helmut Krawinkler, also of SAC, is much appreciated.

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