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RELIABILITY BASED SEISMIC DESIGN CRITERIA OF REINFORCED CONCRETE BUILDINGS IN JAPAN

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ABSTRACT

Because of large uncertainties associated with earthquake loads, the safety of structure is unknown and often justified by engineering judgment. Probabilistic methods can be used to quantify these uncertainties and to provide information for developing a reliability-based design code. The objective of this paper is to evaluate the seismic safety of reinforced concrete buildings designed according to a new Japanese design guideline which is based on an ultimate strength design concept. Reliability performance curves are then presented to describe the design criteria in terms of the probability of the design limit state being exceeded over a given period of time.

KEYWORDS

Reliability design; seismic design; design criteria; earthquake response; reinforced concrete building; response surface; probability method.

MODELING OF EARTHQUAKE GROUND MOTIONS

Site Characteristics

Three major cities in Japan, Sendai, Tokyo and Osaka, are selected as the sites for the buildings (see Fig.1.) The ground conditions at the sites are assumed to be stiff ground (Type 2 ground in the AIJ design code). The historical earthquake data available at the sites are used to construct the probability models of input ground motions. The seismicity data adopted here satisfy the following conditions:

1) The earthquakes are observed between 1885 and 1994 by JMA(Japan Meteological Agency.)
2) The epicentral distance, R, is limited below 350km.
3) The magnitude, M, is larger than 5.5 in the JMA magnitude.
4) The focal depth, D, is less than 100km.

As the results, the total numbers of earthquake data are 1127, 848 and 279 for the sites in Sendai, Tokyo and Osaka, respectively.

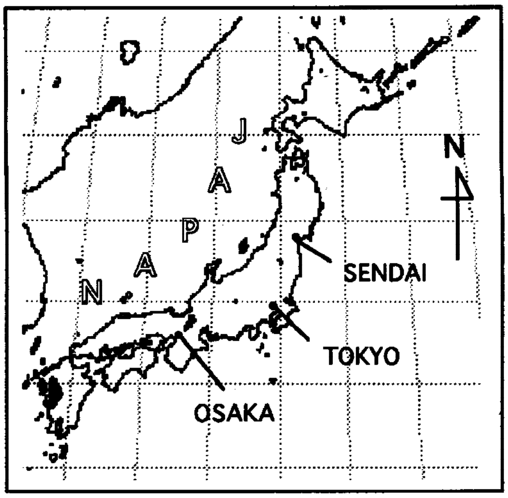


Fig.1. Location of sites

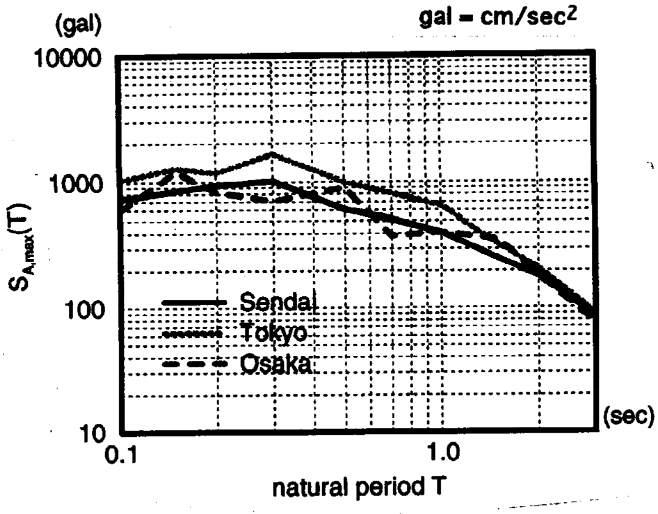


Fig.2. Hazard spectrum of p_f=50% in 100 years

Probability Model of Acceleration Response Spectrum

The following attenuation formula proposed by Kawashima et al.(1985) is used to obtain the earthquake acceleration response spectrum:

$$S_A(T,M,R,GC)=a\,(T,GC)\times 10^{b(T,GC)M}\times (R+30)^{c} \tag{1}$$

where $S_A(T,M,R,GC)$ is an earthquake response spectra, $a(T,GC)$, $b(T,GC)$ and c are the factors tabulated for several values of natural period T and ground condition GC. For each site, substituting the data of magnitude M and distance R into the above equation, the probability distribution function (p.d.f) of $S_A(T)$ is evaluated as a log-normal distribution.

