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# Reliability and performance-based design☆

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**Abstract**

Structural failures in recent earthquakes and hurricanes have exposed the weakness of current design procedures and shown the need for new concepts and methodologies for building performance evaluation and design. A central issue is proper consideration and treatment of the large uncertainty in the loadings and the complex building behavior in the nonlinear range in the evaluation and design process. A reliability-based framework for design is proposed for this purpose. Performance check of the structures is emphasized at two levels corresponding to incipient damage and incipient collapse. Minimum lifecycle cost criteria are proposed to arrive at optimal target reliability for performance-based design under multiple natural hazards. The issue of the structural redundancy under stochastic loads is also addressed. Effects of structural configuration, ductility capacity, 3-D motions, and uncertainty in demand versus capacity are investigated. A uniform-risk redundancy factor is proposed to ensure uniform reliability for structural systems of different degree of redundancy. The inconsistency of the reliability/redundancy factor in current codes is pointed out. 

*Keywords:* Hazards; Uncertainty; Reliability; Performance-based design; Lifecycle cost; Redundancy; Codes and standards

## 1. Introduction

The building industry suffered serious setbacks in recent natural hazards such as Northridge and Kobe earthquakes and hurricanes Hugo and Andrews. These events bring to focus the questions of how the structural engineering profession treats the large uncertainty in the natural hazards and structural capacity and what reliability existing buildings have against future hazards. Although the uncertainty of seismic and wind loads has been well recognized by the profession, the incorporation of uncertainty in most building code procedures has been limited to the selection of design loads based on return period. This design load is then used in conjunction

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**Nomenclature**

| | |
|---|---|
| $C$ | = total cost |
| $C_0$ | = initial cost |
| $C_j$ | = cost of $j$th limit state |
| $C_\mathrm{m}$ | = maintenance cost |
| $C_\mathrm{D}$ | = design capacity correction factor |
| $C_\mathrm{F}$ | = reliability correction factor |
| $\delta_R$ | = capacity coefficient of variation |
| EQ | = earthquake |
| $\lambda$ | = annual discount rate |
| $P_{ij}$ | = probability of $j$-th limit state during $i$-th occurrence of hazard |
| $P_\mathrm{f}$ | = probability of failure |
| $R$ | = response modification factor |
| $\rho$ | = NEHERP reliability/redundancy factor |
| $R_\mathrm{R}$ | = uniform-risk redundancy factor |
| $S$ | = sensitivity coefficient |
| $S_i(t)$ | = $i$th time-varying load process |
| $S_a$ | = spectral acceleration |
| $S_y$ | = structural system yield coefficient (system yield force /weight) |
| $t$ | = structural life |

with a series of factors reflecting the influence of structural period, loading characteristics, site soil condition, structural inelastic behavior, importance of the structure etc. These factors are largely determined based on judgment and experience and often calibrated in such a way that the resultant designs do not deviate significantly from the acceptable practice at the time. Several recent studies of the reliability of buildings designed for seismic loads according to current code procedures in US, Japan, and Taiwan have shown that these procedures can not be expected to satisfy reliability requirements [4,18,32]. There are large differences implied in the reliability, which can be attributed to the differences in hazard characteristics (e.g. seismcity, wind climate), design philosophy, construction practice, and code provisions (e.g. importance factor, ductility reduction factors, gust response factor). In view of the above, a statistical and reliability framework is needed for development of performance-based design in which the uncertainty in the demand and capacity, and balance of cost and benefit are properly considered. In the following, a framework for reliability and performance based design is proposed and some critical issues are addressed.

## 2. Critical issues in reliability and performance based design

### 2.1. Performance goal

The selection of the design earthquake and wind has a strong influence on the performance or reliability goals of the resultant designs. The actual reliability against a specific limit state, however, varies depending on the limit state and the conservatism built into the various factors in the codes. For example, the frequency of collapse has been estimated to be about a factor of 2 below that of the 500-year design ground motion for conventional buildings [17]. Adding more levels of design earthquakes for specific levels of performance [25] would better enforce the performance goals. The selection of the additional design earthquakes and corresponding performance goals, however, needs to be carefully done to assure internal consistency. For example, a 50% in 30 years (43-year) earthquake for serviceability check is found to be too conservative and may control the design [7].

To strictly enforce reliability performance goals, the target probabilities need to be set directly for the limit states rather than for the design earthquake [31]. In evaluation, the performance of the structure is satisfactory if the limit state probabilities are below the target values. In developing reliability-based design formats, one starts from this target reliability goals corresponding to physical limit states such as incipient damage and incipient collapse and develop the required deterministic design format, which will yield a design that satisfies these goals. An example of such bi-level performance acceptance criteria based on this procedure in terms of annual inter-story drift probabilities is shown in Fig. 1. Such a procedure, for example, has been proposed in the US SAC Steel Project [11,12,33].

### 2.2. Determination of reliability

A procedure is needed to determine the reliability for performance evaluation and design of structures whose members and connections may yield, fracture, and deteriorate in strength and/or stiffness such as under severe seismic ground motions. The well-known first order reliability method (FORM) has been successfully used as basis for reliability-based design formats in most recent code procedures. For structures under earthquake loads, however, the responses often become dynamic and nonlinear and have different hysteretic behaviors caused by yielding and brittle fractures of the components. The problem is generally not amenable to FORM since it is difficult to determine the limit state function under these circumstances. For problems of such complexity, the direct time history/simulation method may be more appropriate. To evaluate the reliability, however, one needs to generate a large number of ground motions that represent all future events in the surrounding region, which have an impact on the building [3]. It can become computationally unmanageable, especially for nonlinear analyses of structures of large sizes. To reduce the computation, a small number of earthquake ground motions can be generated using the values of seismic parameters that contribute most to the limit state probability. An example is the well-known "de-aggregation" method in which only the most likely combinations of magnitude and distance that contributes to a given limit state exceedance probability, is used in the simulation. This method obviously depends on the intelligent selection of the seismic parameters for a given limit state under consideration. The procedure is clearly structure and limit state dependent and difficult to apply to general building stock.

An alternative method that is applicable to a wider class of buildings and structures is to generate suites of ground motion corresponding to a given probability of exceedance. For example in the SAC Steel Project, recorded and broad-band simulated ground motions were scaled such that the median (50 percentile) values of the resulting elastic response spectra match those of a set of target spectra corresponding to a given probability of exceedance [23]. In the case of mid-America

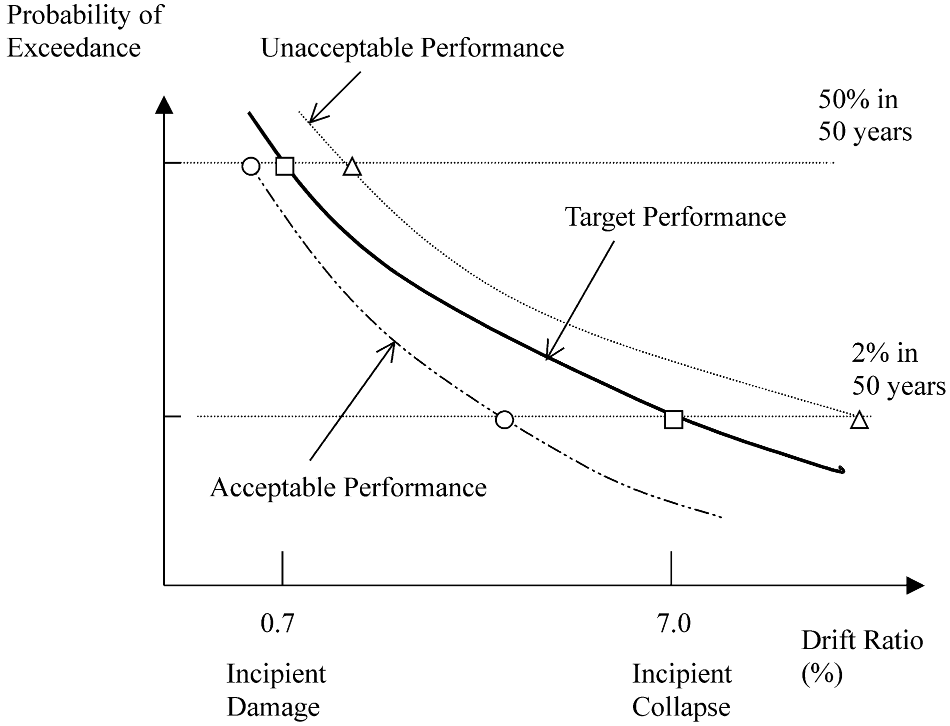



Fig. 1. Bi-level acceptance criteria in terms of 50-year limit state probability.

