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SEISMIC RISK EVALUATION OF R.C. BUILDINGS IN JAPAN DESIGNED IN ACCORDANCE WITH 1990 AIJ GUIDELINES

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CHAPTER 1
INTRODUCTION

1.1 Objective and Scope

Structural design is a process of finding optimum design parameters to satisfy specified objectives such as serviceability, safety of life and property, and economy. In high seismicity areas such as Japan and the western United States, earthquake–resistant design considerations usually dominate the design process.

A commonly accepted philosophy in seismic design is as follows :

- When subjected to a moderate earthquake, a building should remain serviceable and little or no structural yielding should occur. The structural design process in this case typically involves finding the structural stiffness required to satisfy elastic deformation criteria.
- When subjected to a strong earthquake, a building should not collapse. The structural design process in this case focuses on determining the structural strength and design details required to provide the ductility and energy dissipation capacity associated with inelastic structural response during a strong earthquake.

Although the design philosophy is simple to understand, the seismic design procedure consistent with this philosophy is not so simple because of the large uncertainty associated with the seismic load. Furthermore, inelastic structural response is commonly acceptable for large earthquakes, and analysis procedures for inelastic response can be quite complicated. Therefore, the seismic design criteria specified in design codes are typically based on experience and engineering judgment.

Recently, extensive studies have been made in the area of reliability–based design methods. Earthquake ground motions have been modeled by nonstationary random processes considering time–varying intensity and frequency content. Probabilistic methods have been used to quantify structural safety in terms of probabilities of exceeding design thresholds in the lifetime of a building.

The objective of this study is to use probabilistic methods to evaluate the seismic safety of reinforced concrete buildings designed according to a new Japanese design guideline. Based on historical earthquake data available at two sites in Japan, simulated earthquake ground motions are generated using a recently developed nonstationary stochastic process model. Seismic risk analyses are performed to determine the probability of exceeding various design thresholds in a building's lifetime. The safety level implied in the design guideline is evaluated based on the calculated exceedance probabilities.

1.2 Outline

The procedures used to model earthquake ground motions are described in Chapter 2. Two cities in high seismicity areas of Japan are considered –Sendai and Tokyo. The parameters for the probability distribution models of earthquake magnitude and epicentral distance are estimated based on historical earthquake data for the two sites. Using information on magnitude, epicentral distance, and local ground conditions, the acceleration response spectrum at each site is evaluated using an empirical attenuation formula. A power spectral density function of earthquake ground motion which is compatible with the response spectrum is then estimated by an iterative calculation method. Using the calculated power spectrum and earthquake records measured at the site, parameters in the nonstationary random process model are identified and samples of earthquake ground motions are generated by filtering equations. These simulated records are used as input to the nonlinear structural response analysis.

Two different reinforced concrete buildings are designed according to a new design guideline proposed by the Architectural Institute of Japan (AIJ) in 1990. The seismic design procedures are described in Chapter 3. Nonlinear static analyses are carried out to confirm that the designs satisfy the two limit state design criteria in the guideline. The computer program FRAME–D developed at Tohoku University is used for the analysis.

In Chapter 4, risk analyses are carried out to determine exceedance probabilities of various design thresholds. Earthquake intensities are quantified by the effective peak ground acceleration (EPA) and the effective peak ground velocity (EPV), and the probabilities of exceeding various values of EPA and EPV in a building's lifetime are calculated. The exceedance probabilities of two different maximum structural response quantities are also examined ; one is the linear elastic acceleration response spectrum and the other is the maximum story drift of the designed reinforced concrete buildings. Based on the calculated exceedance probabilities, the safety level implied in the Japanese design guideline is discussed. A summary of the research results and conclusions are presented in Chapter 5.

CHAPTER 2
MODELING OF EARTHQUAKE GROUND MOTIONS

2.1 Introduction

The nature of earthquake ground motion depends on many factors such as seismic source characteristics, seismic wave propagation, and local site conditions. Although a large amount of study has been devoted to earthquake motion, the prediction of future earthquakes is still a difficult task. To handle the large uncertainty inherent in earthquake prediction, it is necessary to use probabilistic methods.

The occurrence rate of earthquakes is a basic piece of information needed for the modeling of future earthquakes. The variation of occurrence rate with magnitude is typically modeled by an empirical formula based on historical earthquake data. The Gutenberg–Richter relation is an example of this type of formula (Gutenberg and Richter, 1942). Similar empirical formulas for modeling the variation of occurrence rate as a function of source distance can also be developed.

In addition to occurrence rate, other parameters related to earthquake motions such as peak ground acceleration, response spectra, and earthquake duration are estimated by empirical attenuation formulas. Boore and Joyner (1982) and Campbell (1985) had reviewed various attenuation equations for peak ground acceleration in the United States. In Japan, the first attenuation equation for peak acceleration was developed by Kanai *et al.* (1966). Recently, Fukushima and Tanaka (1990) reviewed existing acceleration attenuation equations for Japan and proposed a new attenuation equation. An attenuation formula for acceleration response spectra was developed by Katayama (1982) for sites in Japan. A similar study for the United States was performed by Trifunac and Anderson (1977). Empirical attenuation formulas for earthquake duration are also available for Japan (Hisada and Ando, 1976) and for the United States (Trifunac and Brady, 1975).

A model of future earthquake ground motions can be made using random vibration theory. Stochastic process models of earthquake motions have been proposed by many researchers. Yeh and Wen (1989) recently proposed a nonstationary stochastic process model which accounts for changes in earthquake intensity and frequency content over time. This stochastic process model was used for simulating earthquake ground motions at sites in the United States by Eliopoulos and Wen (1991). This study also adopts the model to generate earthquake ground motions at sites in Japan.

This chapter describes the methodology used to generate simulated earthquake records. Two different cities in Japan – Sendai and Tokyo – are considered as the sites for the building designed in this study. Probability distribution models of the magnitude and the epicentral distance for both sites are obtained based on historical earthquake data. Parameters for the earthquake ground motion model proposed by Yeh and Wen are identified using the actual earthquake records observed at the two sites. The resulting simulated ground motions are used in the risk analysis which is discussed in Chapter 4.

2.2 Seismic Environment at Sites

2.2.1 General

The Japanese island arc is located along the eastern border of the Eurasian Plate. The Pacific Sea Plate and the Philippine Sea Plate are being subducted beneath the Eurasian Plate from the east and south, respectively. As a result, almost all areas in Japan are susceptible to earthquake. Figure 2.1 shows the distribution of earthquake sources around the eastern part of Japan. The data are taken from historical earthquake records between 1605 and 1992. The minimum magnitude of the data is 7.0 on the JMA (Japan Meteorological Agency) magnitude scale. This corresponds approximately to a magnitude of 6.8 on the local Richter scale. Table 2.1 shows a list of damaging earthquakes in Japan selected from "Chronological Scientific Tables" (edited by Japan National Astronomical Observatory, 1992). Many offshore earthquakes have been accompanied by destructive tunamis (seismic sea waves). The latest large earthquake, the 1993 Hokkaido Nansei–oki earthquake, generated a tunami with a height exceeding 20m (65 ft).

As noted earlier, the sites assumed in this study are Sendai and Tokyo. As shown in Figure 2.1, both cities are located near a densely concentrated area of earthquake occurrence. Both of these cities have experienced destructive earthquakes in this century.

Sendai has a population of nearly one million people. This city has an important role as a political and economic center of the Tohoku area (northern part of Japan). In 1978, the Miyagiken–oki earthquake caused severe damage in Sendai. It was the first time that a big modern city was struck by a large earthquake. In addition to the 28 deaths, the city's lifeline systems, such as water, electricity, and gas, were severely damaged. The locations of earthquake sources in Tohoku area are shown in Figure 2.2. A dense area of earthquake occurrences exists about 200km to the east of Sendai.

The Tokyo metropolitan area, including neighboring prefectures, has a population of more than 31 million people (which is about one–fourth of the entire population of Japan). Tokyo's last big earthquake was the 1923 Great Kanto earthquake in which over 140,000 people were killed. The main cause of death was the fire which started after the earthquake. Today, many seismologists believe that the next large earthquake in Japan will occur in the Tokyo area in the near future (within 10 years) and that the disaster could be far worse than it was in 1923. The earthquake sources around Tokyo are shown in Figure 2.3. It is important to note that the earthquake sources around Tokyo are located much closer to the city than the earthquake sources around Sendai.

The sections below describe the procedures used in this study to develop simulated earthquake ground motions for Sendai and Tokyo.

2.2.2 Modeling of Earthquake Occurrence

Based on the seismicity data recorded between 1900 and 1992, the following data are adopted as representative of the Sendai and Tokyo sites :

1) Epicentral distance, R, is less than 350km.

2) Magnitude, M, is greater than 5.5.

3) Focal depth is less than 100 km.

For the Sendai site, there are 974 records which satisfy these conditions, and the annual occurrence rate of earthquakes is 10.8. For the Tokyo site, there are 721 records, and the annual occurrence rate is 7.8. Since the JMA did not start the official earthquake observation process until 1885, the data above based on the period 1900–1992 are considered reliable.

Figure 2.4 presents a plot of the occurrence rate as a function of magnitude for the both sites. Also shown in Figure 2.4 are curves representing two empirical formulas used to model earthquake occurrence rate. The Gutenberg–Richter (G–R) formula for modeling earthquake occurrence is

$$\log N(M) = a - bM \qquad M > M_0 \tag{2.1}$$

where N(M) is the number of occurrences of earthquakes with magnitudes greater than M. M_0 is the minimum magnitude considered ; a value of 5.5 is selected for this study. The constants a and b are estimated using historical data. As shown in Figure 2.4, the occurrence rate in the region of high magnitude (M>7) is overestimated by the G–R formula. This overestimation may cause a significant error in the evaluation of seismic risk. The truncated G–R formula can be used to account for an upper bound magnitude, Mu. The formula is expressed as

$$\log N(M) = a + \log(10^{-bM} - 10^{-bMu}) \qquad M_0 < M < Mu \tag{2.2}$$

Since the maximum earthquake magnitudes ever observed are 8.5 around the Sendai site and 8.4 around Tokyo site, the upper bound is selected as Mu = 8.5 for both sites. As shown in Figure 2.4, the truncated formula provides a better estimate of occurrence rate in the high magnitude range. Using this truncated G–R formula for N(M), the cumulative distribution function of magnitude is obtained as

$$F_M(m) = 1 - N(m)/N_0, \qquad N_0 = N(M_0) \tag{2.3}$$

Another important aspect to consider in modeling earthquake ground motion is the variation of occurrence rate with epicentral distance, R. Figure 2.5 shows the frequency diagram of epicentral distances for the records of each site. For the Sendai site, it is observed that the occurrence rate increases as the epicentral distance increases. Unlike the Sendai site, the occurrence rate for the Tokyo site increases with the epicentral distance up to about 200 km and decreases for epicentral distances above 200 km. These diagrams can be roughly modeled by triangular distributions as shown by the dotted lines in Figure 2.5.

The relations between magnitude, M, and epicentral distance, R, are shown in Figure 2.6 for both sites. The correlation coefficient, ρ, between M and R is relatively small in each case ; $\rho = -0.0265$ for the Sendai site and $\rho = -0.0158$ for the Tokyo site. Therefore, this study assumes that M and R are mutually independent random variables.

The probability distribution models of magnitude, M, and epicentral distance, R, are developed based on the results presented in Figures 2.4 and 2.5. Although the annual earthquake occurrence rate of the Tokyo site is smaller than that of the Sendai site, the earthquake source distance of the Tokyo site is much closer than that of the Sendai site. Thus, it is difficult to determine which site has a higher overall seismic risk.

2.2.3 Attenuation Formulas

Attenuation equations are used to relate the intensity of earthquake ground motions to the magnitude and the source distance. Typically, peak ground acceleration is adopted as a measure of the intensity of an earthquake. However, to construct the model of earthquake ground motions, information on frequency content is also required. An attenuation formula for Fourier spectra or response spectra can satisfy this requirement. Katayama (1982) proposed the following attenuation formula for acceleration response spectrum ordinates at a damping value of 5 % of critical :

$$RS = \alpha \overline{RS} \tag{2.4}$$

$$\overline{RS} = f_M f_R f_{GC} \tag{2.5}$$

where RS and $\overline{RS}$ represent the actual and estimated acceleration response spectrum ordinates, respectively. The terms f_M, f_R, and f_{GC} are weighting factors for magnitude, epicentral distance, and site soil conditions, respectively. Values of the weighting factors were established by Katayama at 18 different periods; these values are presented in Table 2.2. The α factor accounts for the uncertainty in the predicted response spectra ($\overline{RS}$). Its probability density function is modeled by a lognormal distribution with mean $\mu_\alpha = 1.267$ and standard deviation $\sigma_\alpha = 1.025$. For example, for M=7.0, R=100 km and Type 1 ground conditions, the predicted absolute acceleration response spectrum ordinate for a period (T) of 1.0 sec at 5% damping is obtained using Table 2.2 as follows :

$$\overline{RS} = 0.636 \times 1.40 \times 43.3 = 38.55 \text{ cm/sec}^2 \quad (2.6)$$

Information on earthquake duration is also needed for modeling earthquake ground motions, especially for inelastic structural response analyses. The following empirical relation is commonly used in Japan to relate earthquake magnitude to earthquake duration (Hisada and Ando, 1976) :

$$\log_{10} t_d = 0.31M - 0.774 \quad (2.7)$$

It is assumed that the uncertainty of this relation is much smaller than the uncertainty associated with the attenuation relation. Therefore, only the attenuation uncertainty in Equation (2.4) is considered in the risk analysis carried out as part of this study.

2.3 Earthquake Ground Motion Model

The nonstationary random process model developed by Yeh and Wen (1989) is used to generate simulated earthquake ground motions. The ground motion is modeled as an amplitude and frequency modulated random process and is given by

$$a(t) = I(t)\,\varsigma[\phi(t)] \quad (2.8)$$

where $\varsigma(\phi)$ is a zero mean, unit variance, stationary filtered white noise having a Clough–Penzien power spectrum, I(t) is an intensity function, and $\phi(t)$ is a frequency modulation function. The identification of the parameters to be used in this model is described in the following sections.

2.3.1 Identification of Intensity Function Parameters

The following formula is used for the intensity function :

$$I^2(t) = A \frac{t^B e^{-Ct}}{D + t^E} \tag{2.9}$$

Yeh and Wen (1989) reported that this functional form can express a wide variety of shapes of the intensity envelope. The parameters A, B, C, D and E are estimated from an energy function of an actual earthquake record measured at the site. The energy functions of the record and of the intensity function are defined as

$$E(t) = \int_0^t w^2(t)\, dt \approx \int_0^t I^2(t)\, dt \tag{2.10}$$

where w(t) is the earthquake acceleration record at the site. A nonlinear least squares method is used to identify the parameters to be used in Equation (2.9). For the Sendai site, the earthquake record used for parameter identification was the record measured at Tohoku University during the 1978 Miyagiken–oki earthquake. For the Tokyo site, the acceleration record measured at the University of Tokyo in 1956 is used. Figure 2.7 provides comparisons of energy functions for the actual earthquake records and for the intensity functions identified from the records. Note that these intensity functions are normalized to have a unit total energy. The normalized intensity functions for both sites are shown in Figure 2.8 and the identified parameters are listed in Table 2.3.

The significant duration, t_d, is defined by Trifunac and Brady (1975) as the time interval between 5% and 95% of the total energy. This study assumes that the initial time, t_0, is the time at which 5% of the total cumulative energy is achieved and the total duration, t_F, is the duration of the record. Both factors, t_0 and t_F, can be expressed as the following functions of t_d :

$$t_0 = c_1 t_d \tag{2.11}$$

$$t_F = t_0 + c_2 t_d \tag{2.12}$$

where c_1 and c_2 are constants obtained from the record. The values for t_0, t_d, t_F, c_1, and c_2 for each site are listed in Table 2.4.