Assuming the Poisson process for the occurrence model of earthquakes, the p.d.f of "maximum acceleration spectrum $S_{A,max}(T)$ in t years" is expressed by the following equation :

$$F_{S_{A,max}}(s_A)=\exp\left\{-\nu t\left(1-F_{S_A}(s_A)\right)\right\}=\exp\left\{-n\left\{1-F_{S_A}(s_A)\right\}\right\} \tag{2}$$

where, ν : the annual occurrence ratio of earthquakes, $n=\nu t$: number of earthquakes in t years,
$F_{SA}(s_A)$: the probability distribution of spectrum $S_A(T)$ (=log-normal distribution)

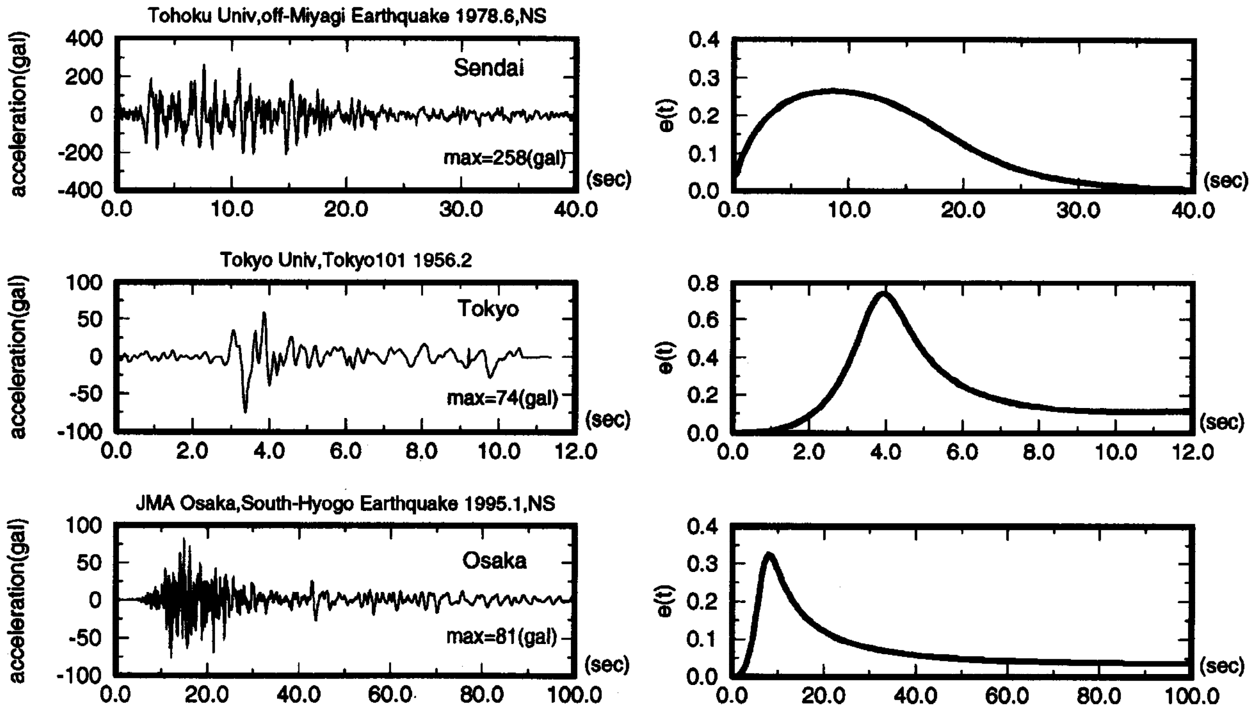
In case of large number n, this distribution asymptotically approaches to be the Type 2 extreme distribution (Ang and Tang, 1975.)

