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RELIABILITY-BASED EARTHQUAKE-RESISTANT DESIGN; THE FUTURE

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ABSTRACT

Reliability-based seismic design is implicit in many current design provisions despite the current limitations in the dynamic analysis methods employed, in the correlations between predicted structural response measures, in the accuracy of the often implicit reliability model, and in the explicit specifications of design and safety criteria. The future will surely remove many of these limitations and improve drastically the speed and facility of the computation of nonlinear response. The paper presents the current and future opportunities for improved estimation the mean frequency of a structure's experiencing any specified nonlinear limit state. The focus is on the interface between the characterizations of the seismicity and the nonlinear structural behavior..

KEYWORDS

probabilistic; reliability; nonlinear; multi-degree of freedom; ductility

INTRODUCTION

The objective of this paper is to present a personal view of the needs and possible future path of project-specific earthquake resistant design and re-assessment based on explicit reliability analysis. This cannot be done without effective estimates of the likelihoods of loss- and injury-inducing structural behavior. The earthquake community has been the most forthright with respect to estimating the site-specific probabilities of future structural loading levels at and beyond linear behavior thresholds. The use of explicit analysis of nonlinear dynamic behavior of the particular structure in question under these loads has been growing, first in the more resource-intensive projects such as offshore platforms and nuclear power plants, and more recently in the building field. Primary interest here is on how scientists and engineers might use and improve such studies to compute the annual frequencies of certain key states of behavior of a given structure. This implies a focus in the paper on the interface between seismology and structural engineering. The first portion of the paper will concern itself, however, with the other needs of an effective future probabilistic seismic design process. These include improved analysis tools and profession-wide policy developments.

Among others, the limitations of this presentation include the following: (1) While using the word "design", the focus here is on assessment or analysis and *structure-specific* analysis at that. Although this capability is the necessary first step, it is non-trivial to develop iterative *design* procedures that use this analysis capability and, just as importantly, simplified, faster, more economical, and more direct analysis schemes to facilitate the creative part of design for seismic loads In addition, until we obtain at least a one-to-two-order of magnitude improvement in computation and in its interfaces, together with proper education, some structural projects will continue to require less elaborate processes than those envisioned here. These projects will undoubtedly continue to depend on generic estimates of nonlinear dynamic behavior. (2) Left out of this presentation are important seismic problems in foundations and geotechnical engineering. (3) Reliability analysts will fault the paper for its limited concern with the randomness of the structural properties; allowance is made in the reliability formulation for such variabilities but their estimation is left for another time; they are of *comparatively* minor importance in the seismic reliability problem except in certain cases where seismic hazard curves are unusually steep, e.g., longer return periods at active plate margins (e.g., Cornell, 1996). (4) The important reliability question of (epistemic) uncertainty, i.e., uncertainty introduced by contemporary limits in the professional knowledge, whether about the seismic hazard or the structural response and capacities, is treated here only tangentially. Again formal allowance is made for it but its estimation is ignored here. This estimation has been a major topic in modern seismic hazard analysis (e.g., Coppersmith and Youngs, 1986, and SHACC, 1995) where uncertainty will always remain a significant factor; in contrast it can be reduced to arbitrarily low (i.e., comparatively low) levels in the structural response, behavior and capacity side of the safety equation by sufficient investment in generic and/or project-specific research and testing. In this structural case the issue of investment in reducing such uncertainty becomes a single element in the larger economic question that sophisticated (near-future?) business decision analysts (whether they work for the engineers, the owner or the insurance company) will consider as part of their analysis of the "value of information" (e.g., Howard, 1984).