The intensity function for each site shown in Figure 2.8 was developed based on one specific earthquake record measured at the site. A different earthquake record at the same site is assumed to have the same general intensity function shape shown in Figure 2.8. If the new record has a significant duration, t_{d1}, the parameters in the new intensity function, (A_1–E_1), can be obtained from the parameters in the original intensity function, (A–E), by the following relations :

$$\eta = t_{d1}/t_d\,,\quad A_1 = \eta^{-B+E-1} A\,,\quad B_1 = B\,,\quad C_1 = C/\eta\,,\quad D_1 = \eta^E D\,,\quad E_1 = E \tag{2.13}$$

2.3.2 Identification of Frequency Modulation Function Parameters

The frequency modulation function is defined as a third order polynomial of the form

$$\phi(t) = \mu_0(t)/\mu_0'(t_0) = r_1\, t + r_2\, t^2 + r_3\, t^3 \tag{2.14}$$

where $\mu_0(t)$ is the mean number of zero crossings and $\mu_0'(t_0)$ is the time derivative of $\mu_0(t)$ evaluated at $t = t_0$. Since the function $\mu_0(t)$ is monotonically increasing, the third order polynomial is usually sufficient to express the frequency modulation function $\phi(t)$. The coefficients r_1, r_2, and r_3 are estimated from the observed earthquake records at a site. Figure 2.9 presents the actual frequency modulation functions for earthquake records and the derived frequency modulation functions of the form of Equation (2.14). Any other records at the same site are assumed to have the same rate of change of frequency content. To maintain the same value of $\phi'(t/t_F)$ among different records, coefficients in a new frequency modulation function, (r_{11}, r_{21}, r_{31}), for a different record with a duration t_{F1} can be obtained from the coefficients in the original intensity function, (r_1, r_2, r_3), as follows :

$$\xi = t_{F1}/t_F\,,\quad r_{11} = r_1\,,\quad r_{21} = \xi\, r_2\,,\quad r_{31} = \xi^2\, r_3 \tag{2.15}$$

The estimated values of r_1, r_2, and r_3 and the duration, t_F, for each site are listed in Table 2.5.

2.3.3 Identification of Clough–Penzien (C–P) Spectrum Parameters

As described below, the parameters to be used in the C–P spectrum are identified based on an estimated power spectrum which is compatible with the target response spectrum.

Each set of values of M, R, and α can be related to a target response spectrum $RS_t(\omega)$ using Equation (2.4). The power spectrum which is compatible with $RS_t(\omega)$ is estimated by the following iterative process. Starting with an arbitrary power spectrum $PS_i(\omega)$ (i=0), a nonstationary random process, $x_i(t)$, is generated using

$$x_i(t) = I(t)\, s_i(t) \tag{2.16}$$

$$s_i(t) = \sqrt{2} \sum_{m=1}^{N} \sqrt{PS_i(\omega_m)\Delta\omega}\, \cos(\omega_m t + \zeta_m) \tag{2.17}$$

where $s_i(t)$ = a stationary random process with the power spectrum $PS_i(\omega)$

ζ_m (m=1,..N) = a uniformly distributed random number such that $0 \leqq \zeta_m < 2\pi$

$\Delta\omega = (\omega_U - \omega_L) / N$

ω_U, ω_L = the upper and lower bound of ω, respectively

$\omega_m = \omega_L + (m-1)\Delta\omega$

The total duration of $x_i(t)$ is to be equal to the time t_F obtained by Equation (2.12). The linear response spectrum $RS_i(\omega)$ corresponding to $x_i(t)$ is then calculated for 5% damping. With this information, an improved estimate of the power spectrum is obtained using the following relation discussed by Shinozuka (1988) :

$$PS_{i+1}(\omega) = PS_i(\omega) \left(\frac{RS_t(\omega)}{RS_i(\omega)} \right)^k \tag{2.18}$$

The exponent k is set equal to 2.0 in the present study. This process is then repeated until convergence is achieved.

The above procedure was used to generate three sample records. For each record, four iterations were required to obtain a wave for which the calculated response spectrum matched the target within an acceptable tolerance. The averaged response spectrum, corresponding to the average of the response spectra for the three sample waves, is then compared with the target response spectrum. Comparisons of the target and average response spectra are shown in Figure 2.10 for sampled values of earthquake data set, (M,R,α). It is seen that the average response spectra fit the target response spectra reasonably well.

For each of the three waves, a power spectrum is calculated. Then, the average of these power spectra (herein referred to as the average power spectrum) is used to identify the parameters of the Clough–Penzien spectrum which is expressed as

$$PS_{CP}(\omega) = S_0 \left[\frac{\omega_g^4 + 4\zeta_g^2\omega_g^2\omega^2}{(\omega_g^2 - \omega^2)^2 + 4\zeta_g^2\omega_g^2\omega^2} \right] \left[\frac{\omega^4}{(\omega_f^2 - \omega^2)^2 + 4\zeta_f^2\omega_f^2\omega^2} \right] \tag{2.19}$$

where S_0, ω_g, h_g, ω_f, and h_f are the spectral parameters to be identified. Figure 2.11 provides a comparison of the average power spectrum and the identified C–P spectrum for the sample earthquake data at each site. The parameter values shown in the figure are obtained by an optimization analysis using a nonlinear least squares method.

2.3.4 Generation of Nonstationary Random Processes

After all parameters are identified, a sample function a(t), corresponding to an amplitude and frequency modulated random process, is generated by the following filtering equations (Yeh and Wen, 1989) :

$$\frac{d^2x_g}{dt^2} + \left(-\frac{\phi''(t)}{\phi'(t)} + 2\zeta_g\omega_g\phi'(t) \right) \frac{dx_g}{dt} + [\omega_g\phi'(t)]^2\, x_g = -[\phi'(t)]^2\, I(t)\, \xi(\phi(t)) \tag{2.20}$$

$$\frac{d^2x_f}{dt^2} + \left(-\frac{\phi''(t)}{\phi'(t)} + 2\zeta_f\omega_f\phi'(t) \right) \frac{dx_f}{dt} + [\omega_f\phi'(t)]^2\, x_f = -2\zeta_g\omega_g\phi'(t)\, \frac{dx_g}{dt} - [\omega_g\phi'(t)]^2\, x_g \tag{2.21}$$

$$a(t) = \frac{2\zeta_f\omega_f}{\phi'(t)}\frac{dx_f}{dt} + \omega_f^2\, x_f + \frac{2\zeta_g\omega_g}{\phi'(t)}\frac{dx_g}{dt} + \omega_g^2\, x_g \tag{2.22}$$

where $\xi(\phi)$ is a zero mean, unit variance, stationary white noise in ϕ with power spectral density S_0. An example of a generated function for each site is shown in Figure 2.12. The response spectrum corresponding to each of these sample waves is calculated and compared with the target spectrum in Figure 2.13. Although some deviation between the calculated and target response spectra is observed, this deviation is judged to be acceptable.

Figure 2.14 presents a flowchart which summarizes the various steps in the procedure to generate ground motions.

Table 2.1 List of destructive earthquakes in Japan

name	area	year	magnitude	casualty
Keityo - jishin	(C) Tokai,Nankai	1605.2.3	7.9	over 2,000 (by tsunami)
Genroku -	(C) Tokai	1703.12.31	7.9-8.2	over 2,300
Houei -	(C) Tokai,Kii	1707.10.28	8.4	20,000
Yaeyame -	(E) Yaeyama	1771.4.24	7.4	12,000 (by tsunami)
Sakikata -	(B) Uzen,Ugo	1804.7.10	7.0	over 500
Zenkouji -	(C) Shinano,Echigo	1847.5.8	7.4	over 5,867
Ansei Toukai -	(C) Tokai	1854.12.23	8.4	2,000 - 3,000
Ansei Nankai -	(C) Kinki,Hokuriku Tokai,Tyugoku	1854.12.24	8.4	?,000
Edo -	(C) Edo	1855.11.11	6.9	over 4,000 (mainly by fire)
Hamada -	(D) Ishimi,Izumo	1872.3.14	7.1	over 600
Nobi -	(D) Aichi,Gifu	1891.10.28	8.0	7,273
Shonai -	(C) Shonai heiya	1894.10.22	7.0	726
Meiji Sanriku-oki jishin tsunami	(B) Sanriku	1896.6.15	8.5	22,072 (all by tsunami)
Rikuu -	(B) Akita,Iwate	1896.8.31	7.2	209
Miyagiken-Hokubu	(B) Miyagi	1900.5.12	7.0	11
Aomoriken-tohooki	(B) east of Aomori	1901.8.9-10	7.2-7.4	18
Geiyo -	(D) Aki-Nada	1905.6.2	7.2	11
Anekawa -	(D) east of Shiga	1909.8.14	6.8	41
Kikaijima -	(E) Amami-ohshima	1911.6.15	8.0	12
Sakurajima -	(E) Kagoshima	1914.1.12	7.1	35
Akitaken-Senhoku	(B) Akita	1914.3.15	7.1	94
Shimabara -	(E) Nagasaki	1922.12.8	6.9	30
Kanto - (Great Kanto)	(C) Tokyo	1923.9.1	7.9	142,807
Tanzawa -	(C) west of Kanagawa	1924.1.15	7.3	19
Tajima -	(D) Hyogo	1925.5.23	6.8	428
Kita tango -	(D) Kyoto	1927.3.7	7.3	2,925
Kita Izu -	(C) Shizuoka	1930.11.26	7.3	272
Nishi Saitama	(C) Saitama	1931.9.21	6.9	16
Sanriku Oki -	(B) Sanriku	1933.3.3	8.1	3,008 (by tsunami)
Shizuoka -	(C) Shizuoka	1935.7.11	6.4	9
Kawachi-Yamato -	(D) north of Nara	1936.2.21	6.4	9
Fukushimaken oki -	(B) Fukushima	1938.11.5	7.5	1
Oga Hanto -	(B) Akita	1939.5.1	6.8	27
Shakotan Hanto oki -	(A) Shakotan	1940.8.2	7.5	10
Naganuma -	(C) Nagano	1941.7.15	6.1	5
Hyuga Nada -	(E) Miyazaki	1941.11.19	7.2	2
Tottori -	(D) Tottori	1943.9.10	7.2	1,083
Toh nankai -	(D) off east of Mie	1944.12.7	7.9	998
Mikawa -	(C) south of Aichi	1945.1.13	7.1	1,961 - 2,306
Nankai -	(D) off Kii Pen.	1946.12.21	8.0	1,330
Fukui -	(C) Fukui	1948.6.28	7.1	3,895
Imaichi -	(C) Tochigi	1949.12.26	6.2-6.4	10
Tokachi oki -	(C) off Tokachi	1952.3.4	8.2	28
Daishoji -	(C) west of Ishikawa	1952.3.7	6.5	7
Yoshino -	(D) west of Nara	1952.7.18	6.8	9
Boso Oki -	(C) Chiba	1953.11.26	7.4	0
Nagaoka -	(C) Niigata	1961.2.2	5.2	5
Hyuga Nada -	(B) Miyazaki	1961.2.27	7.0	2
Kita-Mino -	(C) Fukui	1961.8.19	7.0	8
Miyagiken Hokubu -	(B) Miyagi	1962.4.30	6.5	3
Akitaken oki -	(B) Akita	1964.5.7	6.9	0
Niigata -	(C) Niigata	1964.6.16	7.5	26
Shizuoka -	(C) Shizuoka	1965.4.20	6.1	2
Yonagunijima Kinkai -	(E) near Taiwan	1966.3.13	7.8	2
Ebino -	(E) north of Kagoshima	1968.2.22	5.6-6.1	3
Tokachi oki -	(A) off Tokachi	1968.5.16	7.9	52
Gifuken cyubu -	(D) Gifu	1969.9.9	6.6	0
Nemuro Hanto oki -	(A) Nemuro	1973.6.17	7.4	0
Izu Hanto oki -	(C) off Izu	1974.5.9	6.9	30
Oitaken tyubu -	(D) Oita	1975.4.21	6.4	0
Izu ohshima kinkai -	(C) Izu	1978.1.14	7.0	26
Miyagiken oki -	(B) off Miyagi	1978.6.12	7.4	28
Urakawa oki -	(B) off Urakawa	1982.3.21	7.1	0
Nihonkai Chubu -	(B) off Akita	1983.5.26	7.7	104 (by tsunami)
Naganoken Seibu -	(C) Nagano	1984.9.14	6.8	29
Chibaken Toho oki -	(C) off chiba	1987.12.17	6.7	2
Kushiro oki -	(A) off Kushiro	1993.1.15	7.8	1
Hokkaido Nansei oki -	(A) off okushiri	1993.7.12	7.8	176 (and 68 still missing)

(A):North portion of Japan (B):North-Central (C):Central (E):South

Table 2.2 Weighting factors for the attenuation formula (Equation 2.5)

T (sec)	fM Magnitude (M)					fR Epicentral distance (R:km)					fGC Ground condistion (GC)			
	4.5-5.3	5.4-6.0	6.1-6.7	6.8-7.4	7.5-7.9	6-19	20-59	60-119	120-199	200-405	Type 1	Type 2	Type 3	Type 4
0.1	0.218	0.278	0.296	0.399	1.00	5.10	2.67	2.05	1.00	1.00	126	107	120	106
0.15	0.225	0.274	0.297	0.448	1.00	4.85	3.01	2.15	1.00	1.00	155	130	141	125
0.20	0.185	0.280	0.288	0.499	1.00	5.48	3.24	2.07	1.05	1.00	169	149	161	129
0.25	0.171	0.254	0.283	0.534	1.00	6.86	3.65	2.33	1.21	1.00	135	129	143	129
0.30	0.164	0.269	0.280	0.548	1.00	6.59	3.51	2.25	1.27	1.00	109	130	147	131
0.35	0.161	0.274	0.302	0.588	1.00	5.74	3.05	2.13	1.24	1.00	92.8	126	149	142
0.40	0.152	0.268	0.311	0.557	1.00	5.45	3.01	1.92	1.33	1.00	83.0	122	145	144
0.50	0.108	0.237	0.309	0.593	1.00	6.35	2.91	1.60	1.36	1.00	76.6	113	140	156
0.60	0.0889	0.246	0.321	0.618	1.00	5.88	2.79	1.46	1.32	1.00	62.1	101	134	159
0.70	0.0730	0.222	0.315	0.644	1.00	6.77	2.96	1.56	1.37	1.00	50.0	88.8	118	148
0.80	0.0683	0.214	0.294	0.595	1.00	5.89	2.73	1.54	1.28	1.00	47.9	91.0	115	145
0.90	0.0672	0.214	0.285	0.581	1.00	5.13	2.38	1.48	1.20	1.00	46.4	90.5	113	136
1.00	0.0653	0.204	0.284	0.636	1.00	4.62	2.15	1.40	1.16	1.00	43.3	89.3	107	125
1.50	0.0503	0.138	0.204	0.534	1.00	4.40	2.20	1.44	1.00	1.00	33.0	56.5	68.5	84.6
2.00	0.0605	0.148	0.215	0.585	1.00	3.66	1.99	1.29	1.00	1.00	24.7	36.8	44.1	46.2
2.50	0.0587	0.136	0.183	0.405	1.00	3.50	1.95	1.34	1.00	1.00	21.9	32.7	35.8	33.0
3.00	0.0660	0.138	0.194	0.391	1.00	3.26	1.79	1.35	1.00	1.00	18.8	26.6	28.5	26.6
4.00	0.0704	0.144	0.187	0.395	1.00	2.81	1.61	1.27	1.00	1.00	15.7	20.3	24.1	19.1

Table 2.3 Parameters of intensity functions

	$I^2(t) = A\dfrac{t^B e^{-Ct}}{D + t^E}$				
	A	B	C	D	E
SENDAI	0.6378E+07	0.1391E+01	0.1572E+00	0.4633E+09	0.6776E+01
TOKYO	0.2397E+06	0.4007E+01	-0.118E+01	0.5131E+10	0.1641E+02

