ON THE SEISMIC SAFETY PROVIDED BY UNIFORM BUILDING CODE DESIGN METHOD

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SUMMARY

Based on random vibration methodology, the conditional probabilities of local ductilities have been computed for three different steel moment-resisting frames designed by the Uniform Building Code. The overall seismic safeties of these frames have been investigated by introducing the site-specific seismic risk. Using empirical relationships between structural/non-structural damage and local ductility ratio, the annual risks of damages due to future earthquakes have also been assessed for the three frames.

INTRODUCTION

The main objective of this paper is to quantify the degree of seismic protection provided by the Uniform Building Code (Ref. 1) design method. Specifically, the degree of seismic safety is computed for three steel moment-resisting frames, which represent real world low(4-story), middle-height(10-story), and high-rise(16-story) buildings (Ref. 2). The frames are assumed to have adequate bracing systems to resist out-of-plane motions. Elevations and member sizes of the three steel frames are shown in Fig. 1.

As reported by Lai (Ref. 3), uncertainties in the ground motion representation, the structural dynamic properties, and the method of dynamic analysis can be combined to evaluate local seismic response statistics conditional on peak ground acceleration. Based on this conditional reliability information, the overall seismic safety of the frames can then be assessed by introducing the site-specific seismic risk.

CONDITIONAL PROBABILITY OF LOCAL RESPONSE ON PEAK ACCELERATION

Through a series of sensitivity studies, Lai (Ref. 4) concluded that the local inelastic response of a frame would be relatively insensitive to the variations of Kanai-Tajimi damping (ζ_g) and structural damping (ζ_s). He also suggested that in predicting the conditional probability of building local response with given peak ground acceleration, the following five parameters need to be considered as random: i) strong-motion duration (S_o), ii) Kanai-Tajimi frequency (ω_g), iii) natural period ratio (R_T), iv) yielding strength factor (R_y), and v) local ductility correction factor (R_L). For details, the reader is referred to Lai (Ref. 4).

Using the modified multi-degree elasto-plastic random vibration methodology as developed by Lai (Ref. 4), local response characteristics of a structural frame can be computed for any particular combination of the aformentioned random variables. In this study, five values are used to discretize the probability distribution of each of the five random variables (Table 1). Using the enumeration method and the discrete probability mass functions for the random variables, the conditional probabilities of non-exceedance of the local ductilities (μ) have been computed for five different levels of peak accelera-

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tion (i.e., 0.1, 0.2, 0.3, 0.4, and 0.5g) for the three different steel frames. The results of the first-story local ductility ratios are presented in Figs. 2, 3, and 4, respectively.

As shown in the figures, under a 1/3g peak acceleration ground motion, the probability that the first story will remain elastic is about 79% for the 4-story UBC-designed steel frame. For the 10- and 16-story frames, the probabilities of remaining elastic are 81% and 88%, respectively.

OVERALL SEISMIC SAFETY OF UBC-DESIGNED FRAMES

To quantify the overall seismic safety of a building at a specific site, it is necessary to incorporate information about the local seismic risk. In this paper, a hypothetical site in Boston has been assumed. The weighted seismic risk curve is plotted in Fig. 5, which was originally computed by Tong et al. (Ref. 5).

By combining the local seismic risk information with the results of conditional reliability as presented in the preceding section, the overall seismic safety can be assessed for the three UBC-designed steel frames. The annual probability that the local inelastic response will exceed a given threshold of local ductility factor, μ^*, can be determined as follows:

$$P[\mu > \mu^*] = \sum_{C \cap a^i_{max}} P[\mu > \mu^* | C \cap a^i_{max}] \cdot P[C | a^i_{max}] \cdot P[a^i_{max}]$$
$$\simeq \sum_{a^i_{max}} \left(\sum_{C | a^i_{max}} P[\mu > \mu^* | C \cap a^i_{max}] \cdot P[C | a^i_{max}] \right) \cdot P[a^i_{max}] \qquad (1)$$

where C is the combination of all the pertinent random parameters, and $P[C| a^i_{max}]$ is expressed as follows:

$$P[C| a^i_{max}] = P[\zeta_g] \cdot P[\omega_g] \cdot P[R_T] \cdot P[\zeta_s] \cdot P[R_y] \cdot P[R_L] \cdot P[S_o | a^i_{max}] \qquad (2)$$

Notice that the strong-motion duration S_o is assumed to be conditional correlated with the peak ground acceleration a_{max} (Ref. 6).