PREREQUISITES FOR EFFECTIVE FUTURE PRACTICE

Presumed Improvements in Seismic Engineering

There are several important advances to structural engineering that the future will surely bring whether or not reliability-based project design is used, but they are presumed here and they are as important to a probabilistically centered analysis as to a "deterministic" one, so they should be mentioned. The first is greatly improved computational capabilities. These include not only speed and problem size, but more importantly software and interfaces that facilitate in many ways (ease, speed, understanding, checking, etc.) the process of constructing and manipulating structural models, creating and modifying the mechanical model, conducting systematic, repeated analyses under multiple inputs, sensitivity studies, presenting output and diagnostics, etc. These tools will improve any engineering procedure. Current trends in our fields and others insure that they will appear. Such prophesies for the future of structural computation have been being made since the 1960's at least. Looking back over the past 30 years of the growth of computation in structural engineering, perhaps the only concern for the future one might have is that major investments of resources (including engineers' formal and informal educational time) in existing software may have some tendency, computer enthusiasts' boasts to the contrary, to freeze or at least inhibit the rate of change of structural engineering practice. Costs of updating (software and its users) are often heard as reasons inhibiting the use of LRFD in steel design or nonlinear dynamic analyses in building studies. Yet investment in the development of advanced commercial-quality software and accessibly low prices for such programs both depend on the existence of a significant market. The current surge in interest in nonlinear (static at least) seismic analysis of buildings in California finds most firms dependent on an informal university 2D frame code that is two decades old.

Other developments certain to come are improvements in the structural mechanical modeling of nonlinear behavior under extreme seismic loadings. These improvements must include verification and/or calibration via laboratory and field tests. One hears engineers who have looked at the results of the Northridge event say that our nonlinear predictions must be inherently conservative or there would have been much more building damage under such widespread high ground motions, some more than 150% of the "implied" code capacity. At the same time we have the profession's moment-resisting steel frame embarrassment. Whether used in a deterministic or probabilistic format, our analysis capabilities and our confidence in them will surely improve, hopefully rapidly, in the future. The apparent U.S. renewal of interest in structural testing is evidence of this. Again a cynic might look to the rapid growth of structural computation as past cause of any current research imbalance between physical and analytical modeling; if so the future can expect to be an improvement as nonlinear design bases become more common.

Response-Loss Correlation

The benefits of a reliability basis for seismic design will be improved if we can establish more dependable correlations between, on one hand, those quantities structural analyses and seismic engineers are prepared to predict, such as global and local structural displacements, cumulative hysteretic energy, etc., and, on the other, the quantities that will appear more logically in future criteria developments. These include economic losses due to damage induced, repair costs, and loss of occupancy. They also include estimates of the likelihood of injury or death to the occupants. This is likely to be a far more empirical science than that which engineers are accustomed to, but the relevance to seismic design decision making is obvious. Again this paper will not pretend to cover what is being done now, the economic incentives driving its ongoing improvements, or how future research should evolve. As with structural behavior, under the right circumstances there will be calculatable benefits to reducing our current uncertainty in these correlations, and therefore economic incentive will make the requisite resources available from investors or their insurers to improve these links. Attention below will be restricted to the prediction of probabilities of structural displacements or other mechanically accessible quantities, but it will be presumed that the translation from them to loss estimates is direct and sufficiently accurate.

Criteria Development

Once one has made the effort to estimate the "probability of failure" or, more generally here, the probabilities of each of multiple levels of global and local nonlinear structural behavior, the question inevitably arises as to what is the "acceptable failure probability"? This is a narrow portion of the more general responsibility that the profession, the owners, and the occupants and public (or their representatives) have to establish meaningful policies and criteria with respect to seismic design. With the introduction of the capability to calculate reasonably reliable estimates that any proposed design will lead to severe seismic damage states, important economic losses, or injuries and deaths, the definition of these responsibilities changes.

We have been able in the past to afford criteria that are merely technical and indirectly expressed (as assorted factors and standardized procedures). With the introduction of estimated failure probabilities some criteria development has already moved toward specifying limits on these probabilities of unsuccessful performance. Examples include the Norwegian Petroleum Directorate (NPD, 1993) and the U.S. Department of Energy (U.S. DOE, 1994). The latter seismic criteria set five performance categories with levels of "performance goals" that associate an undesirable structural state with a annual "target" reliability, depending on the system or component's function and the consequences of its failure to remain within the stated behavior limits. For example, a maximum (mean estimate of the) failure probability is 10^{-4} if the limit state is breach of a (non-energetic) hazardous material confinement boundary of a toxic materials facility (Performance Category 3). The staff and the commissioners of the U.S. Nuclear Regulatory Commission have been wrestling for a decade with the question of Quantitative Safety Goals and their use with respect to engineering criteria (Okrent, 1987). A clear statement of the general issues involved has recently appeared in the structural literature (Paté-Cornell, 1994). Both individual life safety constraints and project-level cost-benefit-risk considerations are desirable elements of structural safety criteria. Even if codes were not to be written in such explicit terms it is likely that investor/insurer economic analyses of the facility will ultimately demand such information. Finally, as is beginning to be practiced in some projects, structural optimization studies may well be more routinely based on trade-offs that include cost versus reliability, within life safety constraints, of course.