Table 2.4 Parameters for earthquake duration

	$t_0 = c_1\, t_d$ $t_F = t_0 + c_2\, t_d$				
	t_0 (sec)	t_d (sec)	t_F (sec)	c_1	c_2
SENDAI	2.78	17.2	40.96	0.162	2.22
TOKYO	3.00	4.86	11.40	0.617	1.73

Table 2.5 Parameters of frequency modulation functions

	$\phi(t) = r_1\, t + r_2\, t^2 + r_3\, t^3$			
	t_F (sec)	r_1	r_2	r_3
SENDAI	20.8	1.2112	-0.0416	0.0012
TOKYO	11.4	1.0091	0.0205	-0.0029

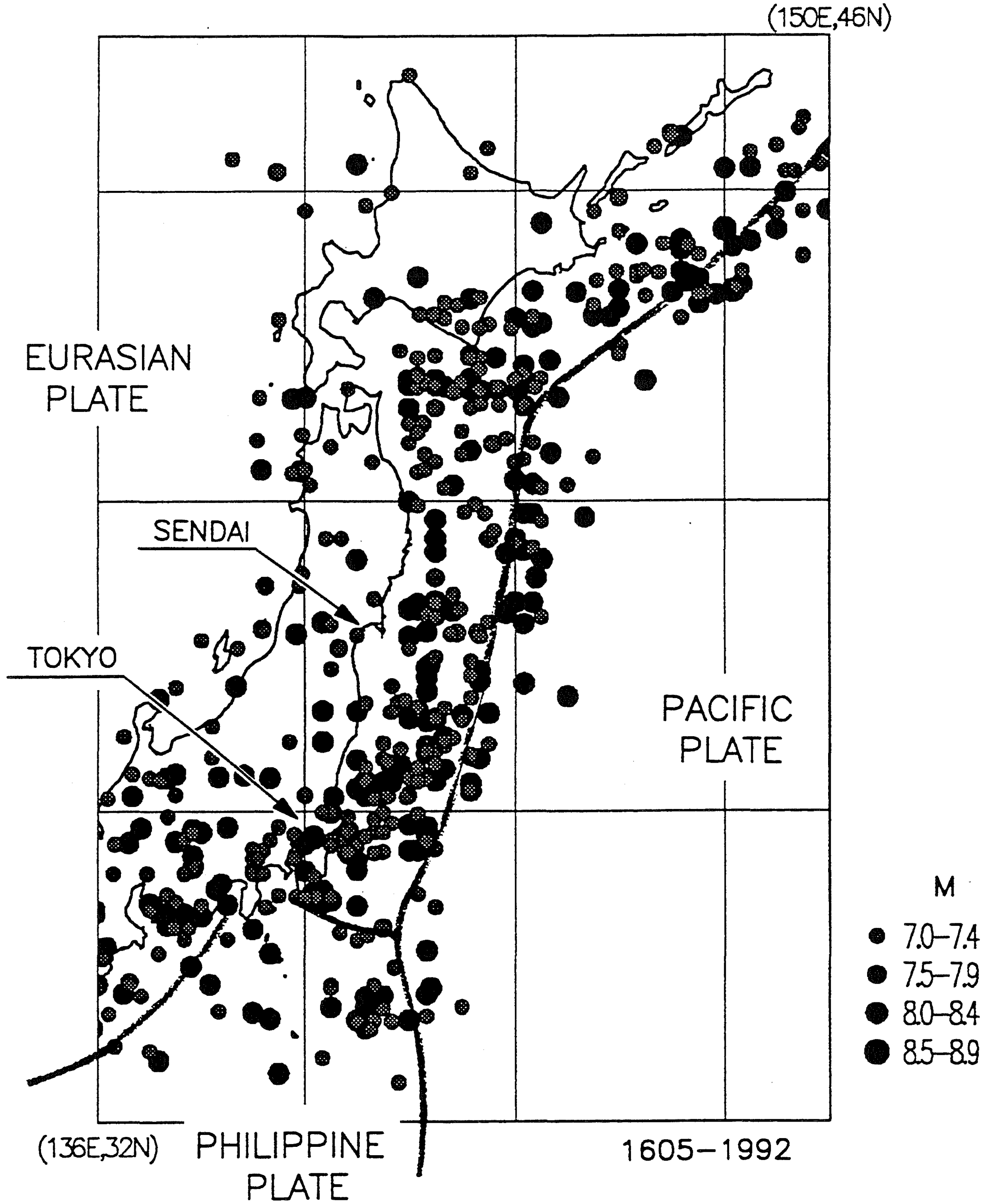


Figure 2.1 Distribution of earthquake sources around eastern Japan

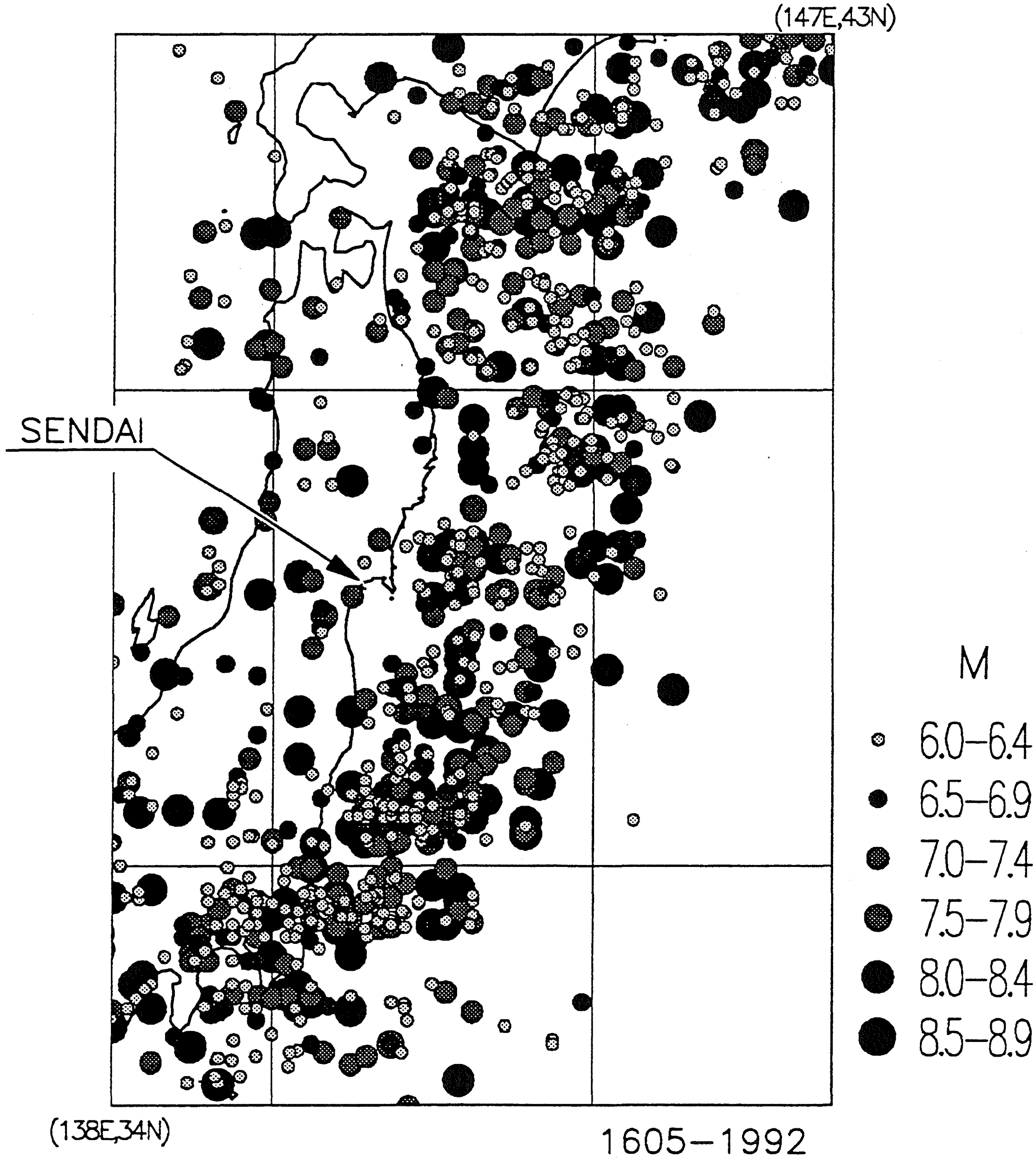


Figure 2.2 Distribution of earthquake sources around Sendai

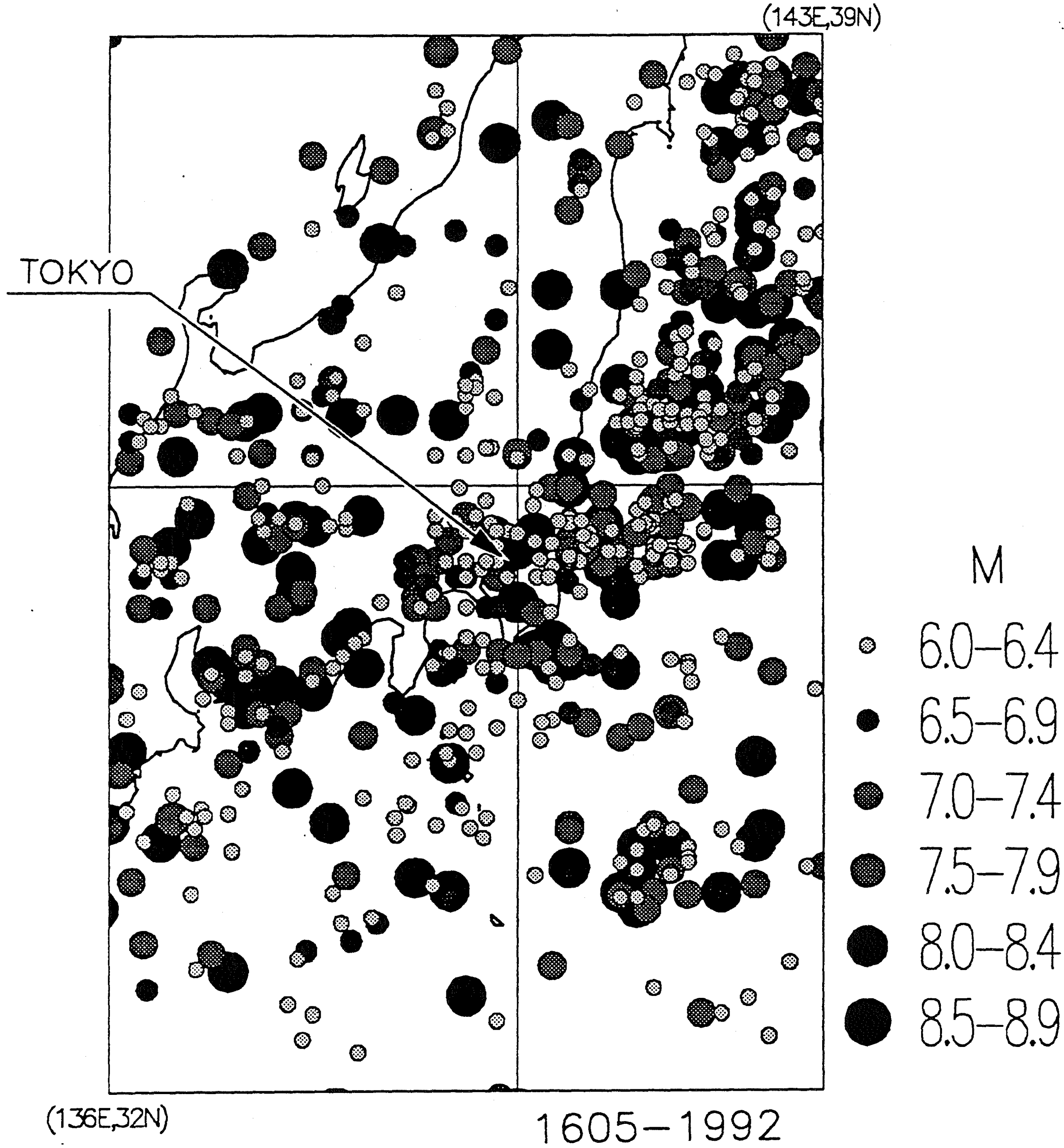
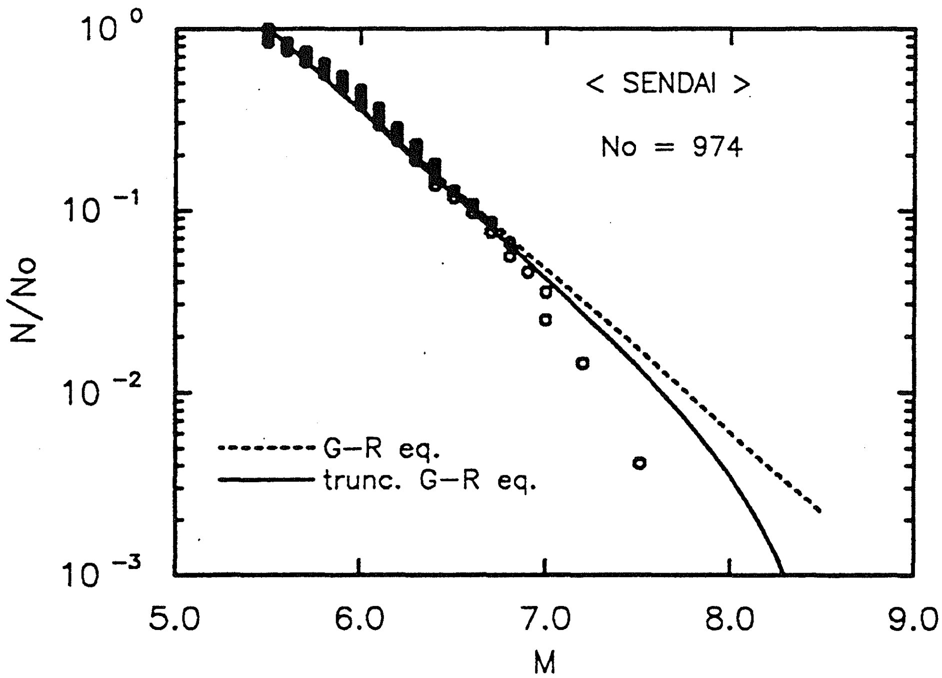