where records are scarce, Wu and Wen [35] generated suites of ten ground motions according to regional seismicity based on the latest ground motion models in which some of the important near-fault effects of large events can be included. These ground motions, therefore, represent what we expect to occur at these sites, corresponding to a specified level of probability of exceedance. Fig. 2 shows the 2% in 50 years target response spectra and the spectra of suite of ground motions for Memphis, TN. The median response spectral of the simulated ground motion is also shown. It fits the target well and has a very small coefficient of variation ($<10\%$). The structural response corresponding to the probability level can be obtained as the median response of structures under such suites of ground motions using a nonlinear time history analysis. This process is then repeated for several probability levels to obtain the performance curve shown in Fig. 1. The accuracy of this method for nonlinear inelastic systems has been proved to be excellent by comparison with results of full simulations by Wu and Wen [35] for several mid-America cities. Although the effect of capacity uncertainty can be included in the above analysis by randomizing the building capacity, there are strong arguments for accounting for this type of uncertainty by using a capacity uncertainty correction factor.

### 2.3. Capacity uncertainty

Capacity uncertainty can be attributed to material variability, non-structural component contribution to resistance such as claddings and partitions, and structural modeling errors such as the highly unpredictable brittle failure of steel connections. The total uncertainty (coefficient of

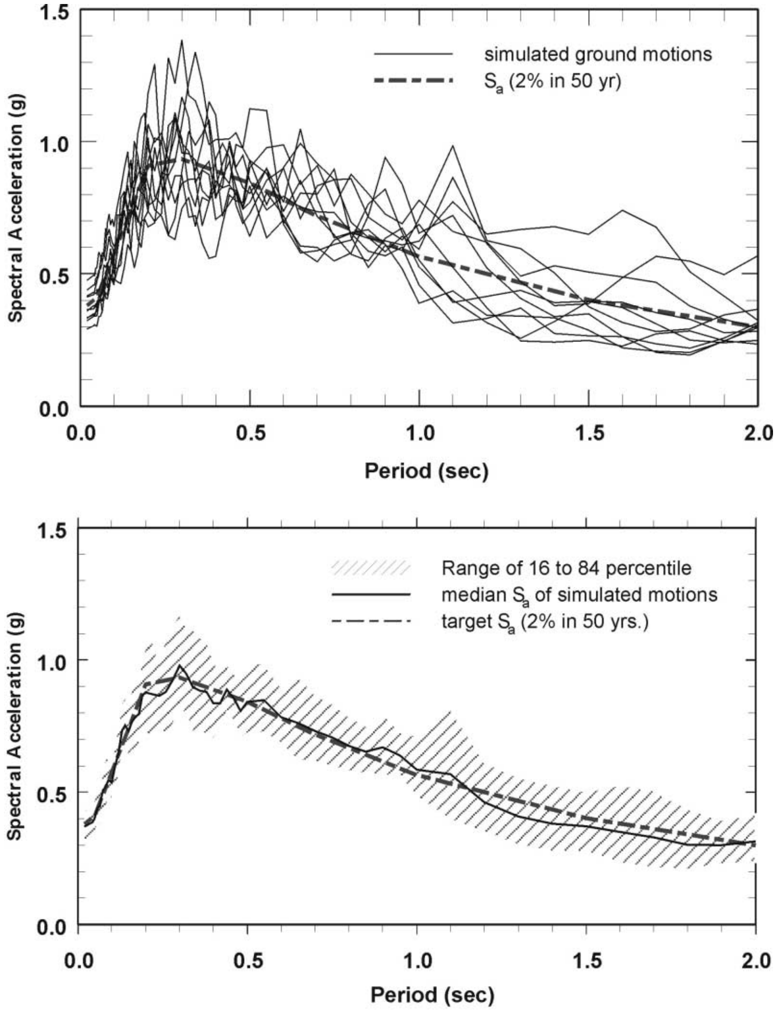



Fig. 2. Response spectra of 2% in 50 years ground motion suite and comparison with target 2% in 50 years response spectrum, Memphis, TN.

variation) on the capacity side has been estimated to be 40% or higher [4,26], which is significant in itself but still small compared with that of the seismic load which is generally in excess of 80%. An interesting implication is that because of this dominance of the uncertainty on the loading side, the effect of the uncertainties can be de-coupled as an approximation. As a result, one can first ignore the capacity uncertainty by using the mean value in the reliability analysis as is done in the above time history/simulation and recover it by a multiplier called capacity uncertainty correction factor. This correction factor can be used in both reliability analysis and reliability-based design. The effect of model and resistance uncertainty on reliability has been investigated systematically by Maes [19], and Cornell [4]. The general effect of modeling uncertainty is a decrease in reliability. It can be incorporated by a correction factor defined as the ratio of the limit state probabilities with and without consideration of the model uncertainty. It is a very convenient tool in both reliability evaluation and development of reliability-based design when model uncertainty is present but cannot be accurately quantified at the time. For example, one can treat the capacity uncertainty separately by first neglecting its effects in the reliability evaluation and design and recover it using the correction factor which can be continually updated as more information on model uncertainty becomes available. The method has been demonstrated [33] that the correction factor for reliability evaluation due to capacity uncertainty is

$$C_{\mathrm{F}} \approx 1 + \frac{1}{2} S^2 \delta_{\mathrm{R}}^2 \tag{1}$$

in which $S$ is the sensitivity coefficient depending on the seismic hazard and reliability level and $\delta_{\mathrm{R}}$ is the coefficient of variation of the capacity. $S$ generally increases with the uncertainty in the hazard and reliability level. In a reliability-based design, the correction factor for design capacity to incorporate capacity for a given target reliability is

$$C_{\mathrm{D}} \approx 1 + \frac{1}{2} S \delta_{\mathrm{R}}^2 \tag{2}$$

For example, for a 2-story steel frame at Los Angeles, the annual exceedance probability of a global (building) drift of 3.3% was found to be $5\times10^{-4}$ without considering capacity uncertainty. At this reliability level and at Los Angeles, $S=3.29$. Assuming a $\delta_{\mathrm{R}}$ of 36%, $C_{\mathrm{F}}=1.7$; i.e. the limit state probability increases by a factor of 1.7 to $8.5\times10^{-4}$, due to the effect of capacity uncertainty. In design, the capacity should be multiplied by a factor $C_{\mathrm{D}}=1.2$ to account for the effect of capacity uncertainty in order to maintain the same reliability level ignoring the capacity uncertainty.