We should assume then that in the future there will be safety criteria and/or project-specific analyses that imply a need for structural engineers to provide estimates of quantities such as the mean annual frequency of an arbitrary occupant becoming a fatality due to seismically induced structural behavior and the annual probability that the structure will suffer each of several levels of damage (per square meter, say). Given the correlation developments anticipated in the previous section, we assume below that it is necessary and sufficient here to explore (only) how the future might evolve with respect to estimating the annual frequencies of various levels of nonlinear structural behavior measures such as ductility and cumulative energy.

COMPUTATION OF STRUCTURAL STATE FREQUENCIES

Attention is now shifted to the calculation of the annual frequency of any given level of nonlinear structural performance. To set the stage, this includes a *brief* summary of how this is done in various forms of the U.S. current state of practice and of the near-term state of the art. How these processes might well in the future is the subject of the remainder of the section, particularly with reference to the interface between seismology and structural engineering.

Current Practice

Current U.S. building design practice involves no explicit estimates of the likelihoods of the structure experiencing specified states of damage. It is based on design ground motions with a specified mean frequency of exceedance, approximately 0.002 per annum, plus linear analysis and an allowable stress design process to a set of forces reduced by factors (R) that reflect the implicit member ultimate-strength/allowable-strength ratios, implicit non-linear static "overstrength" ratios (Ω), and implicit nonlinear dynamic factors (called here F_μ) that represent the system's ability to sustain nonlinear deformations without "life threatening" behavior (Popov et al., 1994). It has been estimated (U.S. DOE, 1994) that the net effect of these conventional design criteria is to produce a frequency of collapse that is about a factor of 2 below that of the design ground motion, or 0.001.

Other current seismic design norms include those for offshore structures (API,1993) which include both a lower linear design basis (involving a ground motion with a 0.005 annual frequency of exceedance) and, more interestingly, a "Ductility Level" check that is based on "best estimate values" of all parameters, fully non-linear dynamic analysis, and an "extremely rare event" ground motion, with a mean return period of "thousands of years"; assume this number is 2000 years implying an annual frequency of exceedance of 0.0005. Then assuming the best estimate objective is met and an estimate of the system's median capacity with respect to collapse is just equal to this ground motion level, the annual frequency of collapse implied is about 0.0005 times a "correction factor" $\exp[(k_1\delta)^2/2]$, in which k_1 is the local slope of the hazard curve (on log-log paper) and δ is the coefficient of variation of the system capacity (see, e.g., Cornell, 1996). This correction factor may be very little larger than unity in some cases, but it grows in active plate boundary regions to 2 or even more, especially for high reliability, critical facilities for which the design ground motion implied by the median capacity is already one with a very small frequency of exceedance.

It is clearly advantageous, however, as these offshore guidelines do, to use a return period for the design basis ground motion that is close to the target frequency of failure. If, in contrast, the *only* design basis return period (recall the API has, in effect, two levels) is reduced to, say, 200 years, the load factor necessary to achieve a median capacity such that the failure frequency is, say, 1/2000 can be shown (from the results in Cornell, 1996) to be about$[2000/200]^{1/k_1}\exp[(k_1\delta)^2/2]$ or approximately $10^{1/k_1}$. This factor depends strongly on the hazard curve slope k_1 which in turn depends on site, structural period, and the level of the target failure probability. Such a scheme is much more sensitive to these problem parameters than that of the API which includes a higher-return-period design-basis event. Further the low-return-period design basis procedure moves the design-basis response to the linear domain and depends on more or less *ad hoc* factors (non-structure-specific, in any case) to adjust from linear to nonlinear predictions of behavior near failure. But even if the design criteria used are "unbiased", that is, produce a nonlinear-analysis-based median capacity with no intentional conservatism remaining, the previous paragraph showed that the specified frequency of exceedance of the design ground motion used to establish that capacity does *not* equal nor even uniquely determine the frequency of the limit state.