In Eq. 1, $P[a^i_{max}]$ can be computed directly from the local seismic risk curve as follows:

$$P[a^i_{max}] = P[a_{max} \geq (a^i_{max} - \Delta a_{max})] - P[a_{max} \geq (a^i_{max} + \Delta a_{max})] \qquad (3)$$

where Δa_{max} is the discretization interval of the annual exceedance probability of peak ground acceleration for the site considered.

Based on Eq. 1 and 2, and using the weighted seismic risk curve for the example site, the overall seismic safeties have been evaluated for the three steel frames. the annual risks of exceeding various levels of the local ductility ratio are plotted in Figs. 6, 7, and 8, respectively. As shown in the figures, the annual exceedance probabilities of local ductility are asymptotic to the weighted seismic risk curve at high levels of peak ground acceleration. They tend to level off when the peak accelerations are relatively small. For the 4-story UBC-designed steel frame, the annual probability of having at least some yielding (i.e., $\mu > 1$) in the first story is estimated at 6.8×10^{-5}. The annual probability of having first-story local ductility exceeding 5 is approximately 5.91×10^{-7}. For the 10- and 16-story frames, the probabilities of having at least some yielding in the first story are estimated at 6.7×10^{-5} and 3.72×10^{-5}, respectively. The annual probabilities of having local ductilities exceeding 5 are approximately 1.14×10^{-6} and 5.91×10^{-7}, respectively.

For each of the three moment-resisting steel frame under study, the annual probability for the first-story local inelastic response exceedance is generally larger than those of the other stories. Since extreme severe local yielding at the bottom story implies total failure of the entire building, it is conceivable that the system failure probability of the building might be approximated by the annual exceedance probability of the first-story local response. Therefore, the first-story results presented herein are used to characterize the overall seismic system reliability for each of the three steel moment-resisting frames designed by the Uniform Building Code.

DAMAGE ASSESSMENT OF UBC-DESIGNED FRAMES

Few attempts have been reported in the literature to assess quantitatively the building damage due to earthquake ground motions. In developing the so-called "spectral matrix method" for predicting building damage, Blume and Monroe (Ref. 7) suggested a "damage factor" for relating the ductility ratio to the structural (or non-structural) damage. The damage factor (DF) has been modified by Blume et al. as follows (Ref. 8 and 9):

$$\text{Damage Factor (DF)} = \left(\frac{\mu - 1}{\mu_{ult} - 1} \right)^{\kappa} = \frac{\text{Building Repair Cost}}{\text{Building Replacement Cost}} \qquad (4)$$

where μ_{ult} is the ultimate ductility of the building (i.e., the point at which the structural deformation increases with decreasing shear force). The quantity κ is an empirical economic factor which varies with different types of buildings. Using this empirical relationship between structural/non-structural damage and the local ductility ratio, it becomes possible to consider the balance between the building replacement cost (or discounted initial cost) and the expected damage due to possible earthquake attacks for a particular building frame at a specific site.

Consider first the 4-story UBC-designed steel frame for which an ultimate ductility ratio of 16 and an economic factor of 2.2 are assumed (Ref. 8). Based on Eq. 4, the damage factors corresponding to various levels of local ductility ratio can be conveniently computed and are listed in Table 2. For example, the local ductility ratio of 5 corresponds to a damage factor of 5.459×10^{-2}.