### 2.4. *Target reliability level*

Determination of the target reliability level for various limit states in the above procedure requires broader social-economical considerations. The target reliability levels for different limit states can be determined by comparison of risks of limit states with other societal risks. Alternatively, one can compare the notional (calculated) probability of limit states with those implied in current designs and adjust accordingly. This approach has been used in the past. For example, Ellingwood et al. [5] calibrated the target reliability of structural members against practice acceptable at the time in developing the AISC LRFD design recommendations, which have been adopted in the ASCE-7 [1] document. The need for a more rational approach to determine target reliability and acceptable risk has received serious attention by researchers and engineers recently [6]. One of such approaches which looks promising is based on minimization of expected lifecycle cost as given in the following.

## 3. Minimum lifecycle cost design criteria

In the recently proposed multi-level performance-based design, e.g. SEAOC Vision 2000 document (1995) [25], and Building Standard Law in Japan [14], the selection of the seismic hazards and the corresponding structural performance levels has largely been based on professional experience and judgment. While collective professional wisdom may be the only recourse at present, it could lead to wasteful or unsafe designs. To strike a balance between the possible high initial cost and potential large losses over the structure's lifetime, the lifecycle cost and the uncertainty in the hazards and system capacity need to be carefully considered. The design procedure based on optimization considering cost and benefit is generally referred to as level IV reliability-based design. For example, Rosenblueth [21] had made strong and convincing arguments for the profession to move from a semi-probabilistic, second moment, or full distribution design format to one based on optimization since it is the only rational procedure to ensure long term benefit to the society. A method based on minimum lifecycle cost is proposed in the following. Details of this method and results can be found in Kang and Wen [16].

### 3.1. The optimization problem

The major considerations in a life cycle cost analysis of a constructed facility are loading and resistance uncertainties, limit states, and costs. The random occurrence in time and the intensity variability of the hazards are modeled by random process models following Wen [30]. Costs include those of construction, maintenance and operation, repair, damage and failure consequence (loss of revenue, deaths and injuries, etc.), and discounting of future loss/cost over time. It is reasonable to assume there are only a small number of limit states to be considered and the loadings that can cause the facility to reach these limit states are due to severe natural and man-made hazards which occur infrequently. Over a time period ($t$), which may be the design life of a new facility or the remaining life of a retrofitted facility, the expected total cost can be expressed as a function of $t$ and the design variable vector $X$ as follows:

$$E[C(t,X)] = C_0(X) + E[\sum_{i=1}^{N(t)}\sum_{j=1}^{k} C_j e^{-\lambda t_i} P_{ij}(X,t_i)] + \int_0^t C_{\mathrm{m}}(X) \mathrm{e}^{-\lambda\tau} \mathrm{d}\tau \tag{4}$$

in which $C_0$ = the construction cost for new or retrofitted facility; $X$ = design variable vector, e.g. design loads and resistance, or load and resistance factors associated with nominal design loads and resistance; $i$ = number occurrences and joint occurrences of different hazards such as live, wind, and seismic loads; $t_i$ = loading occurrence time; a random variable; $N(t)$ = total number of severe loading occurrences in $t$, a random variable; $C_j$ = cost in present dollar value of $j$th limit state being reached at time of the loading occurrence including costs of damage, repair, loss of service, and deaths and injuries; $e^{-\lambda t}$ = discounted factor of over time $t$, $\lambda$ = constant discount rate per year; $P_{ij}$ = probability of $j$th limit state being exceeded given the ith occurrence of a single hazard or joint occurrence of different hazards; $k$ = total number of limit states under consideration; and $C_{\mathrm{m}}$ = operation and maintenance cost per year. The design criteria are determined by minimization of the total expected lifecycle cost with respect to the design variable vector $X$. Additional constraints in the form of reliability and/or resistance may be also introduced in the minimization problem. The above formulation allows tractable solution of the minimization problem and facilitates the sensitivity study of the optimal design decision parameters as shown in the following.

### 3.2. Question of uniform reliability

Most civil systems are subjected to more than one load. When design for more than one load, one is frequently confronted with the difficult question of what level of reliability one should aim for each load. Is uniform reliability the logical choice? While such criteria have been suggested and used in the past, they may not be cost effective since it may be prohibitively costly to maintain high reliability against a low-probability, large-consequence, and large-uncertainty hazard such as earthquakes. For example, based on Ellingwood et al. [5], the 50-year structural member reliability index in current design for seismic loads was found to be approximately 1.75, which is much lower than those for winds (around 2.5) and dead plus live loads (3.5 or higher). It does not necessarily mean that current code seismic load provisions are inadequate against earthquakes. This issue will be addressed in the following.

### 3.3. Optimal design under single hazard

#### 3.3.1. Parametric study

A parametric study is carried out for a time-invariant system under a single hazard modeled by a Poisson process with an occurrence rate of $\nu$ per year and an exponential intensity. A closed form analytical solution of the expected life cycle cost given by Eq. (2) can be obtained as follows,

$$E[C(t, X)] = (aX + C)e^{-X}\nu\frac{1 - \mathrm{e}^{-\lambda t}}{\lambda} + aX \tag{5}$$

in which $aX$ is the initial cost function proportional to the design intensity $X$. Eq. (5) approaches the classical solution of Rosenblueth [21] as $t \Rightarrow \infty$. The close form solution allows easy sensitivity studies of the optimal design intensity to the load parameters, structural life and failure consequence. Fig. 3 shows the optimal design intensity (arbitrary unit) as a function of design life and cost of limit-state being reached (arbitrary unit). Under the condition that the failure cost $C = 20$, which is of the same order of the construction cost (e.g. considering only repair and replacement costs), the design intensity is 3.2 for a facility of a design life of 50 years. Compared with a design intensity of 1.2 for a life of 5 years, the reduction is almost a factor of three. On the other hand, when the failure cost $C = 100$, which is about five times of the construction cost (e.g. considering revenue loss, deaths and injuries), the design intensity reduces only from 4.4 to 3.0. The design intensity based on a criterion of equal lifetime probability of exceedance (10%) is also shown in the figure. It is seen that it would lead to under-design for a system of short life and high failure consequence and over-design for a system of long life and low failure consequences. Fig. 4 shows the dependence of design intensity of failure consequence. When the failure consequence is large, high design intensity is needed, even for facility with short design life. In this case, the additional initial cost ensures much less failure cost and hence saving in the long run. The results show that a rational, quantitative design decision can be made based on results of such a minimum lifecycle cost analysis and can not be obtained based on judgment and experience or consideration of probability alone.

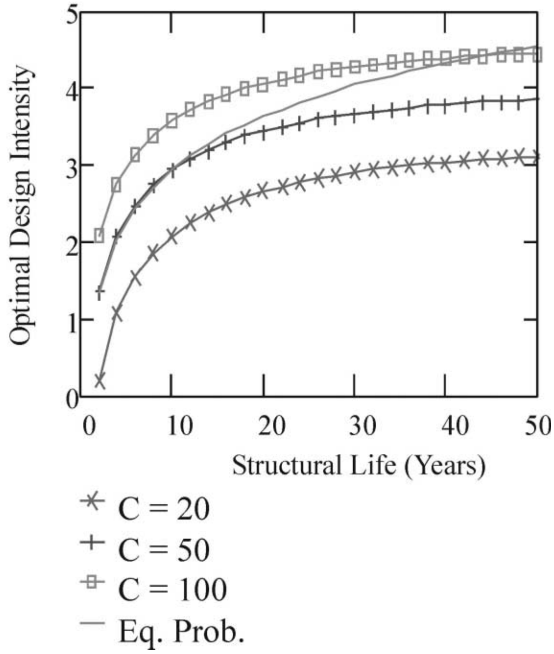



Fig. 3. Optimal design intensity as a function of structural life.