We conclude reluctantly that we cannot expect to achieve specified target reliabilities using code formats of the type in current widespread use

.Current State of Art

The limitations of current practice with respect to providing designs with uniform specified target reliabilities are becoming widely appreciated. Further there is a growing demand to provide schemes that deliver information becoming widely appreciated. Further there is a growing demand to provide schemes that deliver information about multiple levels of behavior (from minor damage to collapse) and associated frequencies of non-compliance. There are several ATC-housed developments underway in the U.S. aimed at improving structure-specific predictive capability across the range of building behavior; they are based on static nonlinear "push-over" analyses coupled with simple, e.g., SDOF, generic estimates of the dynamic element of nonlinear seismic response (see, for example, Krawinkler, 1996). Current drafts associate each level of behavior with a mean return period for the corresponding ground motion. While a clear improvement in many dimensions, from a consistent-reliability basis these draft procedures are likely to suffer still from the concerns outlined above. As demonstrated there, this will be true to some degree even if all the intentional conservatism is "wrung out" of the capacity assessment criteria, which is difficult to implement in the short term because it requires an important re-education and attitude adjustment on the part of the engineer-users.

Other new developments designed to overcome the reliability difficulties identified above in current codes are guidelines derived (in appendices or in background documents) *explicitly* from a reliability model, but modified in their format for conventional application to make use of existing (conservatively biased) structural checking criteria. The first are the seismic guidelines developed for the U.S. Department of Energy, which has responsibility for many existing structures ranging from conventional office buildings through toxic and high-level radioactive materials handling facilities to nuclear reactors (U.S. DOE, 1994). Therefore the agency sought a procedure which recognized the widely different critical behavior states (e.g., life threatening damage in a building or loss of containment of a pressurized gas) and highly different consequences of failure (and hence different target failure frequencies). (Unlike the multi-level building criteria discussed above, in this case there is still only a single criterion for each structure, system or component under consideration.) In addition, practical immediate professional application dictated that the structural criteria be of a familiar kind, in particular *not* based on structure-specific non-linear dynamic analysis nor on an objective of defining an unbiased or median capacity. The approach used (Kennedy and Short, 1994) was: given a specified target failure frequency and given a set of structural criteria (either the UBC in the case of conventional structures or modified commercial nuclear power plant criteria in the case of the more hazardous facilities), calculate the ground motion return period just sufficient to achieve the target reliability. This required an efficient reliability analysis model, estimates of the median capacities implied by the criteria, and estimates of the coefficients of variation of those capacities. These estimates were provided from experience derived in the conduct over the past 20 years of probabilistic seismic risk assessments of commercial nuclear power plants. The reliability model employed is the same one referred to above (Cornell, 1996). The guidelines depend on linear analysis and sets of member-level generic factors (F_{μ}) to account for nonlinear behavior of buildings. (An additional procedure is outlined for special studies that might require nonlinear dynamic analysis.) The design basis ground motion has a resulting return period that is a factor of 2 to 20 larger than the reciprocal of the target failure frequency. The concern about the sensitivity to the slope of the hazard curve was studied with some care and adjustments made where necessary.

A second such procedure now in draft form is a revised version of the API LRFD seismic provisions re-designed in part as international (ISO) standards for seismic assessment of offshore structures (Craig, 1996). In this case too multiple consequence and target failure frequency levels are now identified and then used in the background with an explicit reliability model (here a "lognormal-lognormal scheme") to derive load factor values to be used with a fixed design basis return period (200 years again). This is a linear analysis basis with a system factor to reflect its ductility in the nonlinear range. The hazard curve slope difficulty is dealt with by a regional or site-specific coefficient of variation that reflects the slope. Again provisions are made for an explicit nonlinear dynamic analysis check under certain conditions and ground motion levels that are not well defined as yet.