Figure 2.3 Distribution of earthquake sources around Tokyo



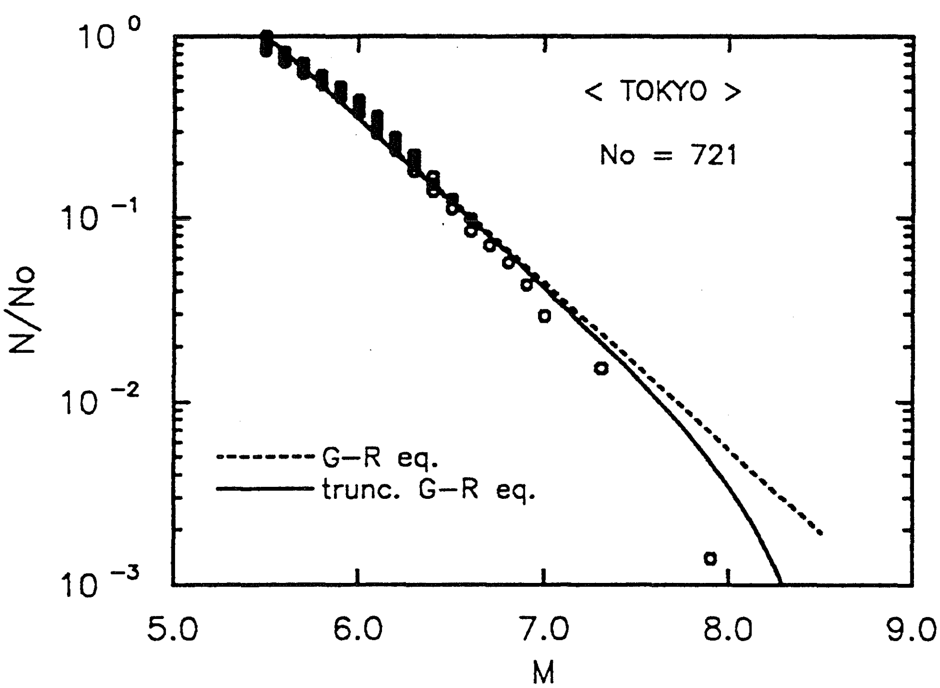


Figure 2.4 Relation between earthquake occurrence rate and magnitude

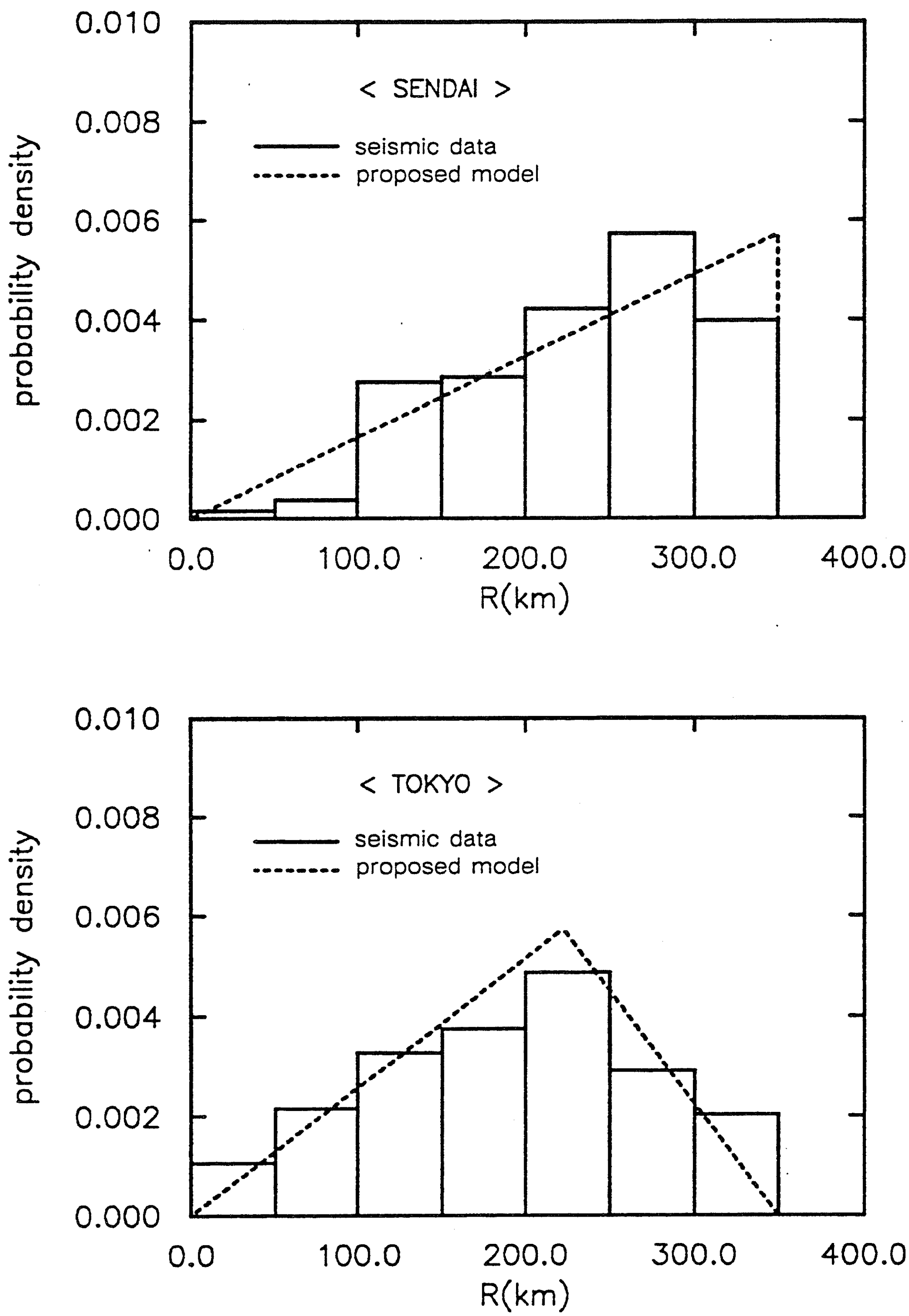


Figure 2.5 Frequency diagram of epicentral distances

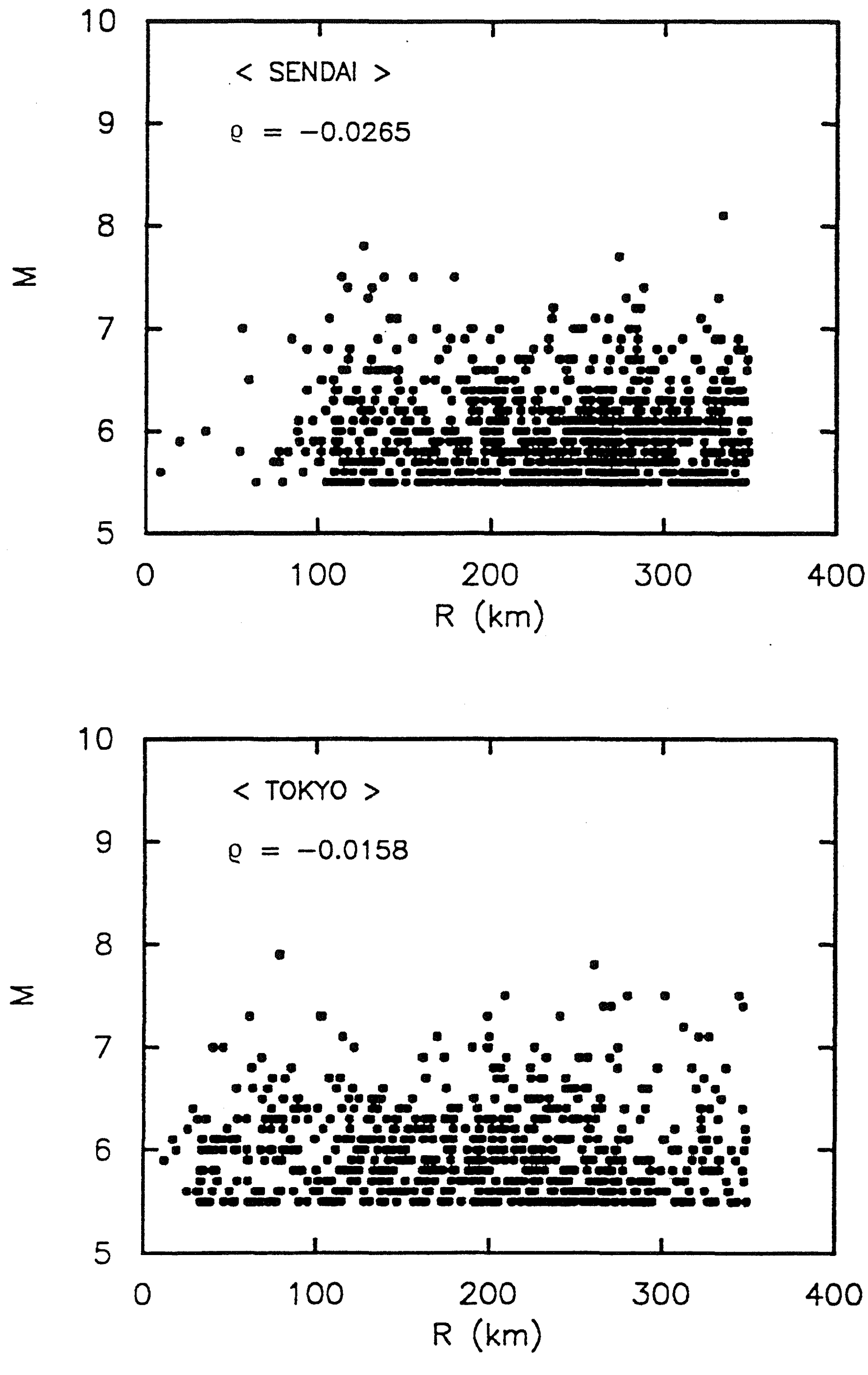


Figure 2.6 Relation between magnitude and epicentral distance

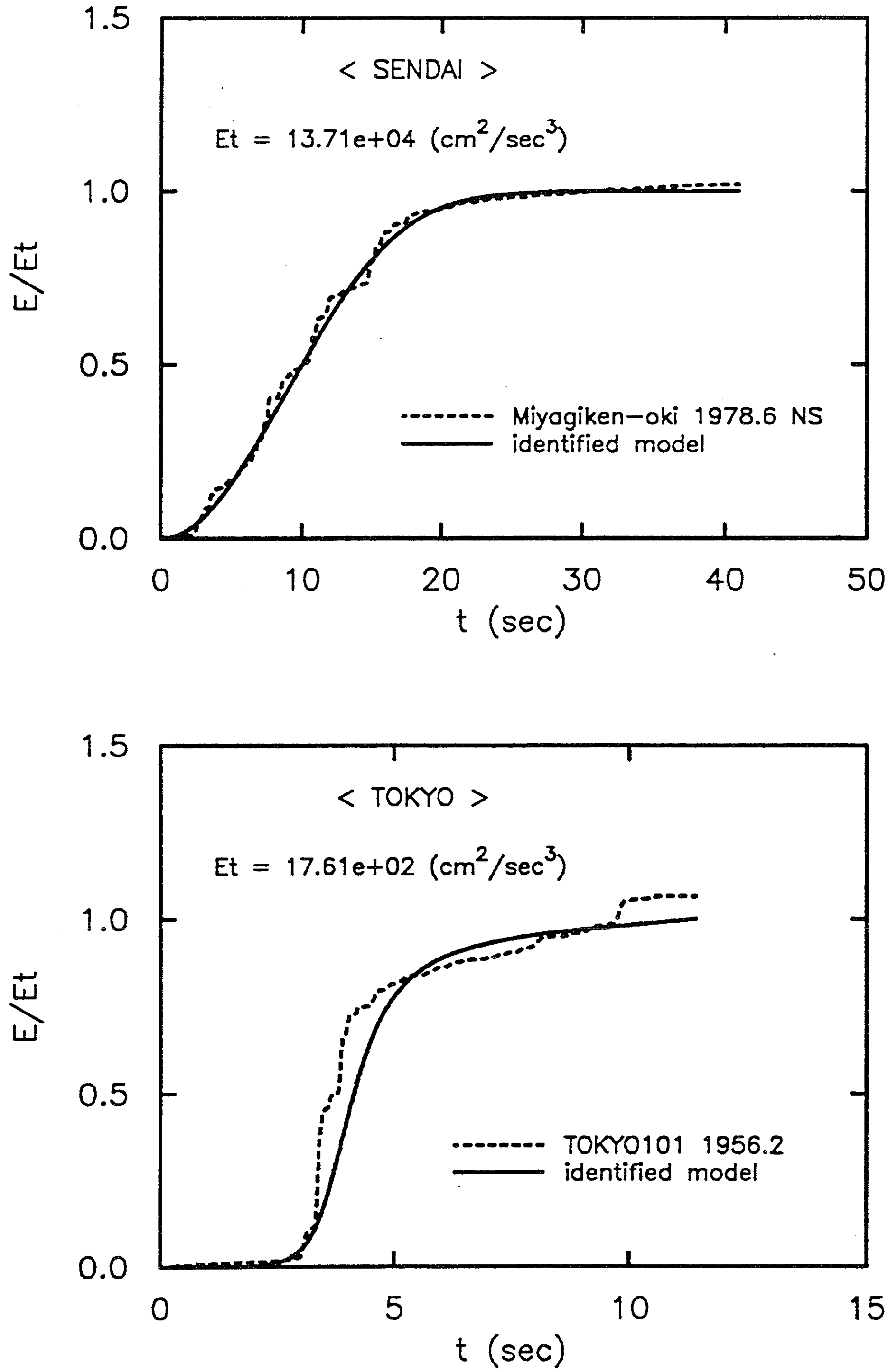


Figure 2.7 Expected energy function of earthquake record

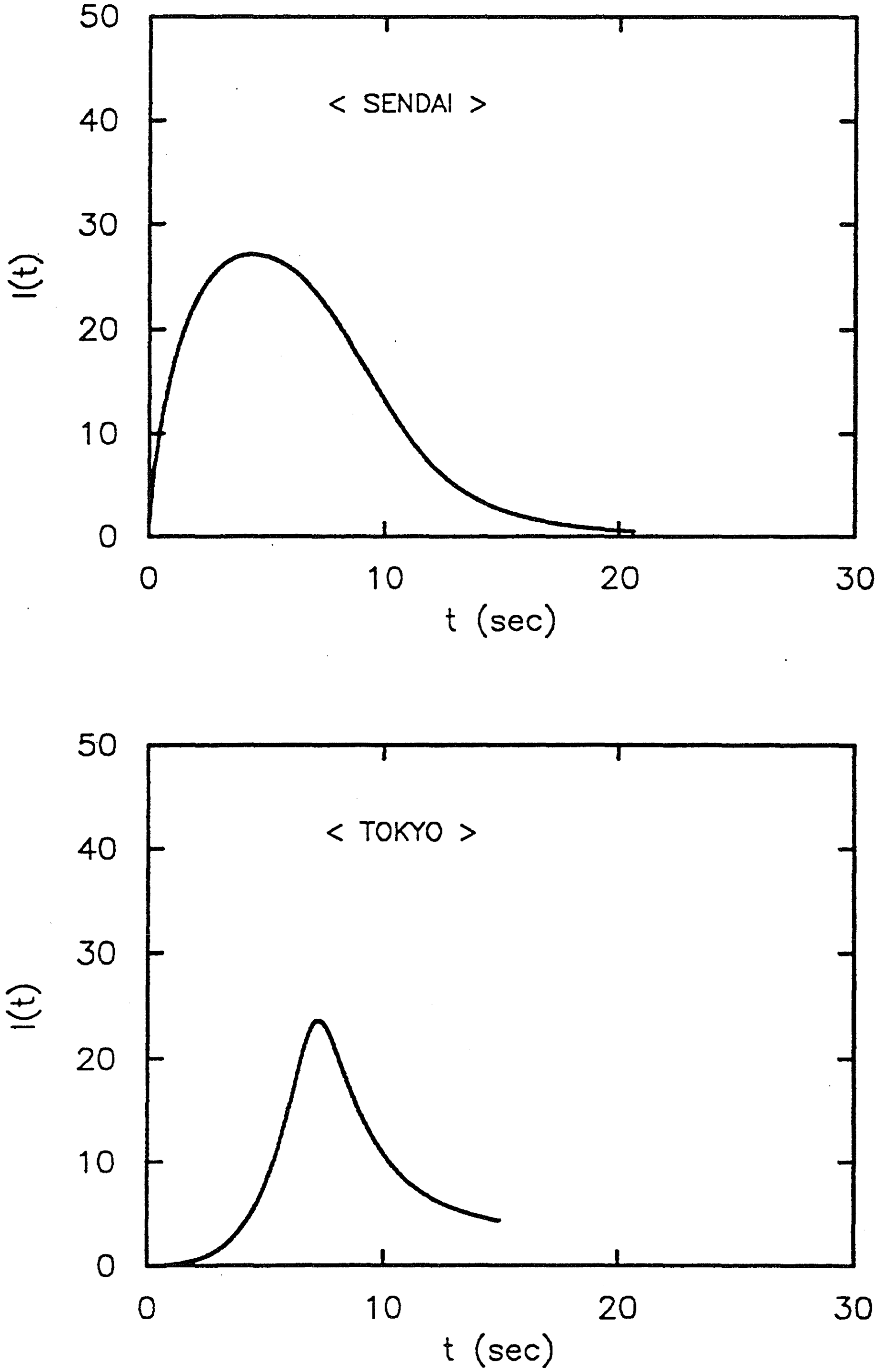
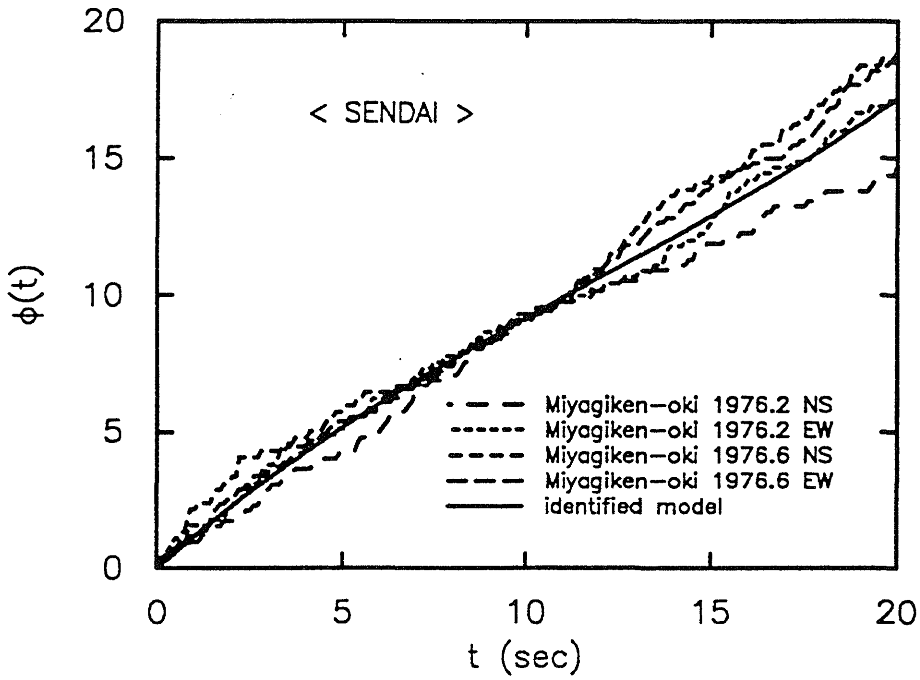