$$F_{S_{A,max}}(s_A)=\exp\left\{-\left(\frac{v_n}{s_A}\right)^{k}\right\},\quad v_n=\exp\left\{\xi_{SA}\sqrt{2\ln(n)}-\xi_{SA}\frac{\ln(\ln(n))+\ln(4\pi)}{2\sqrt{2\ln(n)}}+\lambda_{SA}\right\} \tag{3}$$

where, $\lambda_{SA}=E(\ln(S_A))$: the mean value of $\ln(S_A)$, $\xi_{SA}=\sqrt{Var(\ln(S_A))}$: the standard deviation of $\ln(S_A)$

Fig.2 shows the so-called hazard spectrum which presents the maximum acceleration response spectrum with an failure probability p_f (=1-$F_{SA,max}(s_A)$) of 50% in 100 years. It is seen that the coordinates of acceleration response spectrum at the Tokyo site are generally larger than those at other two sites.

In the following sections, sample values of the maximum response spectrum, $S_{A,max}(T)$, are generated from the probability distribution of Eq.(3), and they are used for the risk evaluation of buildings at the sites.



(a) Observed earthquake records (b) Normalized envelope functions

Fig.3. Envelope functions at the sites

Model of Envelope Function

We adopt the following attenuation formula proposed by Hisada and Ando(1976) to obtain the time duration t_D of input ground motions.

$$t_D = 10^{0.31M-0.774} \tag{4}$$

Substituting the data of magnitude M for each site, the p.d.f of t_D is modeled as a log-normal distribution. The mean value of t_D, considering the correlation to the spectral intensity S_A, is obtained from the following equation (Ang and Tang, 1975.) This study adopts the mean value, μ_{tD}, for the duration of input ground motions.

$$\mu_{tD}=E(t_D|S_A=s_A)=\exp\left\{\lambda_{tD}+\rho\frac{\xi_{tD}}{\xi_{SA}}\left(\ln(s_A)-\lambda_{SA}\right)\right\} \tag{5}$$

The following formula is versatile for expressing various shape of envelope functions:

$$e^2(t) = \frac{A\, t^B e^{-Ct}}{D+t^E} \tag{6}$$

The parameters A,B,C,D and E are estimated from actual ground acceleration records obtained at the sites using a nonlinear least square method (Saito and Wen, 1994.) The observed records and the normalized envelope functions for the sites are presented in Fig.3.

When the duration of earthquake changes from the original one (t_D) to a new one (t_{D1}), the new parameters A1,B1,C1,D1 and E1 for the above equation can be derived as follows so that the shape of the envelope function doesn't change at the new ground motion.

$$\eta=t_{D1}/t_D,\quad A_1=\eta^{-B+E-1},\quad B_1=B,\quad C_1=C/\eta,\quad D_1=\eta^E D,\quad E_1=E \tag{7}$$

Stochastic Model of Input Ground Motions

The following nonstationary stochastic process is used for the model of input ground motions:

$$a(t) = e(t)\, s(t) \qquad (8)$$

where e(t) is an envelope function (Eq.(6)) and s(t) is a stationary stochastic process having a certain power spectrum, $PS(\omega)$. The power spectrum $PS(\omega)$ is determined by iterative calculations to be compatible with the maximum acceleration response spectrum $S_{A,max}(T)$, which is obtained in the previous section.