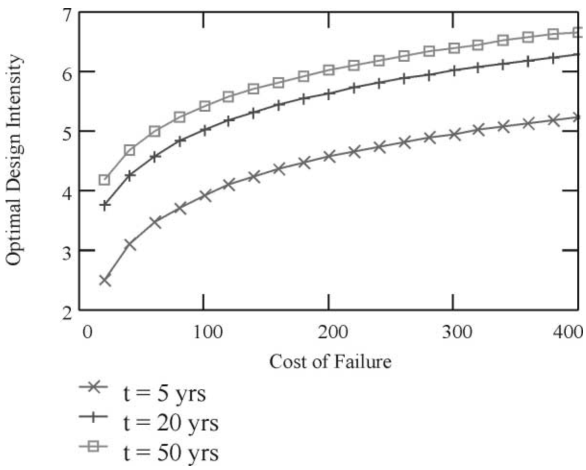



Fig. 4. Optimal design intensity as a function of failure cost.

### 3.3.2. *Application to design against earthquakes*

The method is applied to design of a 3×5 bay, 9-story special moment resisting frame steel office building in downtown Los Angeles. The building is designed for a wide range of base shear and meeting the drift and other requirements of NHERP 97. The system strength is measured by a system yield force coefficient (system yield force determined from a static pushover analysis using DRAIN2D-X divided by the system weight). Five limit states in terms of story-drift are used according to the performance levels of FEMA 273 [9]. The empirical seismic hazard procedure of FEMA 273 is used to calculate the ground excitation demand for a given probability level. To obtain the drift ratio from the spectral acceleration, the method based on uniform hazard response spectra and an equivalent nonlinear single degree of freedom system (SDOF) given in Collins et al. [3] is used. The drift ratio is then multiplied by correction factors to incorporate building capacity uncertainty and converted to damage factor according to FEMA-227. The maintenance cost is not considered in this study. Initial costs are estimated according to Building Construction Cost Data [2]. The nonstructural items were not considered since they are not functions of the design intensity. The damage cost, loss of contents, relocation cost, economic loss (dollar/sqft), cost of injury ($1000/person for minor and $10,000/person for serious injury) and cost of the human fatality ($1,740,000/person) are estimated based on FEMA reports [8]. All costs are given in 1992 US dollars. A constant annual discount rate $\lambda$ of 0.05 is assumed.

The initial cost, expected failure cost with (w/) and without (w/o) considering death and injury, and the total expected lifecycle cost, as functions of system yield force coefficient are shown in Fig. 5. The optimal system yield force coefficients are 0.194 and 0.189 with and without considering human injury and death costs respectively, both higher than the system yield coefficient of 0.14 according to 1997 NHERP [9]. The design is then extended to Seattle, Washington and Charleston, South Carolina with proper adjustment of the cost due to regional variation. The results are shown in Fig. 6 in which the current designs are shown for comparison. It is seen that the lifecycle cost (LCC) based designs are generally higher. The difference is large at Los Angeles,

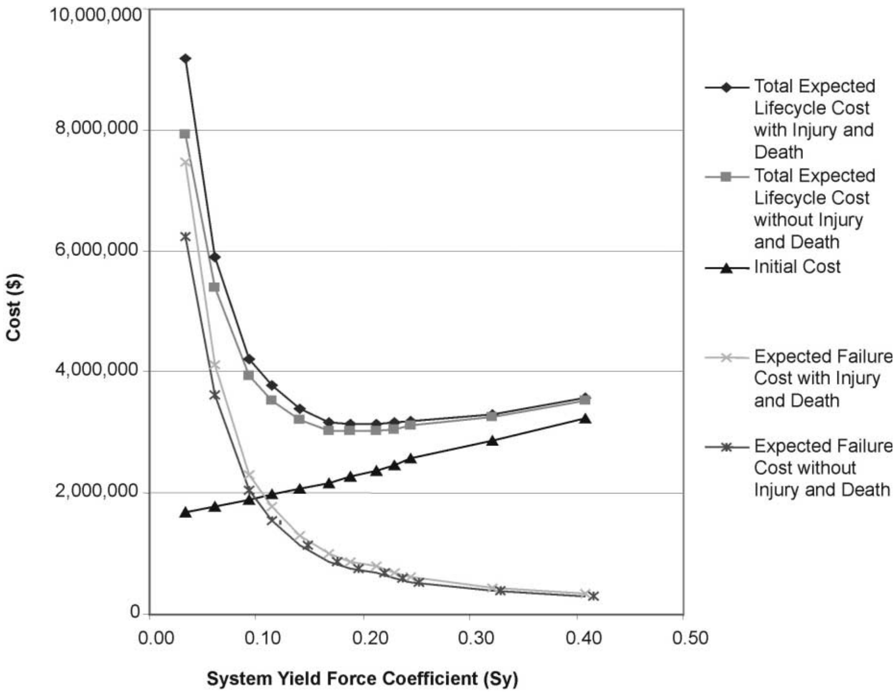



Fig. 5. Expected lifecycle cost as a function of system yield coefficient at Los Angeles.

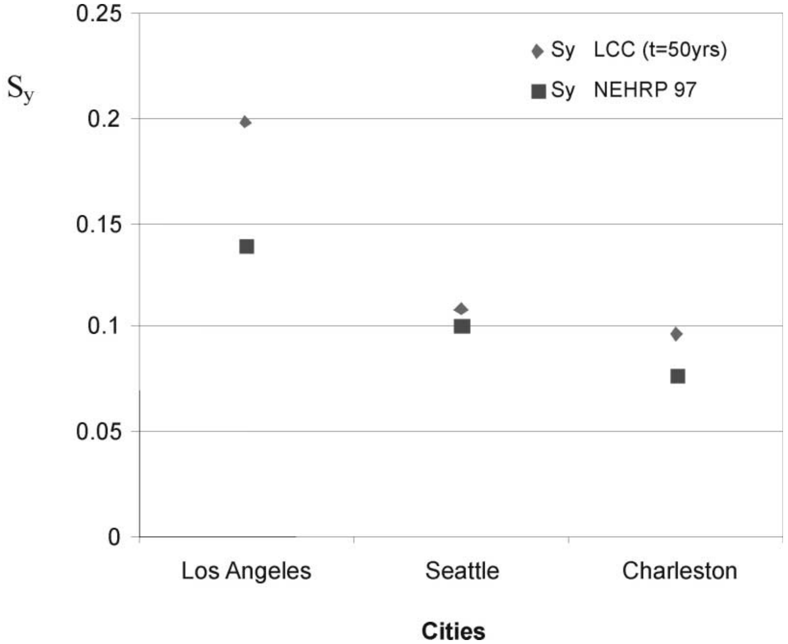



Fig. 6. Comparison of lifecycle cost (LCC) based and current [9] designs.

moderate at Charleston, and small at Seattle. Since some of the important decision parameters such as structural life span, discount rate, injury and death costs, and system capacity uncertainty are difficult to estimate accurately, a sensitivity of the optimal design was also carried out. The results (see Fig. 8, EQ) show that the optimal design intensity depends moderately on discount rate, and increases fast with structural life for $t<20$ years but slowly for $t>50$ years. It is insensitive to cost estimate of injury and death at Los Angeles but quite sensitive at Charleston due to the proportionally much larger contribution of these costs to the overall cost at Charleston. The seismic hazard curve at Charleston has a rather flat tail due to larger uncertainty in the high intensity range. As a result, the ratio of expected cost of death and injury to that of damage and economic losses is much higher at Charleston than at Los Angeles. Although not shown in Fig. 8, the results are found insensitive to the structural capacity uncertainty due to the dominance of the hazard uncertainty.