Figure 2.8 Normalized intensity functions



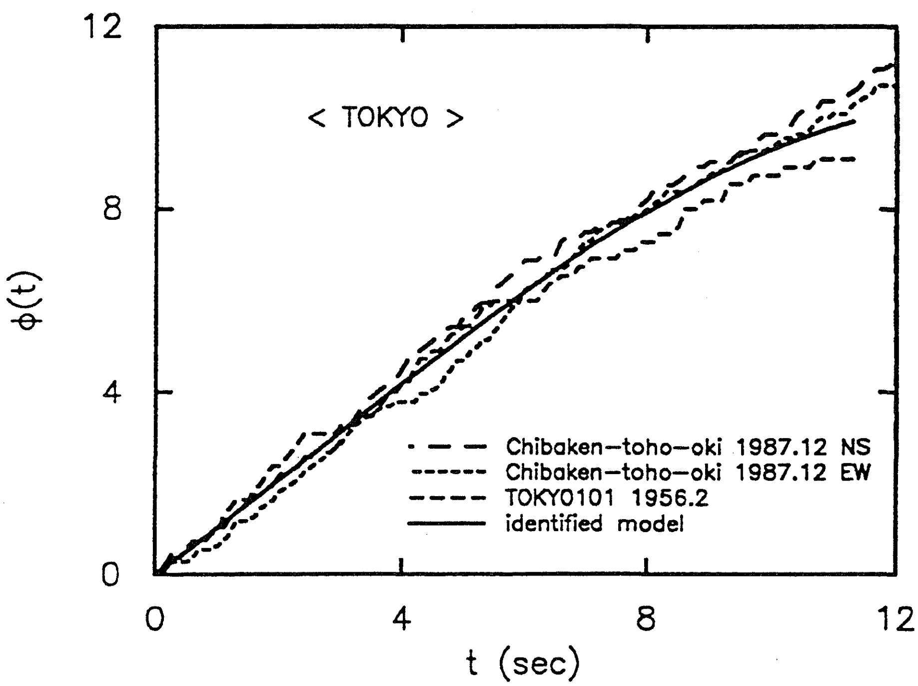


Figure 2.9 Frequency modulation functions

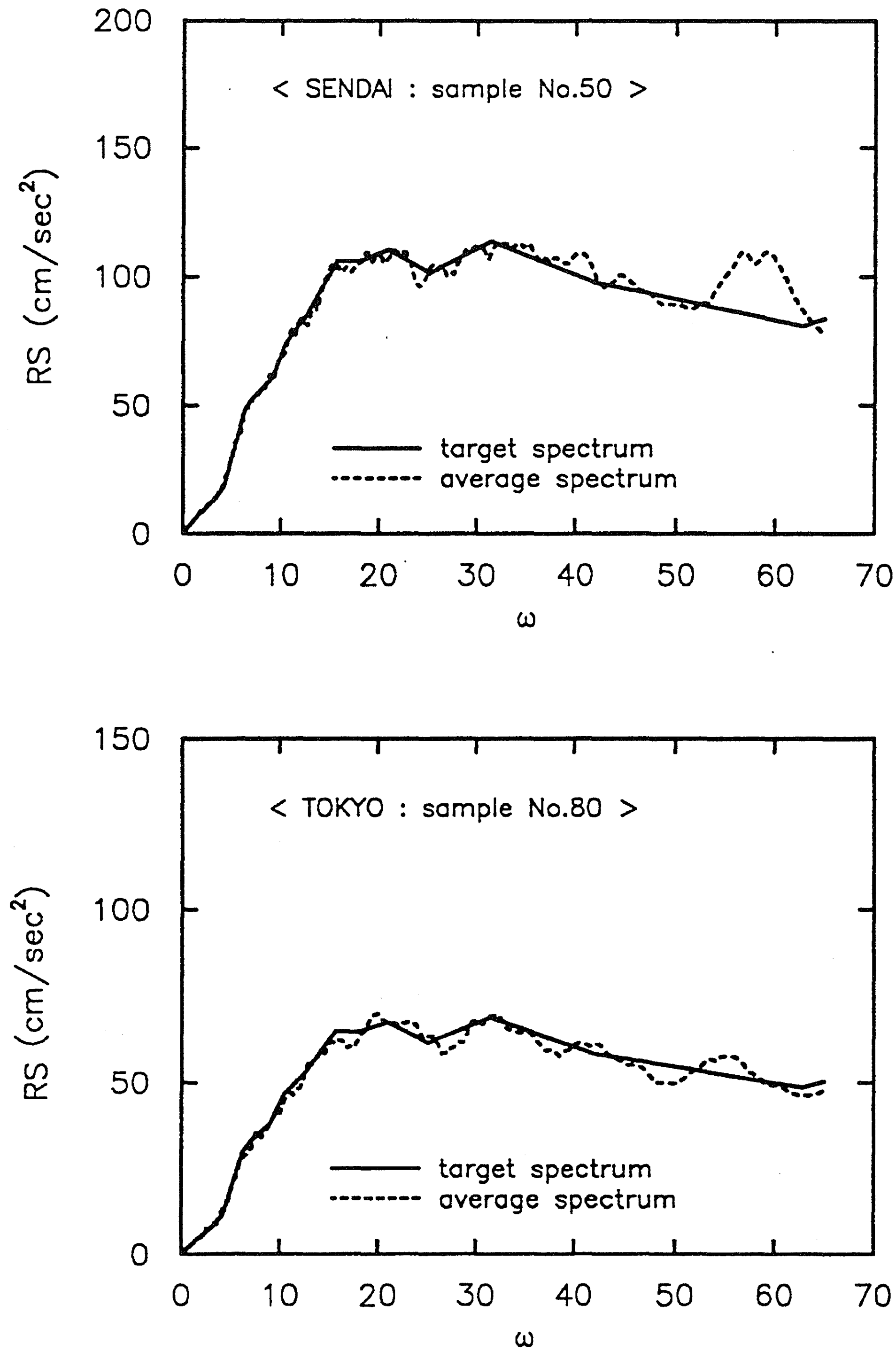
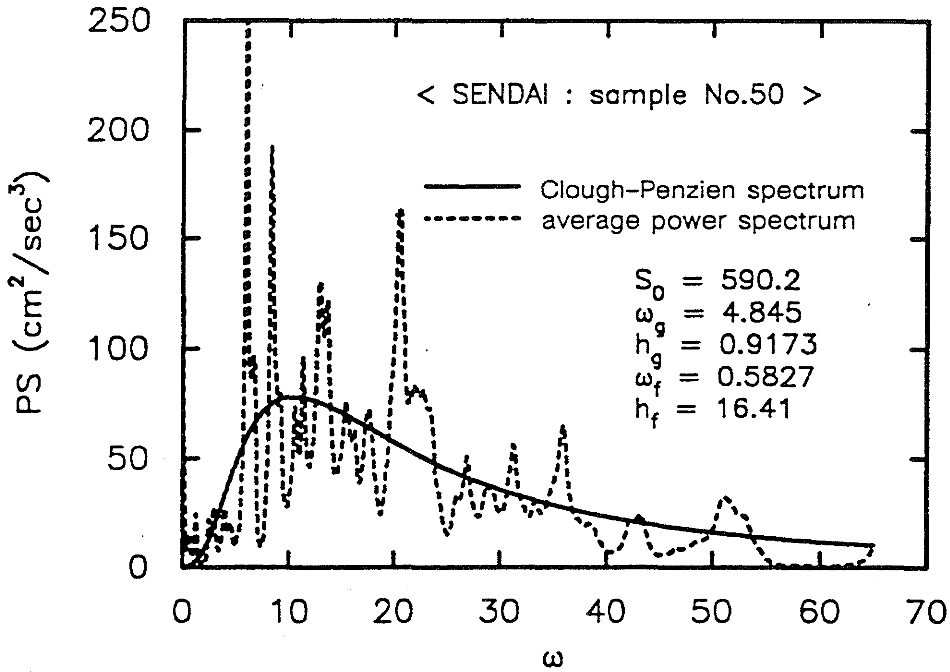


Figure 2.10 Comparison between target response spectrum and average response spectrum



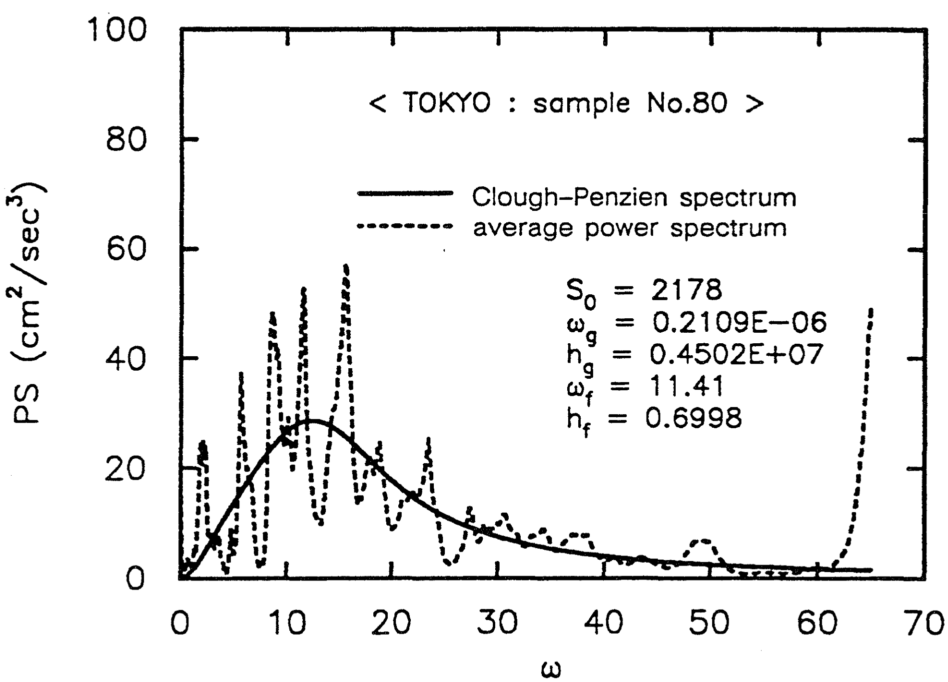


Figure 2.11 Comparison of average power spectra and the identified Clough–Penzien spectra

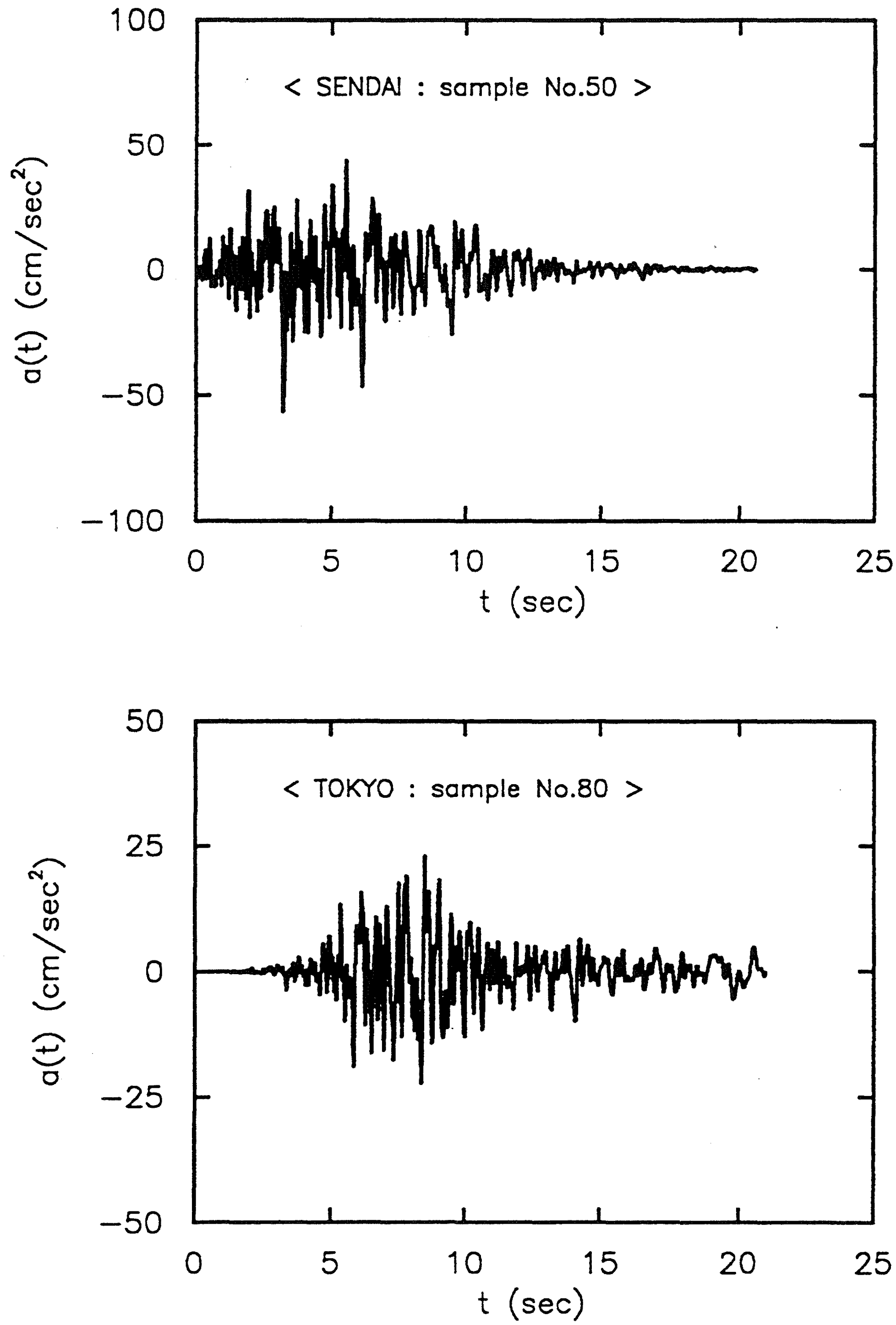
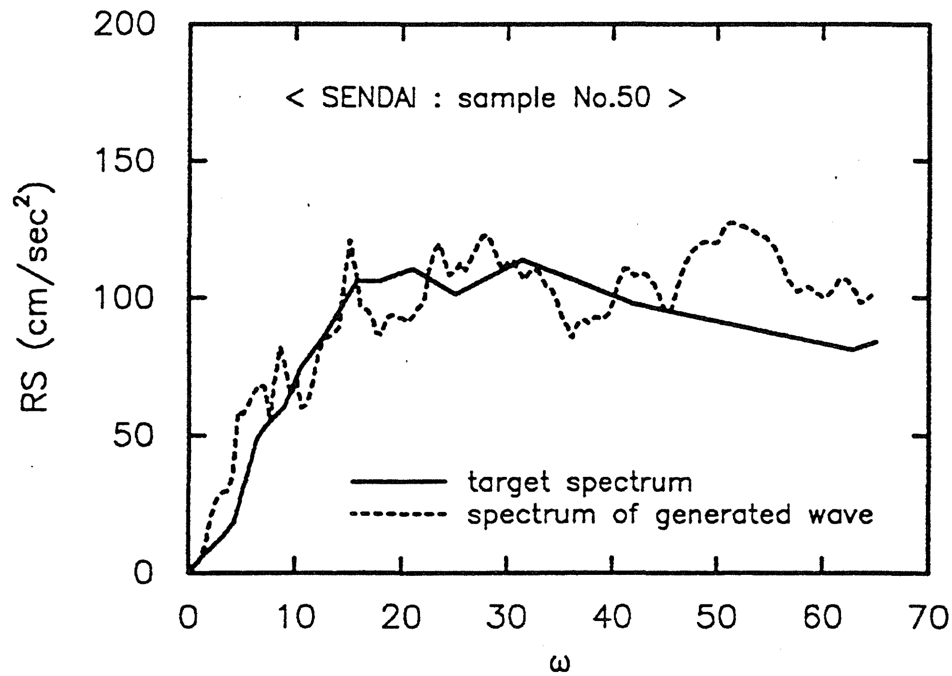