EARTHQUAKE RESISTANT BUILDING DESIGN

Two different reinforced concrete buildings, 7-story and 12-story moment-resisting frames, are designed in accordance with a new Japanese design guideline, which was presented by the Japan PRESSS (PREcast Seismic Structural Systems) working group in 1993. The elevation views of the buildings are shown in Fig.4. The structural dimensions and the design of reinforcement steels are presented in Table 1 and Table 2 (steel size is described in accordance with the JIS standard.) The concrete is assumed to have a normal strength Fc=360kgf/cm2, and the reinforcement consists of Grade SD395 steel reinforcing bars.

The PRESSS guideline requires a nonlinear incremental analysis with a vertical distributed earthquake force to examine the performance of the building under both the serviceability limit state and the ultimate limit state. The design criterion on each design limit state are defined in terms of story drift ratios as follows:

1) At the serviceability limit state (with standard base shear coefficient $C_B = 0.2$), no member is allowed to yield, and the story drift ratio must be less than 1/200.
2) At a story drift ratio of 1/100, the story shear at any story must be more than 0.9 times the design shear for the ultimate limit state (with C_B=0.3). Furthermore, at a story drift ratio 1/50, the story shear must be more than the design shear.

The details of this design method are described in the reference (Saito and Wen, 1994.) The nonlinear earthquake responses of the buildings are calculated using the computer program Frame-D developed at the Tohoku University. The results are used for the seismic risk evaluation in the following sections.

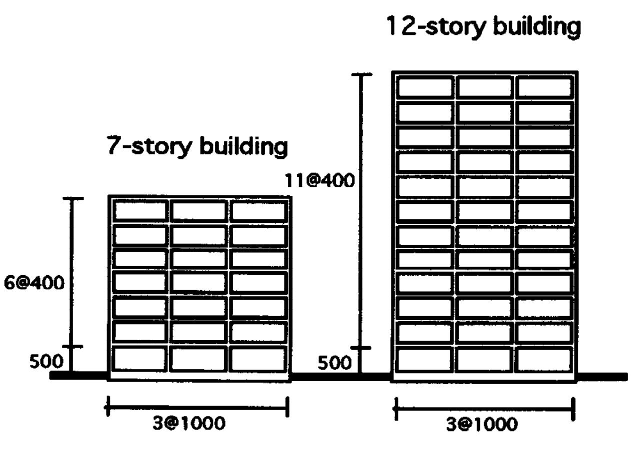


Fig. 4. RC model frames (unit: cm)

Table 1. Dimension of structural members (unit: cm)

7-story build.	1	2-7	Roof
column	85×85	85×85	
beam		40×90	40×90
12-story build.	1	2-12	Roof
column	90×90	90×90	
beam		55×90	55×90

Table 2. Reinforcement steels

7-story build.	1T	1B	2,3	4,5	6	7	R
column	12-D38	10-D38					
beam			6-D38	5-D35		4-D29	
12-story build.	1T	1B	2,3,4	5,6,7	8,9,10	11,12	R
column	16-D41	12-D41			12-D38		
beam			7-D41		5-D41	5-D32	

SEISMIC RISK ANALYSES

Probability Model of Maximum Displacement Response

The seismic risk of the building is evaluated in terms of the probability of the maximum displacement exceeding several thresholds during certain periods of time. Three different periods, t=50, 100 and 500 years, are selected in this study. Since we consider the uncertainty of input ground motions, the large amount of nonlinear response calculations are generally required to obtain the statistical data for the risk evaluation. The response surface method is one of the useful techniques to reduce the number of calculations.

Yang and Liu (1981) modeled the distribution of the maximum response to a nonstationary excitation as a Type 1 extreme distribution. Yao and Wen (1993) obtained the parameters of the distribution as second order polynomial functions of system parameters using the response surface method. Same procedures are adopted in this study, and the "conditional" probability density function of maximum displacement Y is expressed by a Type 1 extreme distribution as follows :

$$F_{Y|X}(y|x)=\exp[-\exp\{-\alpha(y-u)\}],\quad X=S_{A,max}(T) \tag{9}$$

Using the response surface method, the parameters α and u are assumed to be the third order polynomial functions of the random variable, $X=S_{A,max}(T)$.

$$\alpha=f_1(X),\quad u=f_2(X) \tag{10}$$

These functions are estimated by the least square method using the sample values of α and u calculated at seven sample points of X. The sample points of X are selected to be (-2.0, -1.5, -1.0, 0.0,.1.0, 1.5, 2.0) of U, where U is a standardized normal variable of X. Note that the nonlinear earthquake response analyses are only required at these sample points.