Hence, the annual exceedance probability of local ductility ratio can readily be interpreted as the annual probability of exceeding a given level of the damage factor. For the example 4-story UBC-designed steel frame, the annual probability that the damage factor will exceed 2.586×10^{-3} ($\mu > 2$) is estimated at 1.86×10^{-5}, and the probability is 5.91×10^{-7} for damage factor exceedance of 5.459×10^{-2}. The relationship between the annual exceedance probability of the damage factor and the damage factor itself is plotted in Fig. 9.

For the middle-height (10-story) and high-rise(16-story) buildings, Scholl (Ref. 9) suggested an ultimate ductility ratio of 10 and an economic factor of 3. Based on these values, the damage factors have been computed for different levels of local ductility ratios as listed in Table 2. The resulting annual probabilities of damage factor exceedances are also plotted in Fig. 9. As shown in the figure, the annual probability that the damage factor will exceed 8.8×10^{-2}(i.e., $\mu>5$) is 1.14×10^{-6} for the 10-story frame, and it is 5.91×10^{-7} for the 16-story steel frame. For damage factor exceedance of 1.372×10^{-3}, the annual risks are 1.08×10^{-5} and 5.28×10^{-6}, respectively for the 10- and 16-story steel frames designed by the Uniform Building Code.

CONCLUSIONS

Based on the investigation of the three UBC-designed steel moment-resisting frames which represent low(4-story), middle-height(10-story), and high-rise(16-story) buildings, the degrees of overall seismic safeties have been evaluated respectively. For illustrative purposes, a weighted seismic risk curve for a hypothetical site was employed. The results suggest that the annual exceedance probability for the local inelastic response generally decreases with increasing local ductility threshold, and it approaches asymptotically the site-specific seismic risk curve when the peak ground acceleration threshold becomes large.

By using empirical relationships between the structural/non-structural damage and the local ductility ratio, the annual risks of the damage factor (i.e., the ratio between the expected losses due to future earthquake attacks and the building replacement cost) have been computed for the three steel frames designed by the Uniform Building Code. The methodology reported herein can provide the basis for quantifying the effectiveness of alternative design procedures for buildings in reducing the risk of damage and failure under seismic loads.