Figure 2.12 Sample earthquake ground motions



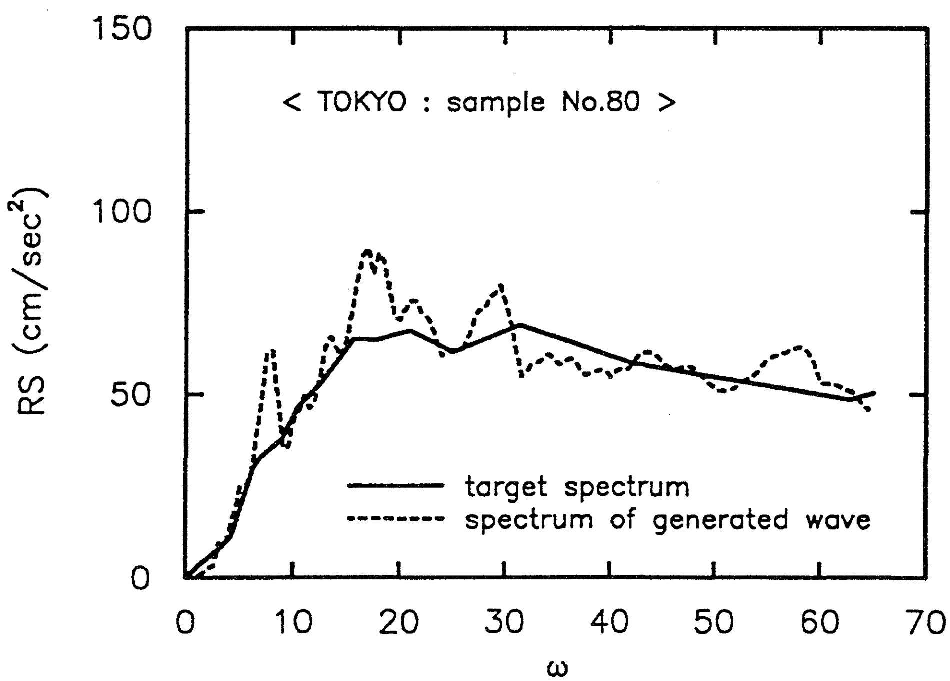


Figure 2.13 Comparison between target spectrum and spectrum of generated sample wave

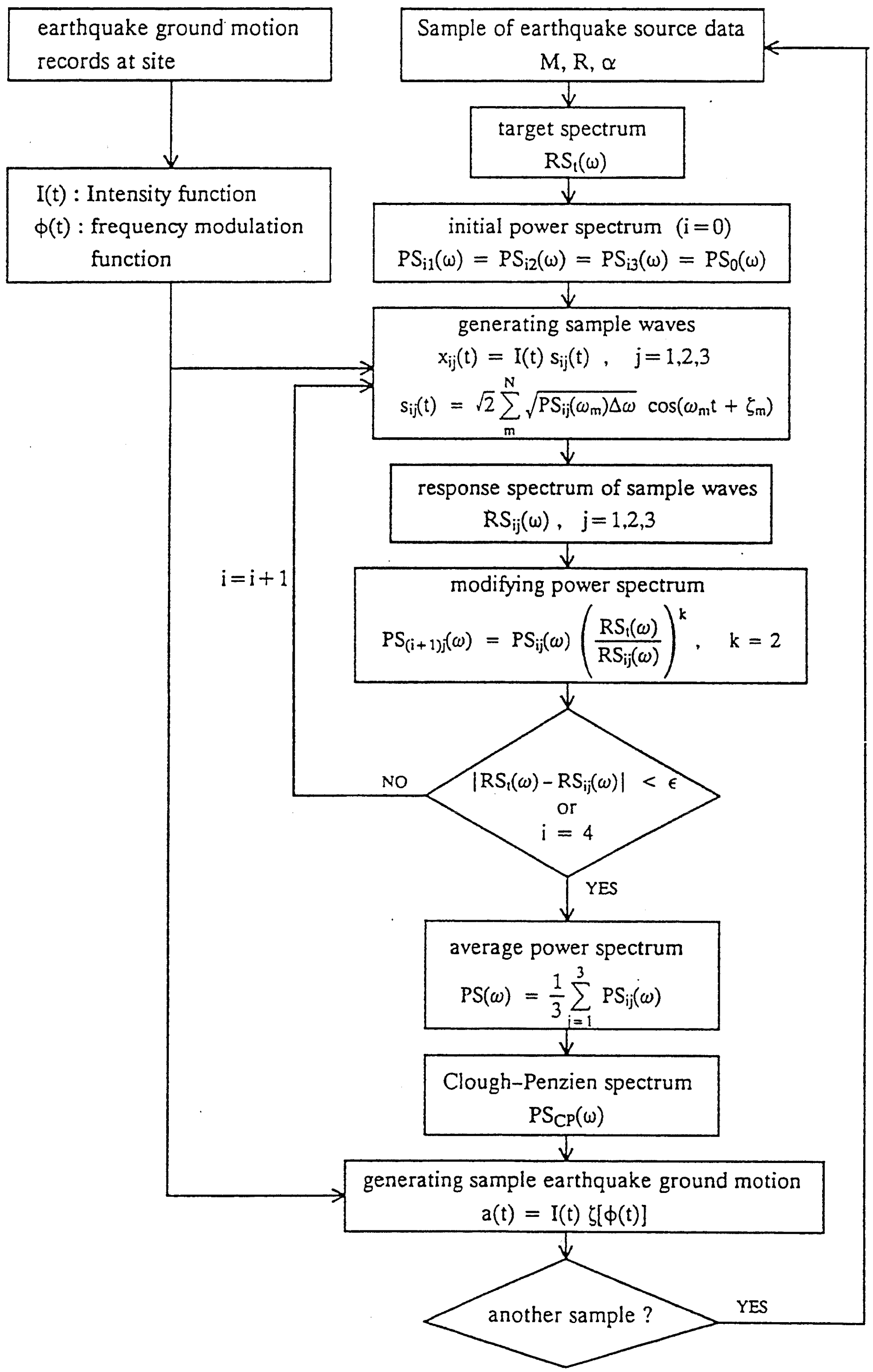


Figure 2.14 Flowchart for generating earthquake ground motions

CHAPTER 3

EARTHQUAKE–RESISTANT DESIGN OF REINFORCED CONCRETE BUILDINGS

3.1 Introduction

The Architectural Institute of Japan (AIJ) published " Design Guideline for Earthquake Resistant Reinforced Concrete Buildings Based on Ultimate Strength Concept " in 1990. This guideline provides seismic design procedures for reinforced concrete moment–resisting frame and wall–frame structures. The basic design concept outlined in this guideline is to design a building to form a specified failure mechanism (a strong–column–weak–beam failure mechanism for a moment–resisting frame) in order to control expected yield deformations during a strong earthquake. This guideline was then included as part of the seismic design procedures described in " Ultimate Strength Design Guidelines for Reinforced Concrete Buildings " which was prepared by the Japan PRESSS (Precast Seismic Structural Systems) Guideline Drafting Working Group and presented at the "Thomas Pauley Symposium" in 1993. A copy of this guideline is presented in Appendix A. Although refinement of the guideline is still in progress, it is adopted in this study as the basis for designing model frames.

The new guideline requires the designer to consider two limit states in the design of reinforced concrete buildings. For medium intensity earthquakes, a proposed building design must satisfy certain serviceability limit state criteria ; for high intensity earthquakes, a proposed building design must satisfy criteria based on an ultimate limit state. For buildings greater than or equal to 31m in height, a static nonlinear analysis is required to examine the performance of the building under each limit state. However, a linear elastic analysis and a simplified limit analysis may be used for buildings less than 31m in height. The earthquake–resistant design procedures are briefly summarized in the following sections.

3.2 Earthquake Resistant Design Procedures

3.2.1 General Requirements

The design guideline is applicable to reinforced concrete buildings (with or without structural walls) which are less than 60m in height. The distribution of mass, stiffness, and strength in the building must be uniform. Structural regularity is judged based on a stiffness ratio, R_s, and an eccentricity ratio, R_e, specified in the 1981 code, " Building Standard Law Enforcement Order. " The stiffness ratio, R_s, is defined as

$$R_s = r_s / \overline{r_s} \tag{3.1}$$

where r_s is the reciprocal of the story drift ratio and $\overline{r_s}$ is the arithmetic mean of all r_s values. It is required that R_s be equal to or greater than 0.6. The eccentricity ratio, R_e, of a story is calculated

$$R_e = e / r_e \tag{3.2}$$

where e is the eccentricity and r_e is the elastic radius of gyration. It is required that R_e be less than 0.15.

The building is designed to form a specified yield mechanism. A strong–column–weak–beam (SCWB) failure mechanism is considered desirable for a moment–resisting frame. This failure mechanism, illustrated in Figure 3.1.(a), results in a uniform story drift over the structural height. Also, the simultaneous yielding at all beam ends produces stable energy dissipation and ductility during a strong earthquake. Unlike the SCWB mechanism, a partial yielding mechanism, resulting in the formation of yield hinges in only a limited number of stories, leads to a concentration of large story drifts in particular stories (see Figure 3.1.(b)).

The yielding hinges in the planned mechanism are designed first. Regions other than the specified yield hinges must be designed not to yield. The design of these non–yielding regions must take into consideration factors which might increase the member design forces such as dynamic effects and the combined effects of bi–directional response.

3.2.2 Design Earthquake Forces

The design of structures for earthquake motions is carried out based on an equivalent static analysis with the design story shear force expressed as

$$Q_i = C_i W_i \tag{3.3}$$

$$C_i = Z R_t A_i C_B \tag{3.4}$$

where C_i = seismic story shear coefficient at story i

W_i = total dead load and live load above story i

Z = seismic zone factor determined from the seismic zone map in Figure 3.2

R_t = vibration characteristic factor which accounts for the dynamic interaction between the structure and the supporting soil

A_i = vertical distribution of story shear

C_B = standard base shear coefficient

The coefficient R_t is expressed as

$$\begin{aligned} R_t &= 1.0 && \text{for } T < T_c \\ &= 1.0 - 0.2\,(T / T_c - 1.0)^2 && \text{for } T_c < T < 2T_c \\ &= 1.6\,(T_c / T) && \text{for } 2T_c < T \end{aligned} \tag{3.5}$$

where T_c is a critical period of subsoil and T is an estimate of the fundamental period of the building. The subsoil period T_c is defined as

$$\begin{aligned} Tc &= 0.4 \text{ sec} && \text{for Type 1 soil (rock, stiff sand or gravel)} \\ &= 0.6 \text{ sec} && \text{for Type 2 soil (other than Type 1 and Type 3)} \\ &= 0.8 \text{ sec} && \text{for Type 3 soil (alluvium consisting of soft soil)} \end{aligned} \tag{3.6}$$

The fundamental period, T, of the building may be calculated using

$$T = 0.02\, h \tag{3.7}$$

where h is the building height (m). Figure 3.3 shows the value of R_t as a function of structural period for different soil types. The vertical distribution factor, A_i, is given by the following expression (see Figure 3.4) :

$$A_i = 1.0 + \left(\frac{1}{\sqrt{a_i}} - a_i \right) \frac{2T}{1 + 3T} \tag{3.8}$$

$$a_i = W_i / W_1 \tag{3.9}$$

Using Equation (3.3), the base shear coefficient, C_B, for each structural period can be calculated, and a "design spectrum" such as the one shown in Figure 3.5 can be plotted. The two spectra in Figure 3.5 developed in accordance with the above procedure are based on a reinforced concrete moment–resisting frame on a stiff soil foundation in a high seismicity zone. For this case, Tc = 0.4, Z = 1.0, $C_B = 0.2$ (Serviceability Limit State), and $C_B = 0.3$ (Ultimate Limit State). On the same figure, the design spectrum calculated using the formula in the 1991 Uniform Building Code (UBC) is presented. The same structural type and soil conditions are assumed, and the site is assumed to be located in one of the high seismicity zones (Zone 4) specified in the UBC. The comparison of the two curves indicates that the design shear coefficient for the building in Japan is about two times the shear coefficient for the building in the United States. However, this fact doesn't necessary imply that Japanese buildings designed according to the new guideline have higher seismic safety than American buildings. The seismic environment and design philosophy are different in each country.

3.2.3 Earthquake Design Criteria

The performance of the designed building must be examined under both the serviceability limit state and the ultimate limit state. For moment–resisting frames, the standard base shear coefficient, C_B, for the design shear force is 0.2 for the serviceability limit state and 0.3 for the ultimate limit state. Performance criteria at each limit state vary with the procedure used for the structural analysis. The criteria applicable to each of the acceptable analysis procedures are described below.

1) Performance criteria for linear elastic analysis

The guideline permits the use of linear elastic analysis for buildings less than 31m in height. The performance criteria for this procedure are as follows :

- At the serviceability limit state, the story drift ratio at any story must not exceed 1/600 for a moment–resisting frame.
- The lateral load resisting capacity must be examined by an approximate limit analysis (such as the virtual work method). The calculated story shear capacity must exceed the required shear force for the ultimate limit state.