Evaluation of Failure Probability

After calculating conditional failure probability, the "unconditional" failure probability is calculated as,

$$p_f(y)=\int_x p_f(y|x)f_X(x)dx \tag{11}$$

where, $p_f(y|x) = 1-F_{Y|X}(y|x)$: the conditional failure probability of Y giving the value of $X=S_{A,max}(T)$.
$f_X(x)$: the probability function of X.

The simple way to calculate the above equation is to apply the Monte-Carlo Method, that is,

$$p_f(y)\simeq\frac{1}{N}\sum_{i=1}^{N} p_f(y|x_i) \tag{12}$$

where N : the number of sample sets Xi (i=1,2,...,N).

Since the parameters of the conditional failure probability $p_f(y|x)$ are already given as the functions of the random variable X (as in Eq.(10)), the failure probability $p_f(y)$ can be calculated just substituting the random samples of X into Eq. (12). It should be noted that there is no need to carry out earthquake response analyses to evaluate the failure probability.

Fig.5 presents the flowchart of main procedures of risk evaluation in this study.

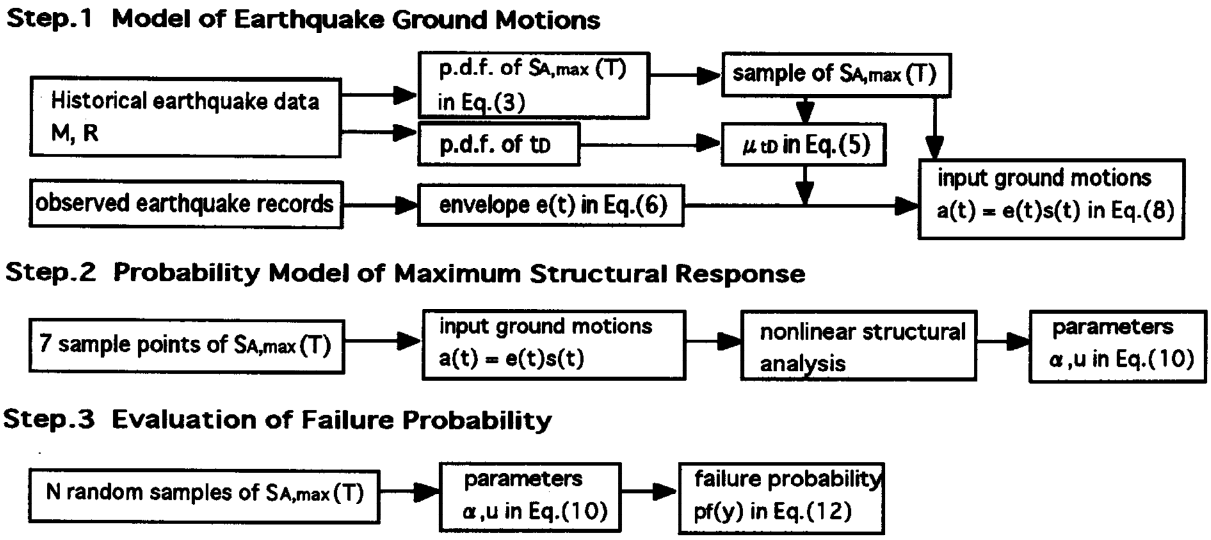


Fig.5. Flowchart for seismic risk evaluation

ANALYTICAL RESULTS

Reliability Performance Curves

Using failure probabilities evaluated for two different buildings (7-story and 12-story buildings) in three different sites (Sendai, Tokyo, and Osaka), the reliability performance curves are presented in Fig.6 for three different time periods (t = 50, 100, and 500 years.) The horizontal coordinate represents the maximum story drift ratio, and the vertical coordinate represents the probability of exceeding the story drift in t years.
The results are summarized as follows.