<u>Future, but Currently Feasible, Practice</u>

All of the desirable prerequisites specified above need not be completed before substantial improvements in the estimation of the frequencies of various levels of post-linear behavior of a specific structure are feasible in practice. In looking even a short distance into the future we can assume that computation will be an order of magnitude faster and cheaper. Even now it is feasible in more special projects to conduct nonlinear time history analyses of a best-estimate model of the structure subjected to a small suite of accelerograms. If these records are reasonably carefully selected and if they run at several scaled levels, starting at a level near the onset of adequately captured to permit structure-specific estimates of the annual frequencies of each of various levels of behavior. These results can be compared with targets to confirm whether or not the structure meets these objectives. If not, re-design is in order.

In this scheme (Bazzurro and Cornell, 1994a and b) the seismic hazard at the site is characterized by the hazard curve of the spectral acceleration at a "reference period", T_{ref}, that many times is simply that of the first mode of elastic response of the structure. The nonlinear response of the structure as obtained from the analyses described above is captured in a set of "F_μ versus μ" curves. These curves are estimates of the *average* scale factor, $\overline{F_\mu}$, necessary to induce some specified level of nonlinear behavior or damage, e.g., global (top story) ductility level μ. Fig. 1 shows a typical set of such curves for the top story ductility and the first-story interstory drift ductility of a 2D moment-resisting steel-frame structure designed to UBC Zone 4 (Collins et al., 1995). In this case the average is taken over 20 records, but 4 or 5 are usually sufficient given that the coefficient of variation of F_μ is seldom more than 30%, which, when divided by the square root of the number of records to give the standard error of estimation of the mean, is small compared to other such coefficients in the total problem. These curves can be prepared from post-processing of the nonlinear runs to display any global, story-level, cross-section level or even fiber-level behavior versus the scale factor. It is helpful to present the plots in this normalized way in order to compare them with one another and with those of other structures, including SDOF representations of the system. The large number of records used in this case make it possible to obtain, in addition, confident estimates of the coefficients of variation (record-to-record) of the scale factors at each specified μ level. For the global ductility they range from 25% to 36% as μ goes from 2 to 4.

With this information available it is a straightforward matter to estimate the frequency p_f that a specified loss or damage state is equalled or exceeded (Cornell, 1996, modified slightly in its interpretation for the present purposes):

$$p_f \cong P[S_a \geq F_R(\hat{\mu}_C) \cdot S_{a_{ref}} \cdot \varepsilon_{\mu_C}] \tag{1}$$

in which S_A is the future ground motion intensity as measured by the spectral acceleration at a reference structural period, $S_{a_{ref}}$ is the ("mildly" random) spectral acceleration at which the system experiences first significant nonlinear behavior, $F(\hat{\mu}_C)$ is the (random) scale factor discussed above for the median (best estimate) ductility level, $\hat{\mu}_C$, that is associated with the damage state and ϵ_{μ_C} is the random quantity reflecting the uncertainty in the damage state-ductility relationship. Then it can be shown that

$$p_f \cong H(\hat{R})\, e^{1/2(k_1 \delta_R)^2} \tag{2}$$

in which $H(\hat{R})$ is the seismic hazard curve for the relevant spectral acceleration evaluated at the median "resistance" to such a damage state, $\hat{R} = \hat{F}_R(\hat{\mu}_C) \cdot \hat{S}_{a_{ref}} \cdot \hat{\varepsilon}_C$, in which the "hat" denotes median value. δ_R is the coefficient of variation of that resistance. In terms of the same coefficients of the three variables on which it depends:

$$\delta_R \cong \sqrt{\delta_{F_R}^2 + \delta_{S_{a_{ref}}}^2 + \delta_{\varepsilon_C}^2} \tag{3}$$

See Cornell, 1996, for a discussion of assumptions, details, interpretations, and illustrative values.

This analysis depends for its simplicity on the assumption that there is no systematic dependence of $\hat{F}_R(\mu)$ on the magnitude and distance of the earthquake. Although this assumption has been shown empirically to be accurate for many simple and a few MDOF systems (e.g., Sewell, 1988, and Bazzurro and Cornell, 1994a), it is always wise to select records for dynamic analyses that are representative of the events that contribute most to the threat at the site. Improved recommendations and methods for doing this are becoming available, e.g., McGuire, 1995. (See also below.) This remains a comparatively soft link in this method's interface between the seismicity and the structural analysis however. The critical interface variable in this method, as with all seismic analysis today, remains the elastic spectral acceleration at some single or set of structural periods.