### 3.4. Optimal design under multiple hazards

#### 3.4.1. Parametric study

Two time-varying loads $S_1(t)$ and $S_2(t)$ due to natural hazards are treated as random processes. The simple Poisson process is used for occurrence for both loads and the intensities given the occurrence are modeled by exponential random variables. The load effect is assumed to be a linear combination of load intensity. A single limit state is considered. The capacity of the system against the limit state is assumed to be deterministic and given by a linear combination of the design load intensities. The cost of maintenance is not considered. The initial cost function is assumed to follow a simple power law of the design load intensities. The case of capacity controlled by one design load is also considered. The optimal solution is determined by searching for the minimum point of the closed form solution of the expected lifecycle cost given by Eq. (4). The system parameters are chosen such that $S_1(t)$ occurs much more frequently and has longer duration, whereas $S_2(t)$ is more intense and variable with a mean and a standard deviation twice those of $S_1(t)$. The power of the cost function are chosen to be 1.2 and 1.5 respectively such that the initial cost will increase slightly faster than a linear function with the design load intensity, and more so for $S_2(t)$. In other words, design for $S_2(t)$ is more expensive. The discount rate is assumed to be 5% per year. Because of the extremely small probability of coincidence the two loads, the contribution of the simultaneous occurrence of the two loads is negligible. The optimal design intensities for both loads as function of the structural life are obtained for three different values of cost of failure (limit state). Values of $C$ from 20 to 50 represents the case of considering only cost of replacement of the structure, whereas values from 100 to 500 represents the case of considering also costs of loss of revenue, injuries, and deaths. As in the single hazard case, the optimal design loads increase with the structural life but the increase is small for structural life longer than 50 years and highly dependent on the failure consequences (cost). The resultant annual limit state probabilities of the optimal design under each load as function of the structural life are calculated and shown in Fig. 7. Because of the dominance of $S_2(t)$, the overall (target) limit state probabilities are almost the same as those under $S_2(t)$ only (shown in dashed lines) and those under $S_1(t)$ (shown

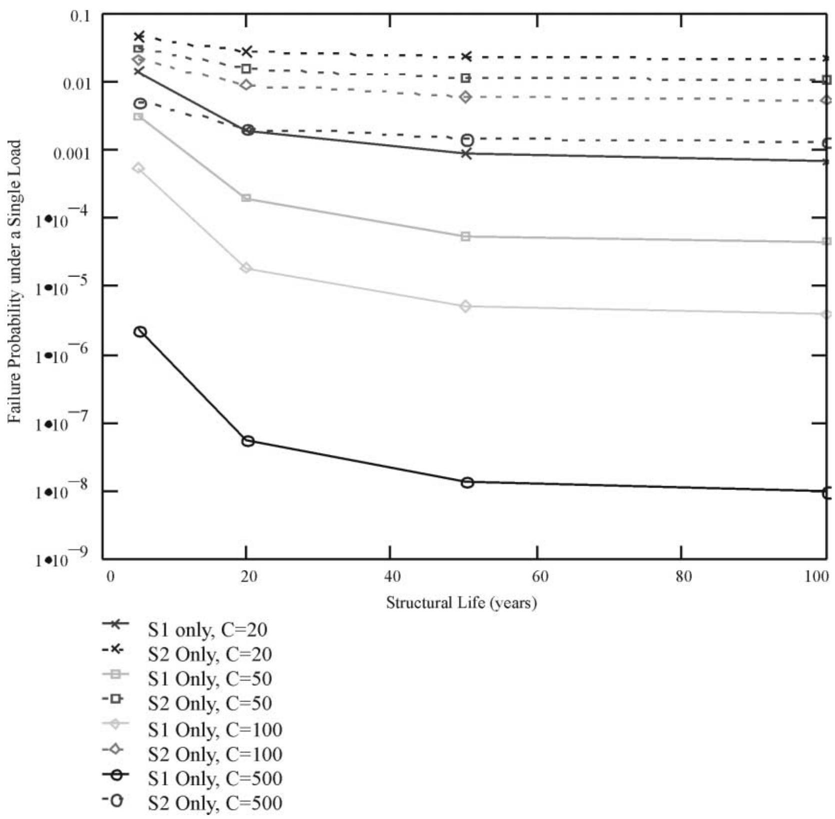



Fig. 7. Annual limit state probability of optimal design under one load only as a function of structural life [$C$ = cost of failure (limit state)].

in solid lines) are lower by at least one order of magnitude. The optimal reliability against each hazard also increases considerably as the failure cost increases from $C = 20$ when only structure damage is considered to $C = 500$ when revenue loss, injury and death are also considered. It is seen that uniform reliability against both loads is not necessary and would not be cost-effective.