1) The failure probabilities of the buildings at Sendai site are generally smaller than those at other sites. It means that the seismic risk of buildings in Sendai site is much smaller than that at other sites.
2) For the 7-story buildings, the failure probabilities of the buildings in Tokyo site are generally larger than those at Osaka site. On the contrary, for the 12-story buildings, the failure probabilities at Osaka site are generally larger than those at Tokyo site.
3) When the period of time, T, changes, the shape of performance curves also change. It means that the nonlinear responses of buildings affect the reliability performance curves significantly.

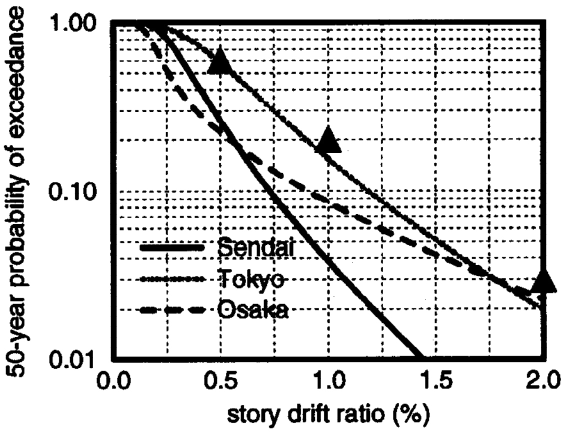
The triangular marks, ▲'s, in Fig.6 represent the following values of failure probability:

$$\begin{array}{lll} pf = 60\% & \text{at story drift ratio } 1/200 & \text{in } T=50 \text{ years} \\ pf = 20\% & \text{at story drift ratio } 1/100 & \text{in } T=50 \text{ years} \\ pf = 3\% & \text{at story drift ratio } 1/50 & \text{in } T=50 \text{ years} \end{array} \tag{13}$$

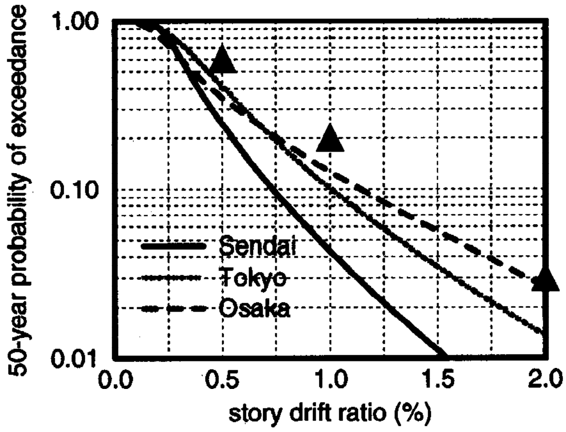
Corresponding failure probabilities for other time periods are obtained using the binomial distribution as follows :

$$p_{f,T=t} = 1.0 - (1.0 - p_{f,T=50})^N, \quad N = (t/50) \tag{14}$$

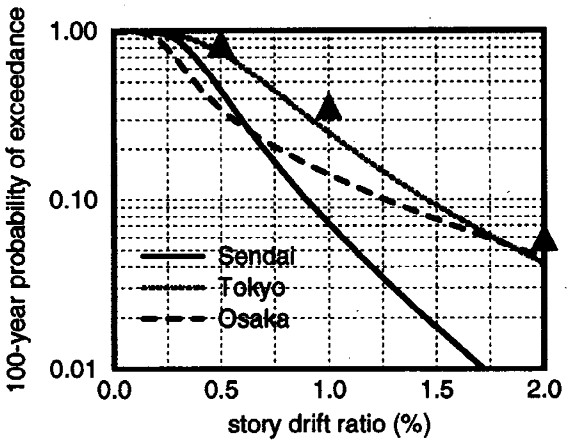
In all cases in Fig.6, the failure probabilities at triangular marks are regarded to be the upper bounds of the reliability performance curves.