Finally this approach, although deformation (ductility) or other damage-measure-based, can be couched in a more conventional force-based resistance format by solving Eq. 2 for the necessary $\hat{R}$ to give the specified reliability $(1 - p_f)$ with respect to any given ductility level or damage state (Cornell, 1996). If preferable for practice, this equation can be simply modified to provide the basis for a more familiar format based on a reference or "characteristic" resistance less than the median (e.g., a specified lower fractile) and/or on a design basis ground motion with a mean return period less than one over the target frequency of failure. As discussed above, however, one must be careful in such modifications to deal with any hazard curve-slope-dependence this latter step might introduce.

The Future: Structure-dependent PSHA

As stated above, it is presumed that at some future point in time computation, mechanics and criteria will have developed such that the primary need of reliability-based seismic design will be the accurate estimation of the annual frequencies of the structure falling into various levels or states of behavior that induce loss, concern, injury, etc., which, in turn, can be sufficiently accurately predicted by correlation with some structural mechanical quantity such as nonlinear deformation. Observation of the evolution of the characterization of earthquake records and of the efforts to predict the these characteristics, suggests the following future to this author. From the earth sciences perspective probabilistic recurrence analysis will produce better and better and more complete estimates of the frequencies of occurrence of events with specified source characteristics. The ground motion estimation efforts will focus more and more on (empirical and theoretical, probabilistic) prediction of the entire time history of the accelerogram given these source characteristics.

What then will happen to the interface between this input and the necessary output, i.e., annual frequencies of various nonlinear structural behavior states? As mentioned above, most current practice uses a spectral acceleration level from a (mapped or site-specific) probabilistic seismic hazard analysis (PSHA) together with various factors, whether *ad hoc* or simple-system-based (e.g., generic nonlinear dynamic SDOF results and/or static push-over models), to predict the nonlinear dynamic responses of buildings to ground motions. When structure-specific nonlinear dynamic analyses *are* done, the process is largely decoupled from the seismicity assessment. At best the current approaches involve selecting one or more "representative events" (i.e., scenario earthquake sizes and locations) and then records "representative" of these events. The bases for the scenario selections are, again at best, based on linear SDOF PSHA results. The implicit assumptions in all these exercises are seldom stated or investigated. These include assumptions about the lack of dependence of nonlinear structural behavior on the source characteristics (e.g., rupture duration) and geometry of the fault relative to the site These are assumptions few would accept without proof to the contrary. While, as suggested above, some empirical evidence suggests less sensitivity than many might expect, there are too many independent variables in the problem (source, path, structure, response measure, etc.) to ever "prove" independence even it were true.

The proposed solution is therefore "structure-specific" PSHA. The seismic hazard equation would become (under the usual assumptions):

$$\lambda(\mu) = \sum \nu_i E_{M_i R_i}[\Phi'[(\ln\mu - \ln\hat{\mu}(M_i,R_i)/\sigma_{\ln\mu}]] \qquad (4)$$

in which $\lambda(\mu)$ is the mean annual frequency of events which cause (for example) global ductility greater than μ in the structure at hand, ν_i is the mean annual rate of earthquakes of interest in source i, $E_{M,R}[...]$ indicates expectation with respect to the subscripted variables, magnitude and distance, $\Phi'(...)$ is the complementary standardized gaussian cumulative distribution function, $\ln\hat{\mu}(M,R)$ is the regression of log μ on magnitude and distance, and $\sigma_{\ln\mu}$ is the standard deviation of the log of μ given magnitude and distance.

The only missing ingredient in Eq. 4 is a set of attenuation laws that predict the nonlinear response (various ductilities, local hysteretic energies, etc.) of the particular structure at hand as a function of magnitude and distance (or other relevant source, path or site parameters). The direct way to obtain such a law is to calculate such structure-specific nonlinear response results for a large suite of accelerograms with different magnitudes and distances and then to perform empirical regressions. This is not as absurd as it may appear. This is exactly what