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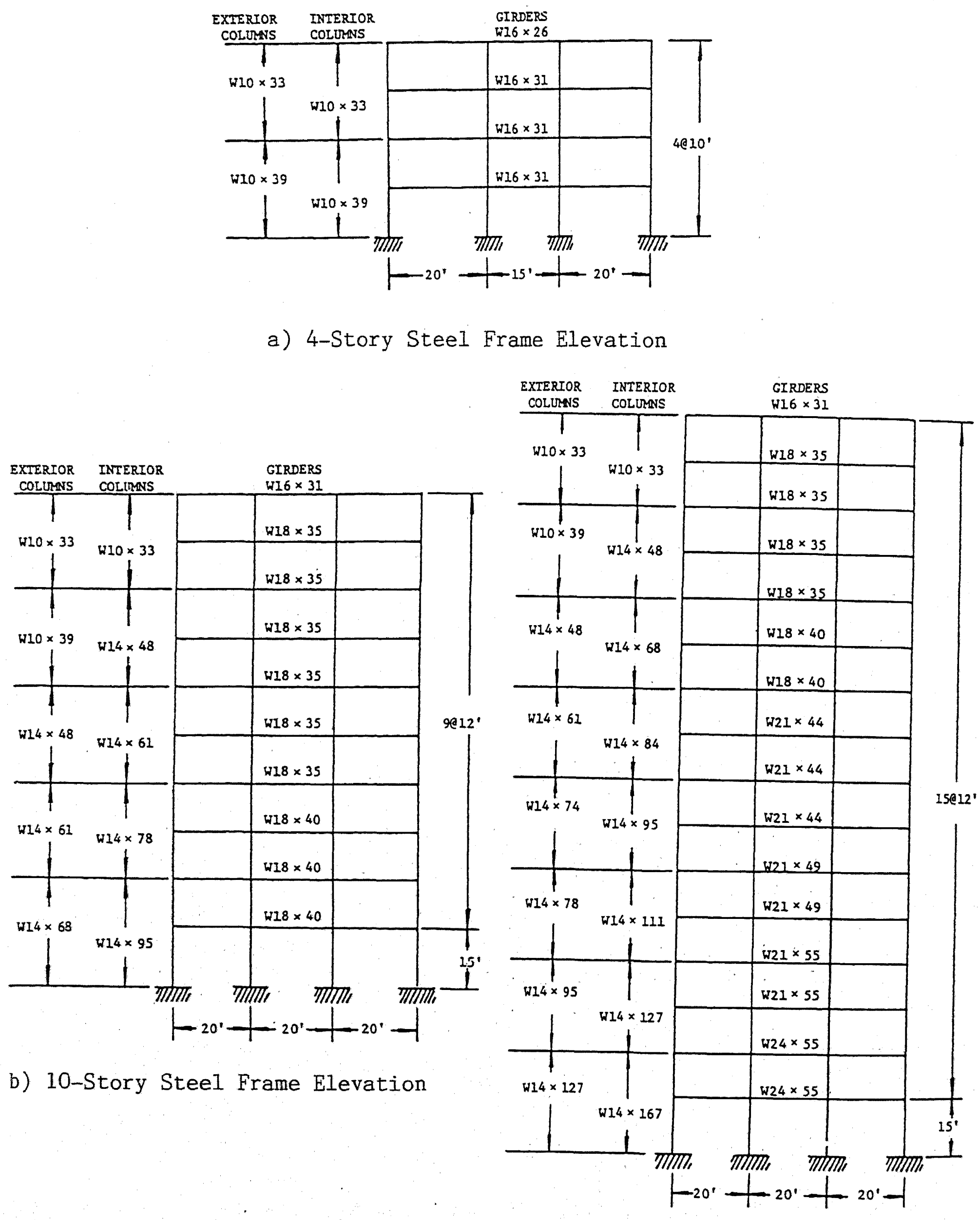
TABLE 1 Discrete Probability Mass Functions of Random Variables for Overall Seismic Safety Analysis

Variables	1	2	3	4	5
ζ_g	0.32	–	–	–	–
ω_g(rad/sec)	6.0	10.0	15.0	20.0	30.0
$P[\omega_g]$	0.0476	0.1474	0.2410	0.3069	0.2571
S_o(sec)	0.5	1.0	3.0	5.0	9.0
$P[S_o \mid a_{max}]$*	0.0917	0.2782	0.2998	0.2161	0.1142
R_T	0.7	0.9	1.1	1.3	1.6
$P[R_T]$	0.1379	0.2328	0.2434	0.2097	0.1762
ζ_s	0.0491	–	–	–	–
R_y	0.8	0.95	1.1	1.25	1.5
$P[R_y]$	0.1113	0.2333	0.2733	0.2507	0.1314
R_L	0.3	0.6	1.0	1.4	1.8
$P[R_L]$	0.1358	0.2738	0.2697	0.1670	0.1537

*The discretized $P[S_o \mid a_{max}]$ varies w.r.t. peak acceleration.

TABLE 2 Relationship Between Damage Factor and Local Ductility Ratio for Different Steel Moment-Resisting Frames

Frames	$\mu = 1$	$\mu = 2$	$\mu = 3$	$\mu = 4$	$\mu = 5$
4-Story	No Damage	2.586×10^{-3}	1.188×10^{-2}	2.899×10^{-2}	5.459×10^{-2}
10- & 16-Story	No Damage	1.372×10^{-3}	1.097×10^{-2}	3.704×10^{-2}	8.779×10^{-2}



a) 4-Story Steel Frame Elevation

b) 10-Story Steel Frame Elevation

c) 16-Story Steel Frame Elevation

FIG. 1 Elevations and Member Sizes of the 4-, 10-, and 16-Story Steel Frames Designed by Unifrom Building Code (Ref. 2)