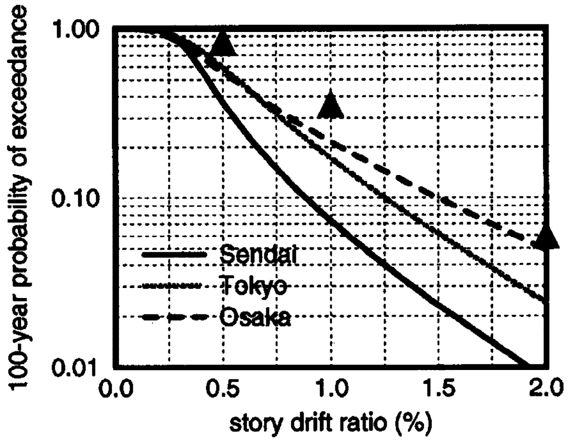
(a) 7-story RC buildings (t = 50 years)



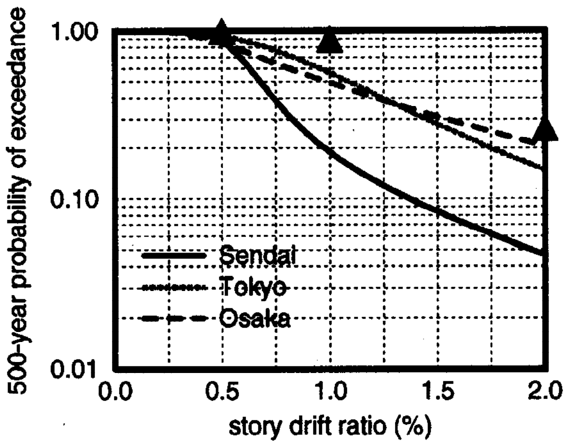
(b) 12-story RC buildings (t = 50 years)



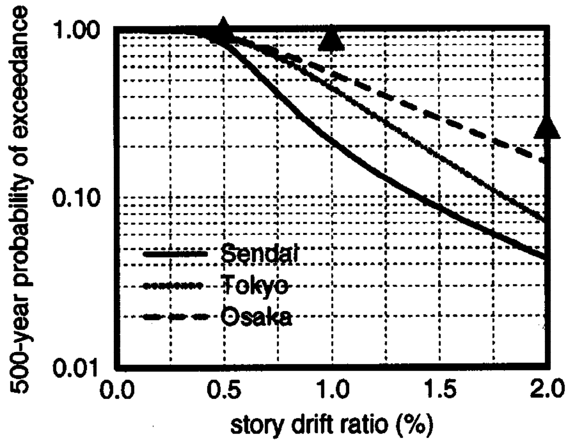
(c) 7-story RC buildings (t = 100 years)



(d) 12-story RC buildings (t = 100 years)



(e) 7-story RC buildings (t = 500 years)



(f) 12-story RC buildings (t = 500 years)

Fig.6. Reliability performance curves

CONCLUSIONS

The conclusions drawn from the study are follows:

1) Efficient procedures to evaluate seismic reliability of structures considering the uncertainties on input ground motions are presented. Since the presented method reduces the burden of calculation works considerably, it can be applied to rather complicated problems such as the seismic risk analysis of large scale structures.
2) Reliability performance curves of reinforced concrete buildings are evaluated for three different sites, Sendai, Tokyo and Osaka. The seismic risk at Sendai site is found to be smaller than the other sites. The upper bounds of calculated reliability performance curves are evaluated using some specific values of failure probabilities.

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