2) Performance criteria for static nonlinear analysis

For buildings greater than or equal to 31m in height, the guideline requires a nonlinear incremental static analysis which properly takes into consideration the inelastic behavior characteristics of the structural members. The performance criteria for moment–resisting frames are as follows :

- No member is allowed to yield at the serviceability limit state, and the story drift ratio at any story must not exceed 1/200.
- At a story drift ratio of 1/100, which is called the design limit deformation, Ru1, the story shear at any story must be more than 0.9 times the design shear for the ultimate limit state. At a story drift ratio of 1/50, which is called the design proof deformation, Ru2, the story shear must be more than the design shear for the ultimate limit state.

These requirements are shown schematically in Figure 3.6.

3.3 Design of Reinforced Concrete Moment–Resisting Frames

Two different reinforced concrete office buildings, 7–story and 12–story moment–resisting frames, are designed according to the guideline. The plan and elevation views of the structures are shown in Figures 3.7 to 3.9, and the member sizes are listed in Table 3.1. The concrete is assumed to have a nominal strength $F_c = 360$ kg/cm^2, and the reinforcement consists of Grade SD395 steel reinforcing bars (yield strength $f_y = 4{,}000$ kg/cm^2).

Two different locations in Japan are considered – Sendai and Tokyo. Since the zone factor is the same for both sites(Z=1.0), the design shear forces are identical for both sites. These design forces are listed in Table 3.2. The critical period of the subsoil, T_c, is 0.6sec which corresponds to stiff soil conditions. The same value of T_c is used for both sites. A strong–column–weak–beam failure mechanism is adopted for this design. Plastic hinges are assumed to form at the beam ends and at the base of the first story columns. These yielding members are designed to comply with the requirements for the ultimate limit state. The non–yielding members (i.e., columns) are designed to prevent yielding. The procedures used to design the structural members are described in the following section.

3.3.1 Structural Member Design

The AIJ design guideline provides minimum performance requirements for the design of buildings and does not describe the methods to be used in designing structural members. In this study, the simple design procedures which are commonly used in Japan for the design of moment–resisting frames are adopted. The procedures used in the design of yielding members and the procedures used in the design of non–yielding members are described below.

1) Design of yielding members

Figure 3.10 provides a schematic representation of the method used to determine the design flexural strength of beam members. Assuming that all beams in the i–th story have the same yield moment, M_{yi}, the equilibrium equation can be written approximately as

$$6M_{yi} = Q_{i-1}\, h_{i-1}/2 + Q_i\, h_i/2 \qquad (3.10)$$

where Qi is the design story shear at the ultimate limit state (base shear coefficient C_B=0.3) and h_i is the height of the i–th story. The yield moment, Myi, obtained from the above equation is then modified to determine the moment at the face of the column, M_{yi}', based on the size of the beam–column joint (see Figure 3.10). The amount and layout of beam reinforcement is then determined so that the resulting flexural strength of the member is greater than M_{yi}'.

The flexural strength of beam is obtained by

$$M_{yb} = 0.9\, a_t\, \sigma_y\, d \tag{3.11}$$

where a_t = area of reinforcement steel in the tension side

σ_y = realistic yield strength of steel

d = effective depth of beam section

Although the actual yield strength of steel, σ_y, is not known, a study of statistical data on yield strength indicates that it is usually higher than the nominal strength, f_y. As is commonly done in Japan, the realistic yield strength, σ_y, is assumed to be $1.1\, f_y$ (4,400 kg/cm).

As noted earlier, a yield hinge is assumed to form at the base of each first story column. However, a large yield deformation at the first story column may significantly affect the overall structural behavior. Furthermore, columns are not considered to be as ductile as beams because of the large axial forces. For these reasons, the yield deformation at the first story column must be restricted. The flexural strength of the bottom of the first story column can be obtained using a simple procedure such as a virtual work method. The design equation for column members is shown in the next section.

2) Design of non–yielding members

All columns (except those in the first story) must be designed with sufficient shear and flexural strength to avoid yielding. Also, beams must be designed so that yielding will occur only at the specified yield hinges. The procedure of shear strength design is not described here, because flexural failure is dominant for moment–resisting frame members. The design flexural strength of columns (excluding first story columns) is determined using the empirical formula

$$\sum M_c = c \sum M_b \tag{3.12}$$

where $\sum M_c$ = sum of moments, at the center of the beam–column joint, corresponding to the design flexural strengths of the columns

$\sum M_b$ = sum of moments, at the center of the beam–column joint, corresponding to the design flexural strengths of the beams

c = a factor of safety used to prevent column yielding

As is commonly done in Japan, the factor c is selected as 1.3 in this study. It can be noted that the ACI building code (Building Code Requirements for Reinforced Concrete, ACI–318–89) recommends using the same relation with the factor c equal to 1.2. The amount and layout of reinforcement steel in the column is then determined so that its flexural strength is greater than M_c.

The flexural strength of a column is obtained by

$$M_{yc} = 0.8 a_t \sigma_y D + 0.5 N D \left(1 - \frac{N}{F_c b D}\right) \tag{3.13}$$

where D = the depth of section

b = the width of section

N = axial force due to live and dead loads

The design member strength might differ from member to member. It is common practice to adjust the design flexural strength to take into consideration (a) available bar sizes, (b) use of common bar sizes in a story, and (c) use of continuous reinforcing bars through a beam–column joint. According to the AIJ design guideline, the amount of moment adjustment is limited to 20% for moment–resisting frames. The reinforcement steel used in each member (both yielding and non–yielding) is listed in Table 3.3.

3.3.2 Static Nonlinear Analysis of Frame

To evaluate the performance of the designed frame when subjected to loads corresponding to the serviceability limit state and ultimate limit state, a static nonlinear analysis is carried out. (Since the 7–story building is less than 31m in height, such an analysis would not be required by the guide–

eline.) In the analytical model of the building, the columns and the beams are represented by line elements having nonlinear rotational springs at their ends (see Figure 3.11). The force–deformation relation of the rotational springs is represented by a modified Takeda model which represents the typical moment–rotation relation of reinforced concrete members (see Figure 3.12). Other assumptions in the analysis are follows :

- The cracking moment, M_c, is assumed to be equal to one–third of the yield moment, M_y.
- The stiffness reduction factor, α_y, which corresponds to the ratio of the secant stiffness at the yield point to the initial elastic stiffness, is assumed to be 0.3.
- The stiffness after yielding is assumed to be 0.001 times the initial elastic stiffness.

The frame analysis program FRAME–D, developed at Tohoku University, is used to carry out the analysis. Story shears and story drift values for each directional frame of the two buildings are shown in Figures 3.13 and 3.14. In these figures, the solid lines represent the variation of story shear and story drift of each floor as the applied loads were increased. The vertical distribution of the loads was assumed to be equal to that defined by Equation (3.8). The dashed lines in the figures represent the values of story shear and story drift at the point in the analysis at which one story drift has reached R_{u1} and R_{u2}, respectively. Table 3.4 provides a comparison of these results with the performance criteria outlined in the AIJ guideline. The comparison confirms that the performance criteria are met. Specifically, (a) the maximum drift ratio at the load for the serviceability limit state is less than 1/200, (b) the base shear at the design limit deformation, R_{u1}, is more than 0.9 times the design shear, and (c) the base shear at the design proof deformation, R_{u2}, is greater than the design shear.

Figures 3.15 and 3.16 show the hinge locations and the corresponding member ductility factors at each limit deformation, R_{u1} and R_{u2}. The member ductility factor is defined as the ratio of the maximum rotational displacement at the end of a member to the yield rotational displacement. In the 7–story frame, some columns in the middle story exceeded yield level at the design proof deformation, R_{u1}. However, these exceedances are quite small, and are judged to be acceptable. In all frames, the planned strong–column–weak–beam failure mechanism was almost achieved.

The AIJ design guideline does not provide any specific criteria about limits on ductility factors. Such limits are not provided because ductility factors obtained from a static nonlinear analysis do not account for the dynamic effects. However, it is recognized that some limits on member ductility factors are desirable. A committee formed to propose guidelines for the design of reinforced concrete buildings (of which the first author is a member) has suggested the following limitations :

- At the design limit deformation, R_{u1}, the ductility factor of any beam should be less than 2.0. The ductility factor at the bottom of a first story column should be less than 1.5.
- At the design proof deformation, R_{u2}, the ductility factor of any beam should be less than 4.0. The ductility factor at the bottom of a first story column should be less than 3.0.

These limitations are almost satisfied in the designed frames.

Table 3.1 Dimensions of structural members

Unit : cm

7-story	column	beam(X-frame)	beam(Y-frame)
Roof	———	550 × 900	550 × 1050
2–7	850 × 850	550 × 900	550 × 1050
1	850 × 850	———	———

12-story	column	beam(X-frame)	beam(Y-frame)
Roof	———	550 × 900	550 × 1050
2–12	900 × 900	550 × 900	550 × 1050
1	900 × 900	———	———

Table 3.2 Design story shear forces

7-story	h_i (cm)	W_i (ton)	α_i	A_i	C_i	Q_i (ton)
7	400	1835*	0.167	1.862	0.559	1025.17
6	400	1513	0.305	1.533	0.460	1539.42
5	400	1513	0.443	1.354	0.406	1975.09
4	400	1513	0.581	1.233	0.307	2357.53
3	400	1513	0.719	1.141	0.342	2698.58
2	400	1513	0.856	1.066	0.320	3005.13
1	500	1575	1.000	1.000	0.300	3292.50

12-story	h_i (cm)	W_i (ton)	α_i	A_i	C_i	Q_i (ton)
12	400	1835*	0.099	2.425	0.727	1334.82
11	400	1513	0.181	1.959	0.588	1967.86
10	400	1513	0.262	1.717	0.515	2503.61
9	400	1513	0.344	1.557	0.467	2976.78
8	400	1513	0.425	1.438	0.431	3403.04
7	400	1513	0.507	1.344	0.403	3791.26
6	400	1513	0.589	1.267	0.380	4147.18
5	400	1513	0.670	1.200	0.360	4474.18
4	400	1513	0.752	1.142	0.343	4777.08
3	400	1513	0.833	1.091	0.327	5056.32
2	400	1513	0.915	1.044	0.313	5314.35
1	500	1575	1.000	1.000	0.300	5562.00

* This weight includes the assumed weight of roof structures

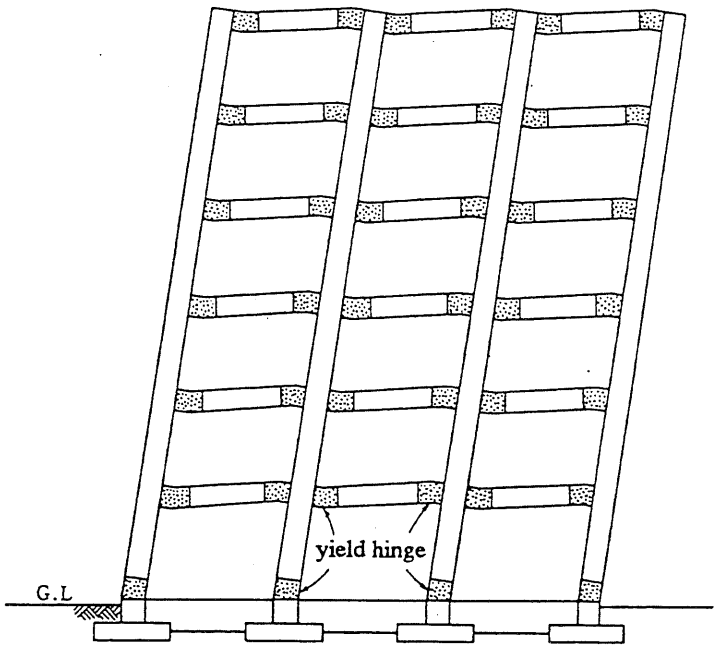
Table 3.3 Design of reinforcement steel

<table>
<tr><th>7–story</th><th>column</th><th>beam(X–frame)</th><th>beam(Y–frame)</th></tr>
<tr><td>R</td><td>——</td><td rowspan="2">4–D29</td><td rowspan="2">4–D29</td></tr>
<tr><td>6, 7</td><td rowspan="4">10–D38</td></tr>
<tr><td>4, 5</td><td>5–D35</td><td>5–D35</td></tr>
<tr><td>2, 3</td><td>6–D38</td><td>6–D38</td></tr>
<tr><td>1 top</td><td rowspan="2">——</td><td rowspan="2">——</td></tr>
<tr><td>1 bottom</td><td>12–D38</td></tr>
</table>

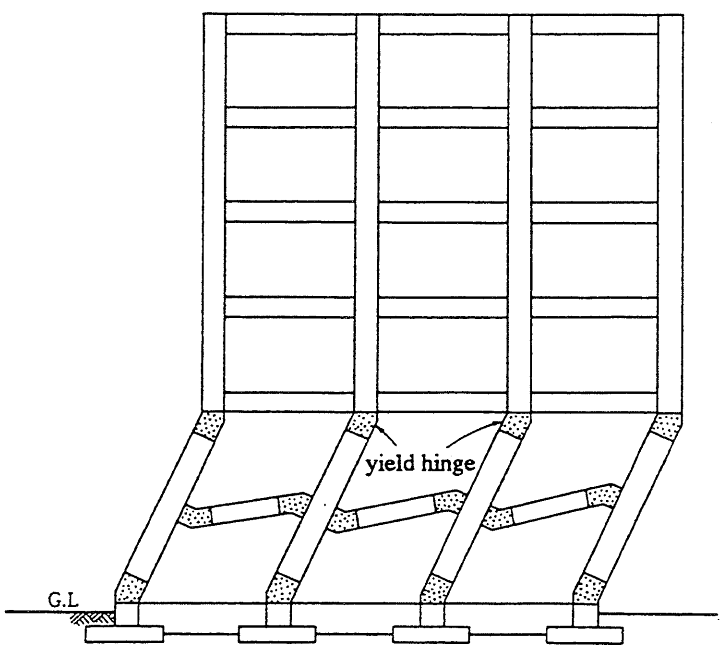
<table>
<tr><th>12–story</th><th>column</th><th>beam(X–frame)</th><th>beam(Y–frame)</th></tr>
<tr><td>R</td><td>——</td><td rowspan="2">5–D32</td><td rowspan="2">5–D32</td></tr>
<tr><td>11, 12</td><td rowspan="2">12–D38</td></tr>
<tr><td>8, 9, 10</td><td>5–D41</td><td>5–D41</td></tr>
<tr><td>5, 6, 7</td><td rowspan="3">12–D41</td><td>6–D41</td><td>6–D41</td></tr>
<tr><td>2, 3, 4</td><td>7–D41</td><td>7–D41</td></tr>
<tr><td>1 top</td><td rowspan="2">——</td><td rowspan="2">——</td></tr>
<tr><td>1 bottom</td><td>16–D41</td></tr>
</table>

Table 3.4 Check of design criteria

<table>
<tr><th colspan="2" rowspan="2"></th><th>Serviceability limit state</th><th colspan="2">Ultimate limit state</th></tr>
<tr><th>$C_B = 0.2$</th><th>$R_{u1} = 1/100$</th><th>$R_{u2} = 1/50$</th></tr>
<tr><td colspan="2">Criteria</td><td>$d_{max} < 1/200$</td><td>$C_B > 0.27$</td><td>$C_B > 0.30$</td></tr>
<tr><td rowspan="2">7 story</td><td>X</td><td>$d_{max} = 1/339$</td><td>$C_B = 0.371$</td><td>$C_B = 0.381$</td></tr>
<tr><td>Y</td><td>$d_{max} = 1/337$</td><td>$C_B = 0.355$</td><td>$C_B = 0.364$</td></tr>
<tr><td rowspan="2">12 story</td><td>X</td><td>$d_{max} = 1/354$</td><td>$C_B = 0.308$</td><td>$C_B = 0.321$</td></tr>
<tr><td>Y</td><td>$d_{max} = 1/266$</td><td>$C_B = 0.281$</td><td>$C_B = 0.304$</td></tr>
</table>