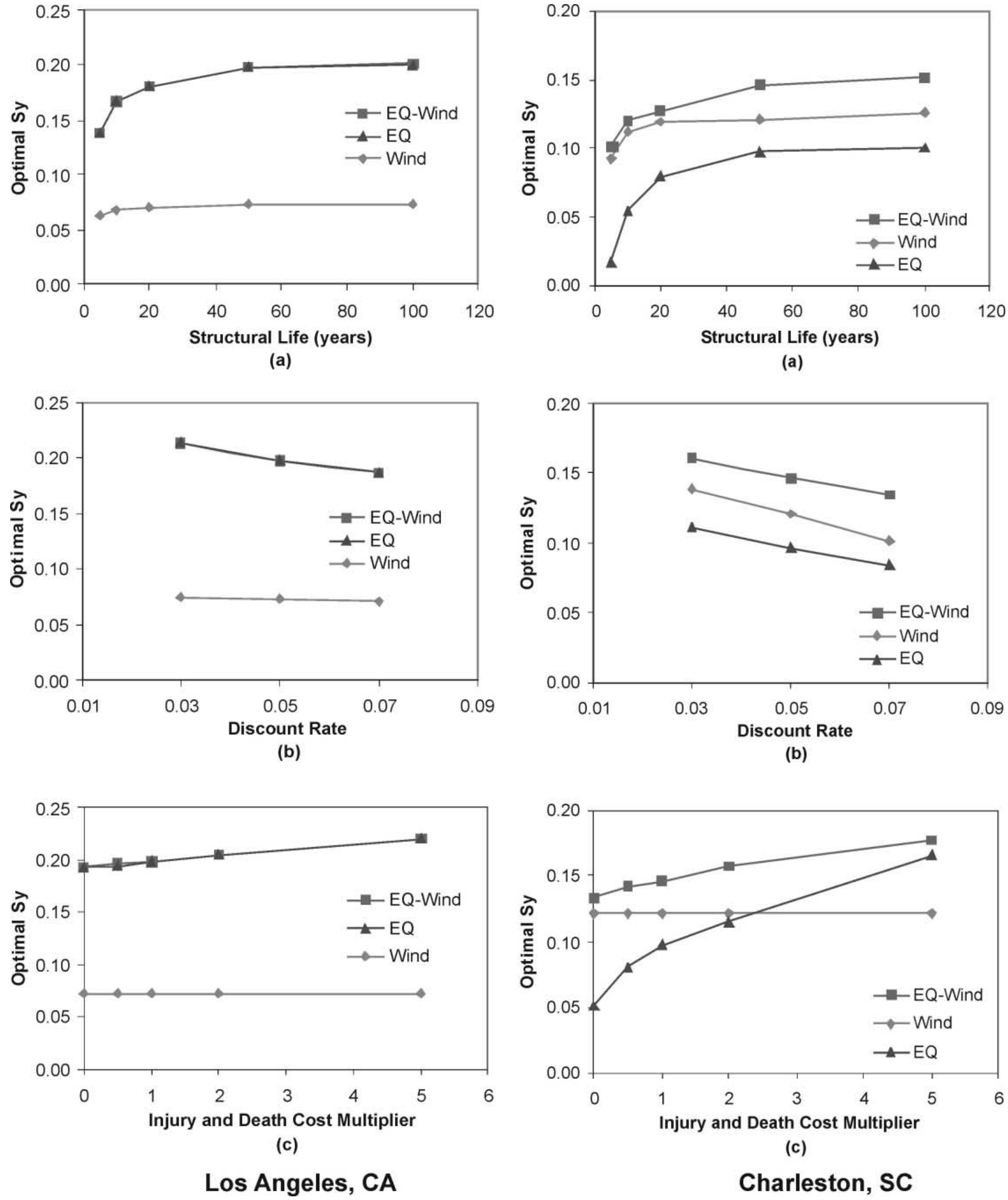



Fig. 8. Sensitivity of optimal design to structural life, discount rate, and injury/death cost multiplier.

### *3.4.2. Application to design against earthquake and wind hazards*

The application of the method is then extended to design in the three cities under both wind and earthquake hazards. It is of interest of compare the contribution of different hazards in the life-cycle cost design since at Angeles and Seattle seismic loads dominate whereas at Charleston wind loads play a very important role. The wind hazard and structural response analyses are based on provisions in ASCE-7 [1]. In addition, the structural envelope limit states are also considered since damage of structural envelope can lead to damage and loss of the building content, which has been shown to be most costly. As in the previous case, the structural envelope represented by glass windows was designed for a wide range of wind intensity. The window glass panel limit states are treated separately based on latest analytical and empirical studies [20] in which both wind pressure and wind-borne missiles are considered. It was found that the optimal design for window glass is dominated by the wind-borne missile effect. The optimal glass thickness was found to be 0.8 inches for Charleston and 0.53 inches for Los Angeles.

The optimal design of structural strength under both hazards is again expressed in terms of the system yield force coefficient $S_y$. Fig. 8 shows the sensitivity of the optimal $S_y$ under a single hazard and under both hazards to the structural design life, discount rate, and the multiplier applied to the cost of death and injury. At Los Angeles, the design is dominated by seismic load. The wind load contribution is so small that there is practically no difference between the design for both winds and earthquakes and that for earthquakes only. The design is sensitive to structural life for $t < 20$ years and becomes almost constant for $t > 50$ years. It is moderately dependent on the discount rate. The insensitivity of the design to the cost multiplier is due to the dominance of expected lifecycle cost of damage and economic losses since the probability of death and injury is extremely small by comparison. Considering the difficulty of assessing these factors, the optimal design coefficient is reasonably estimated to be 0.20. At Charleston, the wind hazard becomes more important. Seismic loads, however, still contribute. In other words, wind load does not "control", as would be the case in traditional design procedure. For example, for a design life of 50 years, the optimal $S_y$ increases from 0.121 under winds only to 0.146 when seismic load is also considered. The optimal design is more sensitive to the multiplier due to the larger uncertainty associated with the large seismic events causing larger contribution to the total expected lifecycle cost. Considering again all these factors, the optimal design $S_y$ may reasonably estimated to be 0.15. If these two optimal design strengths are used the implied reliability of the optimal design against individual hazards at Los Angeles and Charleston are shown in Fig. 9. It is seen that in both locations, the reliability against winds is much higher than that against earthquakes except at very low drift levels ($<0.2\%$, not shown in the figure). The finding from the parametric study that uniform reliability against different hazards is not required in an optimal design is further confirmed. The optimal target reliabilities are higher at Los Angeles than at Charleston primarily due to the much higher seismic hazard at Los Angeles.

## 4. Reliability and redundancy

Redundancy of structural systems has attracted much attention of engineers after the large number of failures of structural systems in recent earthquakes. Yet the definition and interpretation of structural redundancy vary greatly among individuals and disciplines and can lead to

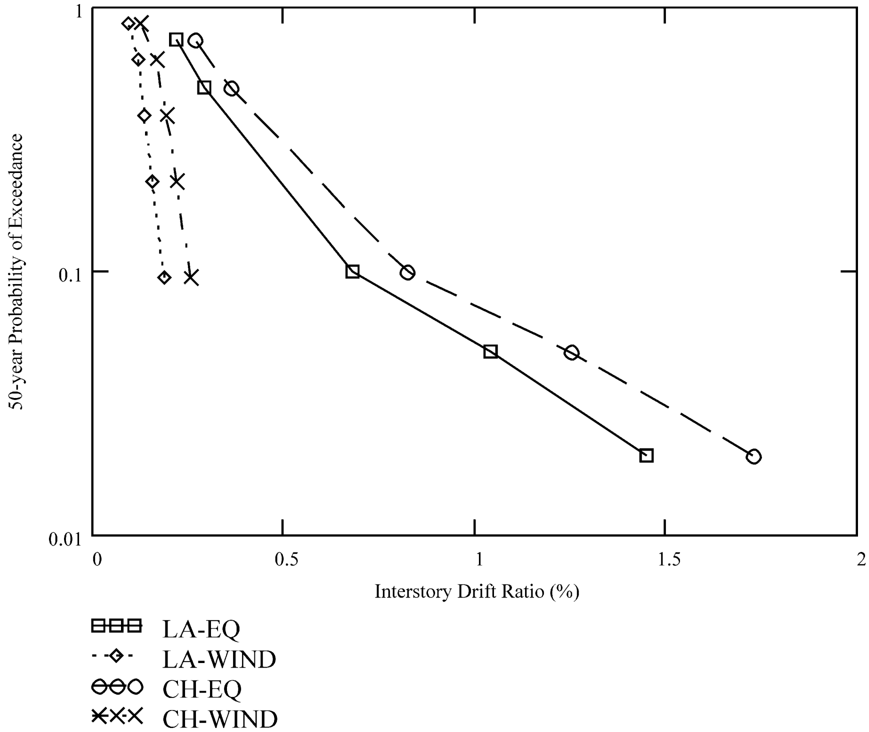



Fig. 9. Reliability of optimal design against individual hazard (winds or earthquakes) at Los Angeles, CA and Charleston, SC.

misunderstanding. Most define redundancy according to the structural configuration. For example, in the recent Uniform Building Code [15], there is a new reliability/redundancy factor $\rho$, as a function of the floor area and maximum element-story shear ratio. The allowable range of $\rho$ for the seismic lateral force can vary by as much as 50%. There has been much criticism by engineers on the rationale behind this new provision. In view of the large uncertainty of the seismic excitation and structural resistance, the redundancy of a structural system under seismic load cannot be treated satisfactorily without a careful consideration of the uncertainty. For example, a simple deterministic system of a given strength and identical parallel members under tension force will fail when the strength is exceeded regardless of the number of the parallel members since all members will reach the collapse threshold at the same time. Therefore, there is no advantage of having more members. The situation is drastically different if there is uncertainty in both loading and member strength as has been shown by Gollwitzer and Rackwitz [13]. Under random static loads, the parallel systems have significant redundancy (much higher reliability) if there is adequate number of members, moderate degree of ductility, low strength correlation among members, and small load variability compared with that of the member resistance. Similar conclusions have reached by Wen et al. [34] for simple parallel systems under stochastic time varying loads. It is clear that all these factors have not been considered in redundancy study of structures thus far, especial under random dynamic loads. The same is true in the case of the largely empirical redundancy factors proposed in code procedures.