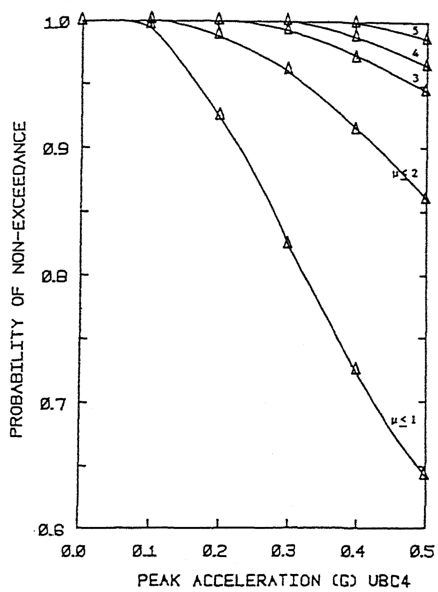


FIG. 2 Non-Exceedance Probability of 1st-Story Local Ductility Given Acceleration for 4-Story Frame

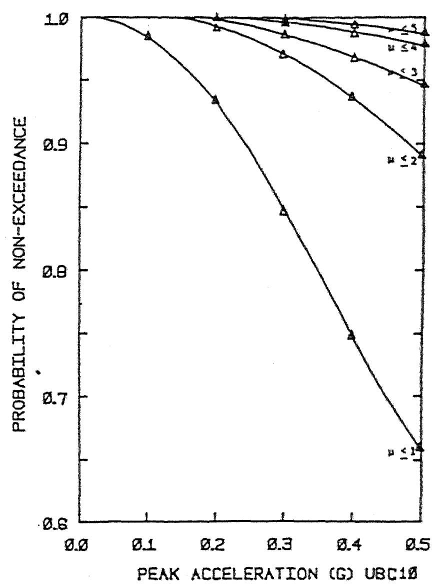


FIG. 3 Non-Exceedance Probability of 1st-Story Local Ductility Given Acceleration for 10-Story Frame

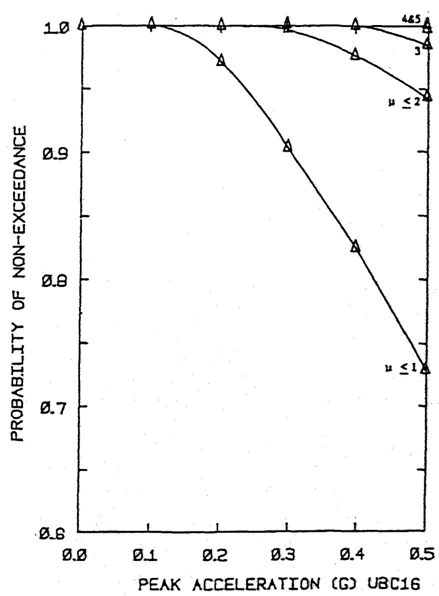


FIG. 4 Non-Exceedance Probability of 1st-Story Local Ductility Given Acceleration for 16-Story Frame

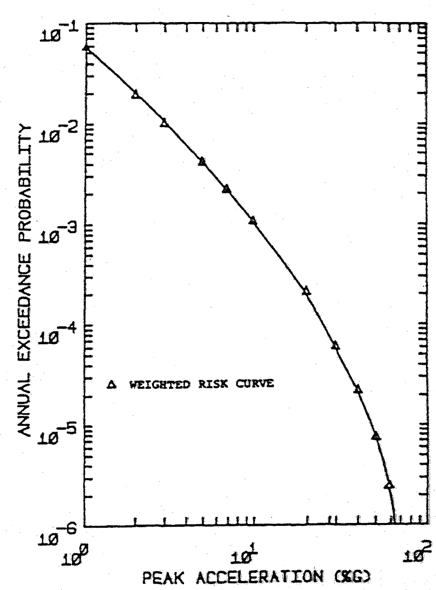


FIG. 5 Seismic Risk Curve for Hypothetical Boston Site (Ref. 5)