McGuire did for a large suite of linear SDOF systems in 1974 (McGuire, 1974) and what Sewell did in 1988 for many nonlinear SDOF systems (Sewell, 1988) with far less computational power than we have today, much less in the future. Further, various simplifications can be imagined; for example, the attenuation law for the global ductility can be written as

$$\mu(M,R) = S^{-1}_{A,\,inelastic}(S_{A_{yield}};M,R) \tag{5}$$

in which $S_{A_{yield}}$ is the spectral acceleration at which yield, or first significant nonlinearity, is expected and $S^{-1}_{A,\,inelastic}(x:;M,R)$ is the *inverse* function with respect to μ of the attenuation law of the so-called inelastic spectral acceleration, $S_{A,\,inelastic}(\mu;M,R)$ (Sewell, 1988, and Inoue, 1990). But in the development of Eq. 1 and Eq. 2 this latter attenuation law is implicitly the usual attenuation law for elastic spectral acceleration, $S_A(M,R)$, divided by $F_R(\mu)$. The underlying assumption made in these cases is that $F_R(\mu)$ is independent of magnitude and distance. As mentioned this assumption has been investigated empirically for some SDOF and MDOF cases and found to be a reasonable first approximation, for ductility and certain cumulative hysteretic energy measures, at least. Thus, given the ready availability of *elastic* spectral acceleration attenuation laws, establishing the attenuation law for μ in this case was limited to *confirming lack of dependence* of the *structure-specific* $F_R(\mu)$ on magnitude and distance, a much easier task than establishing its dependence. The result is that the dependence of the structure-specific μ on magnitude and distance is identical to that of the "underlying" elastic spectral acceleration. More generally this dependence will have to be estimated on a case by case basis.

An advantage of this approach that some will find attractive is the stronger separation of seismological and structural assessments. The structural engineer asks the strong motion seismologist for nothing more than accerograms (for any given magnitude and distance); no through-structure processing (e.g., spectral acceleration estimation) by the seismologist is required and no *a priori* judgements by the seismologist about what characteristics matter to the structure are necessary. The coupling of the recurrence and structural effect attenuation (associated with the integrations implied by Eq. 5) would have to be conducted by the seismologist and engineer in concert, however.

Still greater separation can be obtained, however. Fig. 2 shows a new concept, the seismic hazard contour (Bazzurro et al.,1996). It is a "construct" of PSHA that is unique to a site for a specified annual frequency of exceedance but it is *independent* of the attenuation law. In this specific example the contour is the 0.001 contour for a site located 45 km from the center of a straight 650 km fault whose magnitude recurrence curve is of the characteristic magnitude type. Events greater than or equal to magnitude 5 occur with mean frequency 0.01 per year. Their distribution is exponential with a b value of 0.64 through magnitude 7.0. Characteristic events are presumed to be uniformly distributed between 7.0 and 7.5 with a mean frequency of 0.001 per year. The implication is that such a contour can be provided once and for all by the seismologist as a representation of the seismicity (recurrence and geometry) in the region surrounding the site. The structural engineer can then use this contour as follows for *all* structure specific applications. Shown in the figure, for illustration only, is a line of iso-PGA associated with the Sadigh 1987 attenuation law for PGA (as reported in Joyner and Boore, 1988). Note that this particular level of PGA (0.106g) produces a point of tangency to the PSHA contour at magnitude 7.3 and distance 48 km. The method of construction of the contour (namely "inverse-FORM") implies that this PGA value is in fact that with a 0.001 annual frequency of excedance. It must be emphasized that the iso-PGA line is shown only for illustration; in practice the engineer need only evaluate the attenuation law at a small set of points *on* the contour searching for that point with the *largest* PGA. An important implication with respect to structure-specific PSHA is that the engineer need *not* determine an entire nonlinear response attenuation law for his structure; he needs only to estimate the (median) ductility at a small set of (M, R) pairs on the contour provided by the seismologist until he finds the pair with approximately the maximum ductility. This ductility value is the 1000-year mean return period value. This (M, R) is the preferred "design basis event" for ductility design to a 0.001 per annum criterion. Note that unlike methods described above this scenario event recognizes any magnitude or distance effect on the nonlinear behavior of the MDOF structure that a method based on linear spectral acceleration alone cannot. For greater accuracy the PSHA contour method should be corrected to account for the scatter in the data about any attenuation law; this can be done by at least three different methods

(Bazzurro et al., 1996) that, along with extensions to and implementations of the method, are under current investigation. One easy correction method ("omission factors") gives 0.130g; the exact answer being 0.134g.