### 4.1. Redundancy of structural systems under seismic loads

The redundancy of steel moment frames and dual systems consisting of moment frames and shear walls are studied. The nonlinear behavior of the members and connections including inelastic and brittle fracture failures are accurately modeled according to test results. The ground motions corresponding to three different probability levels (50, 10, and 2% in 50 years) developed in the SAC Steel Project [23] are used as excitation. To realistically model the load redistribution and effect of possible un-symmetric failure of members, 3-D response analysis methods were developed which allow evaluation of the bi-axial interaction and torsional response and member resistance is modeled by random variables. The study and findings are briefly described in the following. Details can be found in Wang and Wen [27,28] and Song and Wen [24].

### 4.2. Steel moment frames of different connection ductility capacity

Two low-rise steel moment frame buildings in the LA area, of 2 stories and 2 by 3 bays, and 3 stories and 3 by 5 bays, were designed according to current code procedures. The diaphragms are assumed to be rigid in its own plane but flexible out of plane. Plastic hinges can form at the ends of beams and columns. Fracture failures of connections occur when the capacity as a function of ductility and cumulative energy dissipation is exceeded. The capacity is modeled as random variable based on test results. The smooth hysteresis model [29] was extended and used to describe the post-yielding ductile and brittle behavior of the members and connections and calibrated against test results. It reproduces the nonlinear behavior well (Fig. 10). A 3-D response analysis method is then developed based on these element models. Response statistics under the SAC ground motions were obtained. Table 1 shows the 50-year probabilities of maximum column drift ratio of the 3-story steel building exceeding incipient collapse capacity under various assumptions of the ground excitation and structural response behavior. The incipient collapse

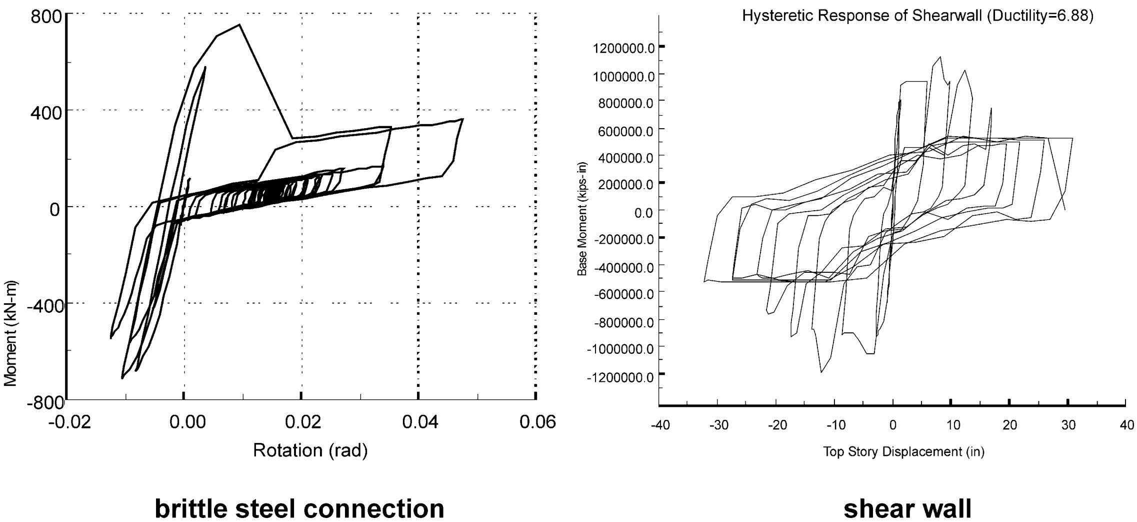



Fig. 10. Analytical models for components of moment frames and dual systems.

Table 1
50-Year incipient collapse probability and uniform risk redundancy factor for 3-story steel frame building

| Assumption of excitation, connection behavior, and accidental torsion | Drift ratio capacity (%) | $P_f$ | $R_R$ | $R$ |
|---|---|---|---|---|
| Uni-directional, ductile, no torsion | 0.08 | 0.004 | 1 | 8 |
| Uni-directional, brittle, no torsion | 0.05 | 0.021 | 0.986 | 7.88 |
| Bi-directional, ductile, no torsion | 0.08 | 0.037 | 0.828 | 6.63 |
| Bi-directional, brittle, no torsion | 0.05 | 0.124 | 0.546 | 4.37 |
| Bi-directional, ductile, torsion | 0.08 | 0.034 | 0.850 | 6.80 |
| Bi-directional, brittle, torsion | 0.05 | 0.129 | 0.538 | 4.31 |
| Bi-directional, brittle, column damage, and torsion | 0.05 | 0.182 | 0.471 | 3.77 |

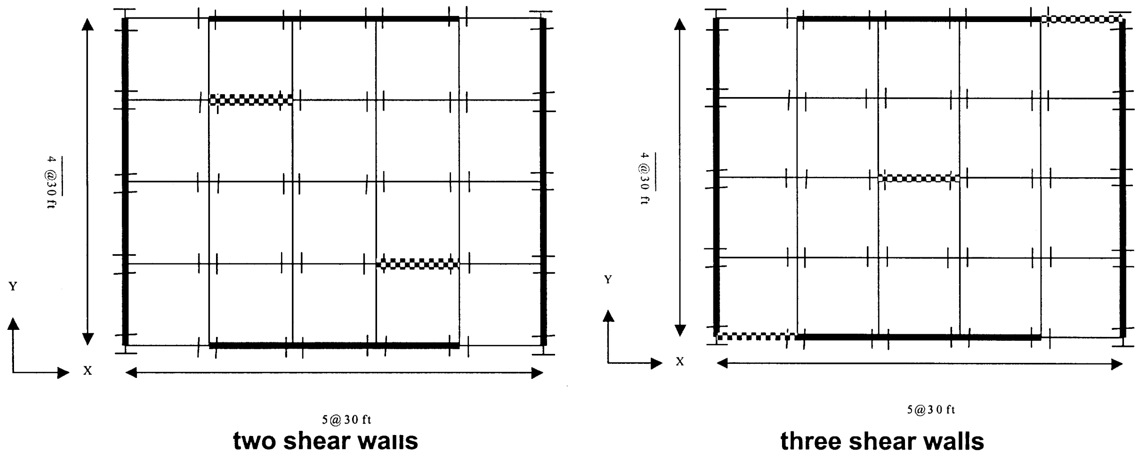
thresholds are assumed to be a maximum column drift ratio of 8% for ductile systems and 5% for systems with possible brittle connection failures, based on interim SAC research results by Yun (1998). It can be seen that the brittle fracture failure of the connections have only moderate effects compared with ductile systems under uni-axial excitation. The coupling of such failures with bi-axial interaction and torsional response due to possible un-symmetric member failures however, significantly increases the displacement demand on the structures (inter-story and global drift) and the probability of incipient collapse.