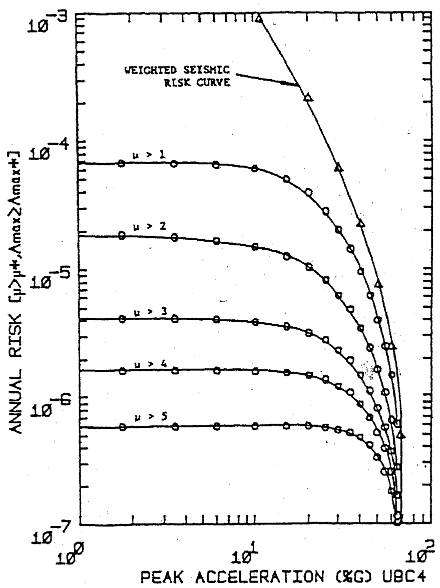


FIG. 6 Annual Risk of 1st-Story Local Ductility for 4-Story Frame

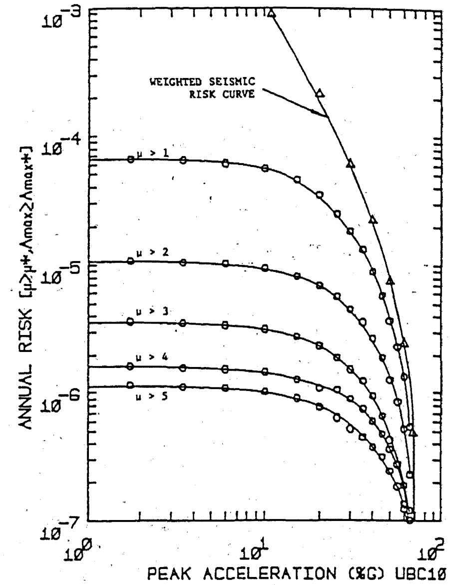


FIG. 7 Annual Risk of 1st-Story Local Ductility for 10-Story Frame

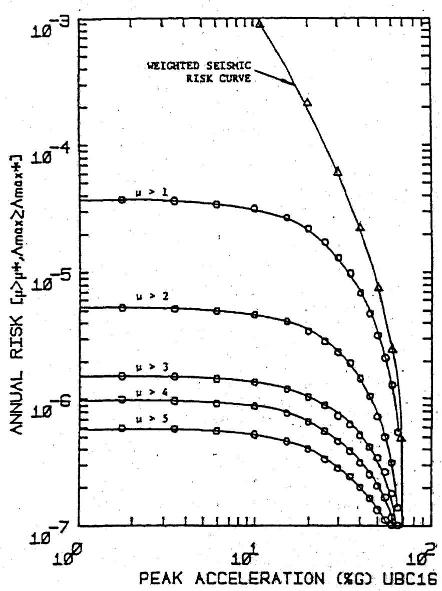


FIG. 8 Annual Risk of 1st-Story Local Ductility for 16-Story Frame

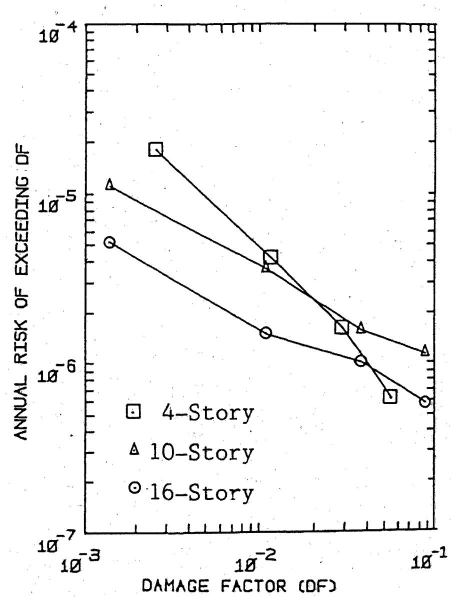


FIG. 9 Annual Risks of Damage Factors for Three UBC-Designed Frames