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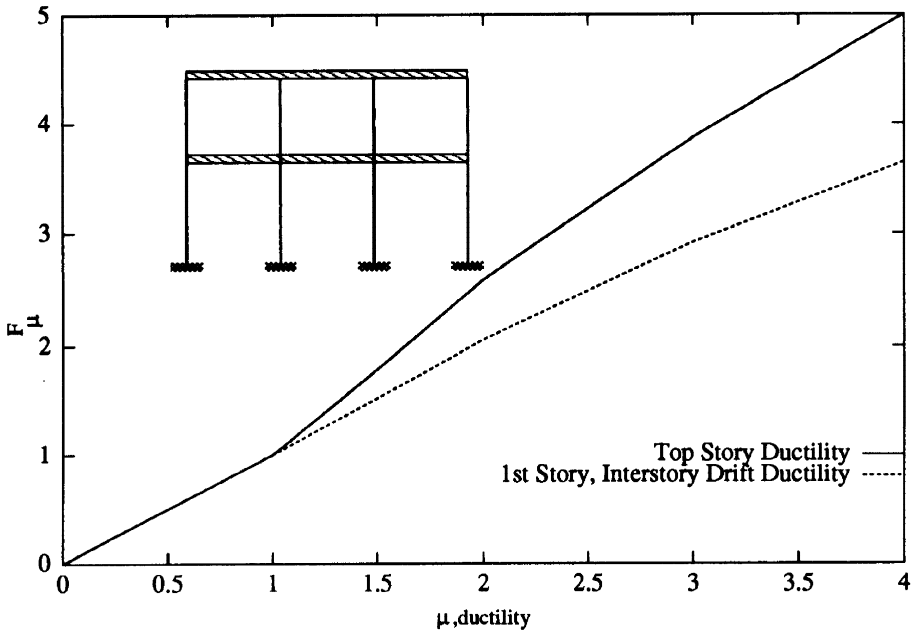

Fig.1 2D Steel MRF: Record Scale Factor($F_\mu = S_A/S_{A,\text{first nonlinearity}}$) versus Top Story Ductility and First-Story Interstory Drift Ductility

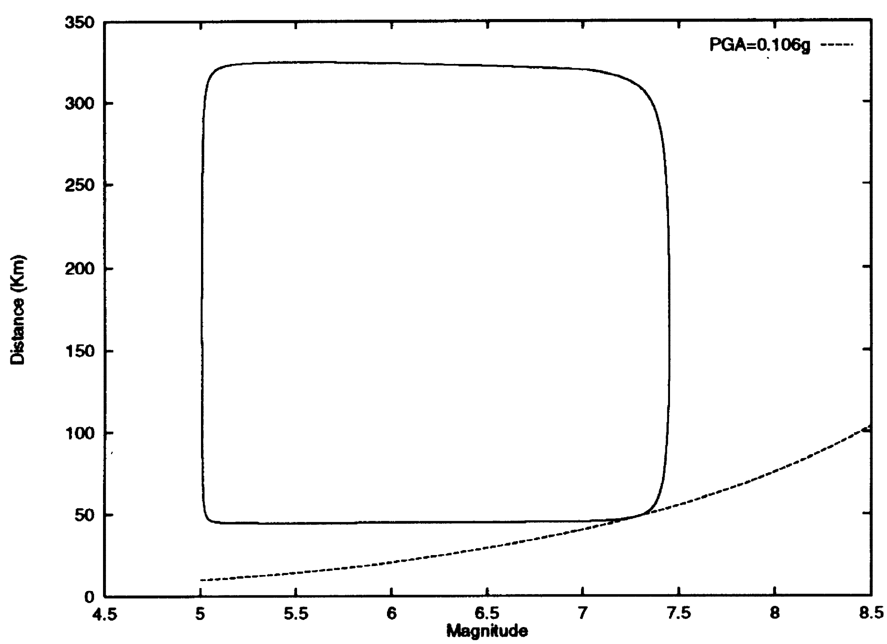

Fig.2 A Site-specific PSHA Contour for 0.001 Mean Annual Frequency of Exceedence