### 4.3. Uniform-risk redundancy factor $R_R$

It is obvious from the above that redundancy can be accurately defined only in terms of system reliability, therefore, it can be incorporated into design only through a reliability-based design procedure. To achieve uniform reliability in the current design procedure, for systems with different degrees of redundancy, a uniform-risk redundancy factor, $R_R$, may be used in conjunction with the widely used response modification factor, $R$, to determine the required design force. $R_R$ is defined as the ratio of the system spectral acceleration capacity corresponding to the actual 50-year incipient collapse probability $P_f$, to that required to achieve the allowable (target) $P_f$. For example, one can set the target $P_f$ to be 2% in 50 years and calculate these two spectral acceleration values based on the foregoing reliability analysis. Details can be found in Wang and Wen [27,28]. For a system with inadequate reliability/redundancy, $R_R$ will be smaller than unity. When the system has adequate reliability/redundancy; in other words, $P_f$ is lower than the allowable value, $R_R$ is equal to unity. $R_R$, therefore, functions as an adjustment factor to assure that the target (allowable) reliability will be achieved. The design seismic force is reduced by a factor of $R$ multiplied by $R_R$. Table 1 shows $R$ and $R_R$ for the 3-story building under various assumptions of component ductility capacity and excitation characteristics. For example, if an allowable value of $P_f$ is 2% in 50 years, $R_R$ varies from 1 under the assumption of uni-axial excitation and ductile connections, to 0.471 under bi-axial excitation with possible brittle connection failure. The results indicate that for a structure of a given configuration, ductility capacity and structural 3-D response (bi-axial and torsional motions) can greatly change the structural reliability/redundancy and lead to a large increase in the required design force which have not been considered in current code procedure.

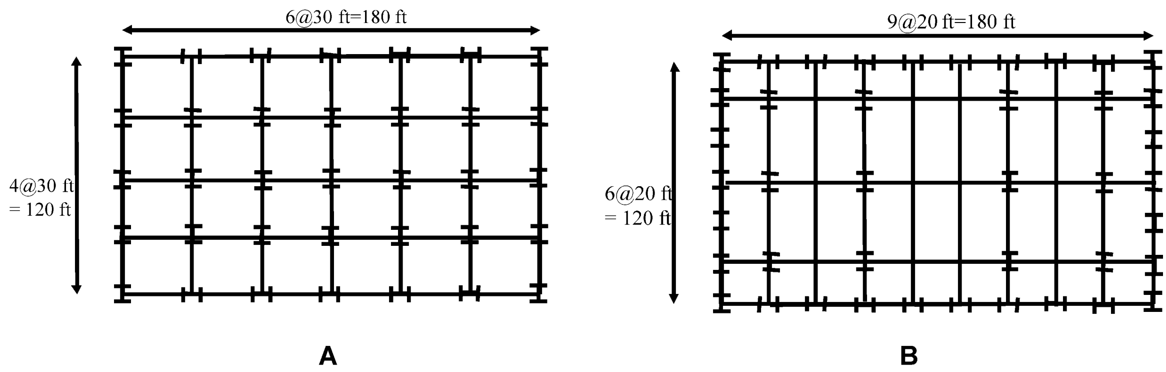
### *4.4. Dual systems and moment frames of different configurations*

The investigation has been extended to dual systems of moment frames and shear walls [24]. Factors considered include the number and layout configuration of the walls and the columns, the ductility of the walls, the relative stiffness of walls versus the moment frame, the uncertainty and the correlation of the strength of the walls. The interaction of the frame with the wall is important and highly dependent on the intensity of the ground excitation. An equivalent member model based on DRAIN-3DX has been developed for the shear walls. It reproduces well the nonlinear behavior of the walls (Fig. 10). For example, Fig. 11 shows the layouts of two 5-story, 4×5 bay dual systems of different numbers and configurations of shear walls. Fig. 12 shows two 3-story moment frames with different layouts. System A has 5 larger columns in the exterior lateral resistance frames compared with 7 smaller ones for System B. The dynamic properties (stiffness,




(1 ft = 0.3048 m)

Fig. 11. 5-Story dual systems of different configurations.




(1 ft = 0.3048 m)

Fig. 12. 3-Story moment frames of different configurations.

Table 2
Comparison of redundancy strength requirements based on uniform-risk factor method and UBC (97) $\rho$ factor method

| System | | $1/R_R$ | $\rho$ |
|---|---|---|---|
| 5-Story, one-way dual systems of different number of shear walls | 1 | 1.20 | 1.50 |
| | 2 | 1.17 | 1.02 |
| | 3 | 1.03 | 1.00 |
| 5-Story, one-way dual systems of different ductility capacity of shear walls | 5.4 | 1.14 | 1.00 |
| | 6.4 | 1.03 | 1.00 |
| | 7.4 | 1.00 | 1.00 |
| 3-Story SMRF systems of different numbers of bays, ductile connections, no torsional motion | 4×4 bay | 1.00 | 1.24 |
| | 6×6 bay | 1.00 | 1.00 |
| 3-Story SMRF systems of different numbers of bays, ductile connections, torsional motion | 1×1bay | 1.06 | 1.25 |
| | 2×2 bay | 1.00 | 1.25 |
| | 3×3 bay | 1.00 | 1.24 |
| 3-Story SMRF systems of different numbers of bays, brittle connections, torsional motion | 1×1bay | 1.58 | 1.25 |
| | 2×2 bay | 1.56 | 1.25 |
| | 3×3 bay | 1.45 | 1.24 |

ductility capacity) of these systems of different configurations are otherwise comparable. The limit state of concern is incipient collapse, i.e. 1.6–3% drift ratio of the shear walls depending on the aspect ratio and 5–8% column drift ratio [12] for the moment frames depending on brittle or ductile connection behavior. The target reliability is probability of exceedance less than 10% in 50 years for shear walls and 2% in 50 years for the system.

Response analyses under the bi-axial SAC ground motions were carried out and the required design forces according to the uniform-risk redundancy factor $R_R$ are shown in Table 2 and compared with the $\rho$ factor of UBC 97. The first column shows systems of different configurations, e.g. of different numbers of shear walls and ductility capacities for the dual systems and different numbers of bays of moment-resisting frames for the SMRF system. It is seen that for the 5-story dual system, the 3-shear wall system satisfies the requirement according to both methods. $\rho$ Factor requires a 50% increase in strength for the 1-shear wall system compared with only 20% according to the uniform-risk approach. On the other hand, the ductility capacity of the shear walls is not a factor in $\rho$ whereas the uniform-risk approach requires an increase of 14% when the ductility capacity is reduced from 7.4 to 5.4. For the 3-story ductile SMRF with no torsional motion, the number of bays (or size of columns and beams) causes no differences in structural response and both systems satisfy the target relaibility. The $\rho$ factor, however, requires a 24% increase for the system with smaller number of bays. On the other hand, when brittle connection failures and torsional motions are considered, the $\rho$ factor would overestimate the required strength for ductile system by about 25% and underestimate that for the brittle system by 20 to 30%.

## 5. Summary and conclusions

A framework is proposed for reliability and performance based design for natural hazards. The major factors in design are considered and properly treated in this framework including uncertainty in hazard demand and structural capacity, nonlinear structural response behavior, redundancy, balance of costs and benefits, and target reliability in design for a single or multiple hazards. Sensitivity studies are carried out to identity most important system and cost parameters. Examples are given on design of multi-story buildings against earthquakes and winds at Los Angeles, Seattle, and Charleston. The conclusions can be summarized as follows:

1. Currently available structural response and reliability analysis methods have the capability of treating the major factors in design and allowing development of risk-based, comprehensive, and yet practical design procedures familiar to engineers.
2. Structural design is highly dependent on consequence of structural limit states. Minimum expected lifecycle cost is a viable approach to setting reliability and performance goals. In design against multiple hazards, uniform reliability against different hazards is not required and hazards of large uncertainty and high consequence generally dominate.
3. Reliability/redundancy factor in current codes as a function of structural configuration only has been proved to be inadequate and yield inconsistent results. Redundancy factor needs to be considered in the framework of reliability-based design. The uniform-risk redundancy factor proposed is one possible such approach to ensure adequate reliability/redundancy.

The framework provides a basis that can be used to develop more rational codes and standards.

## Acknowledgements

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