(a) Strong–column–weak–beam failure mechanism



(b) Partial yielding mechanism

Figure 3.1 Failure mechanisms of frame (adopted from Reference 2)

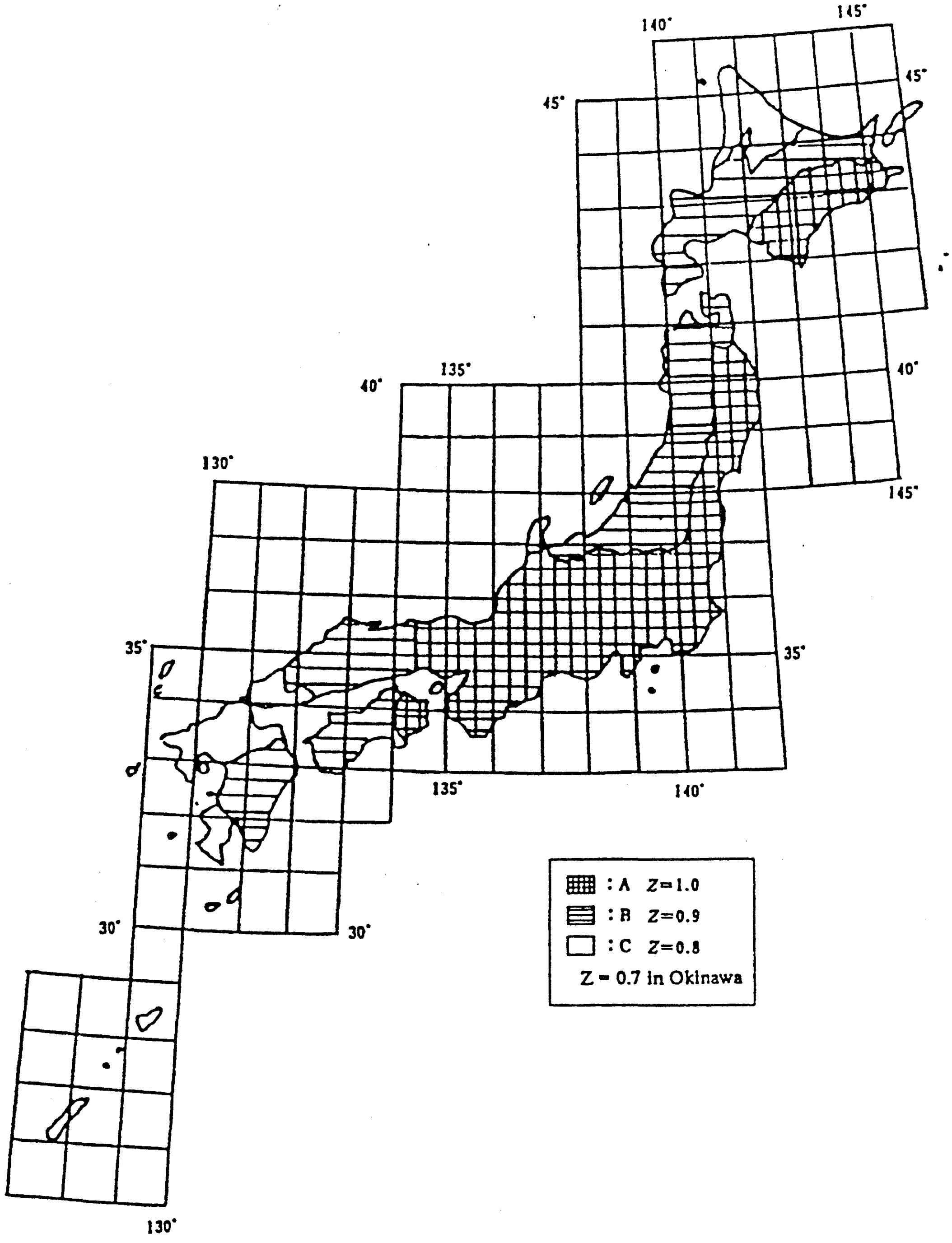


Figure 3.2 Seismic zone factor Z (adopted from Reference 17)

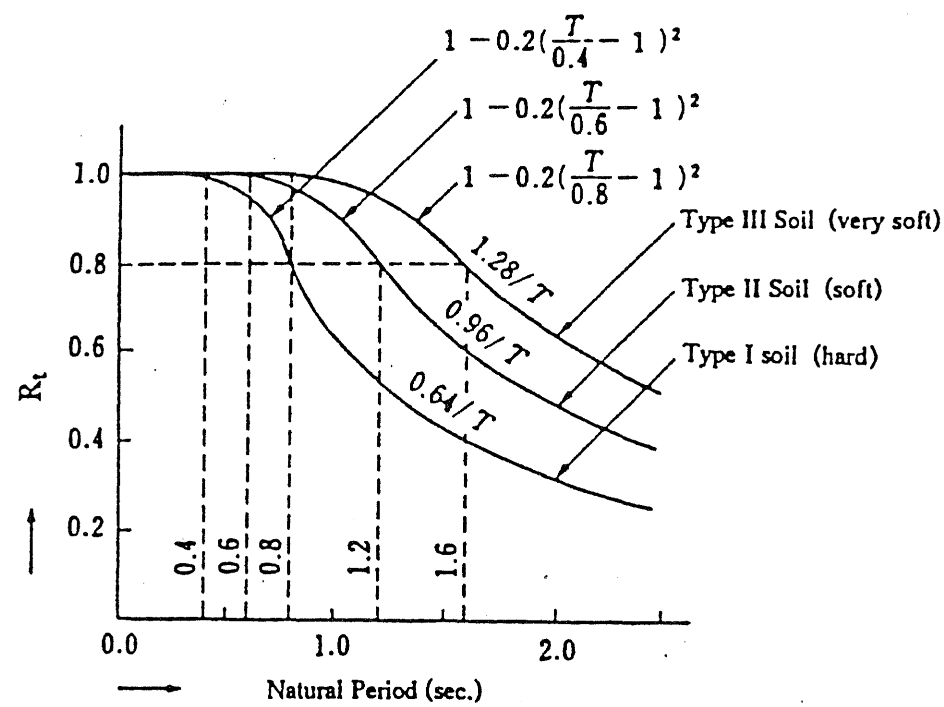


Figure 3.3 Vibration characteristic factor R_t (adopted from Reference 17)

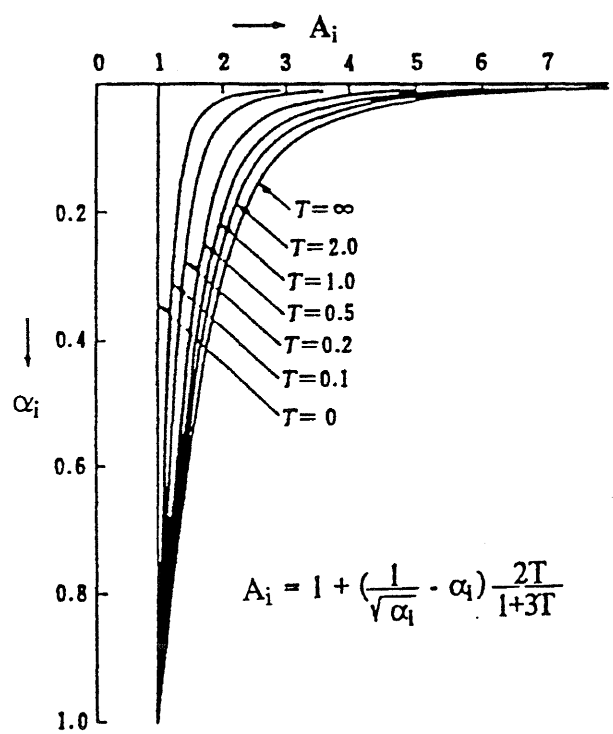


Figure 3.4 Vertical distribution of story shear A_i (adopted from Reference 17)

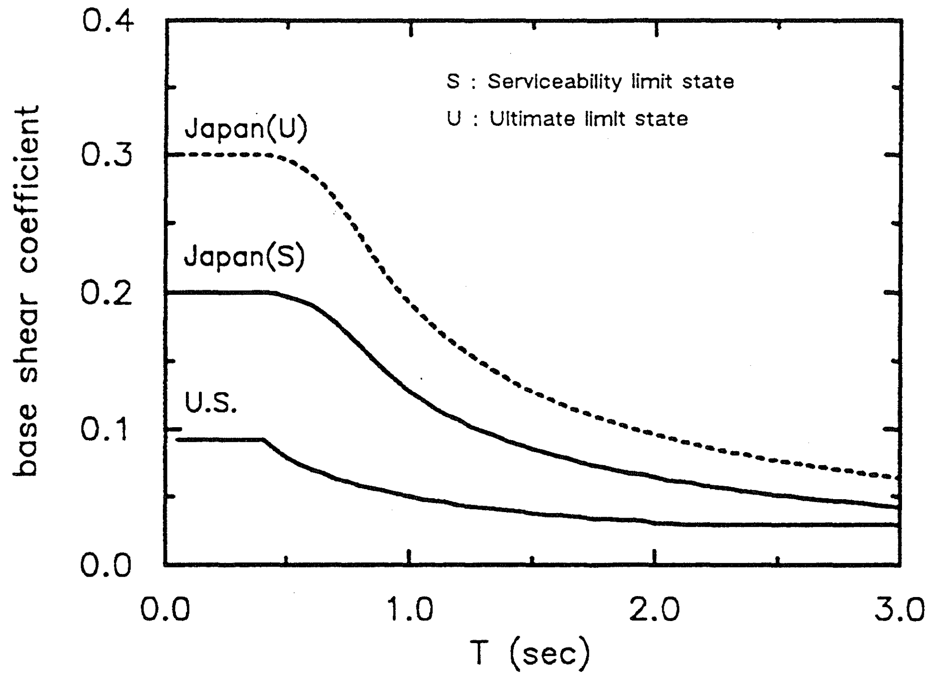


Figure 3.5 Design spectra of Japan and the United States

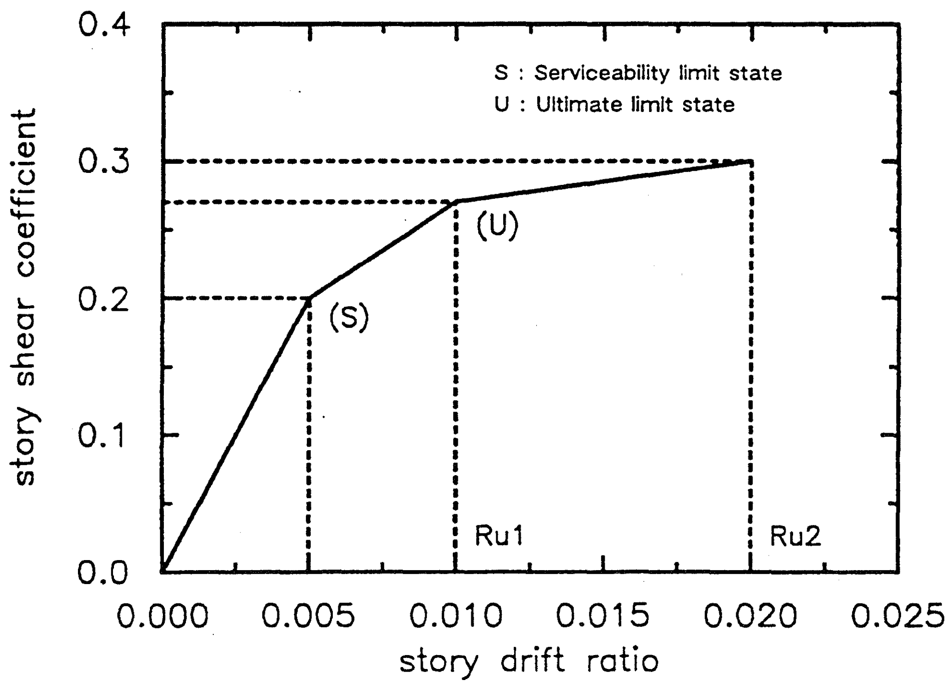


Figure 3.6 Performance criteria in the guideline

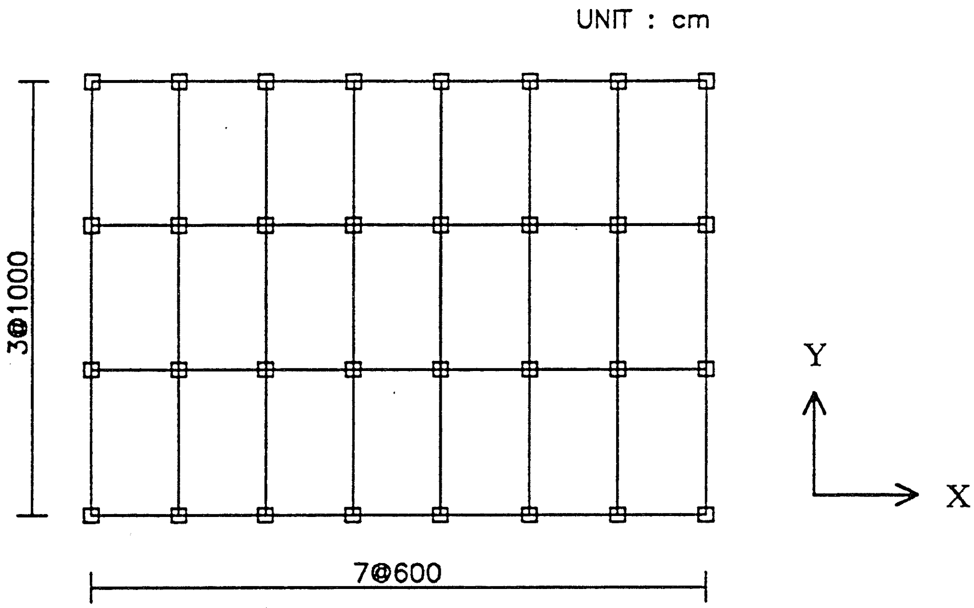


Figure 3.7 Plan view of 7–story and 12–story buildings

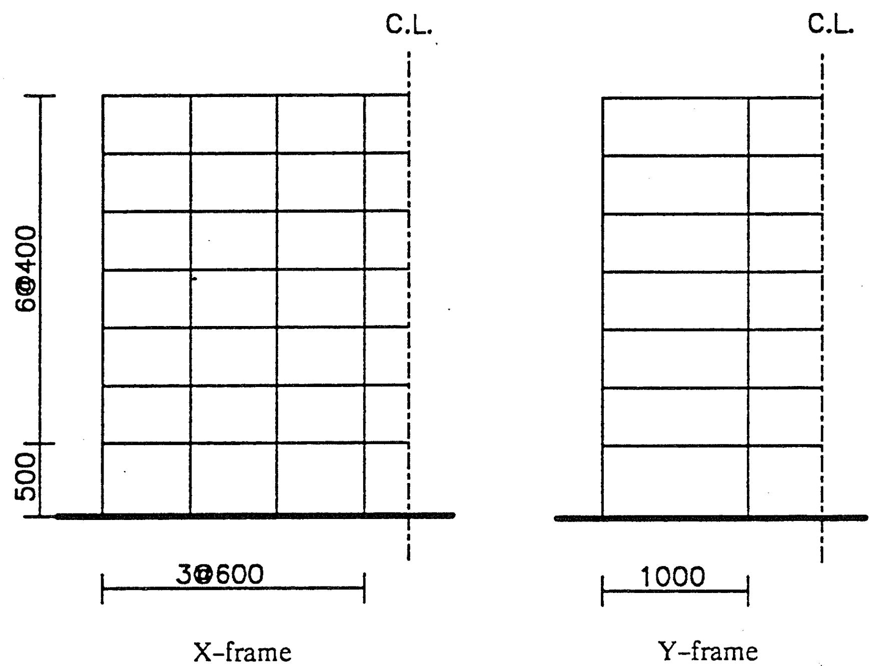


Figure 3.8 Elevation view (7–story building)

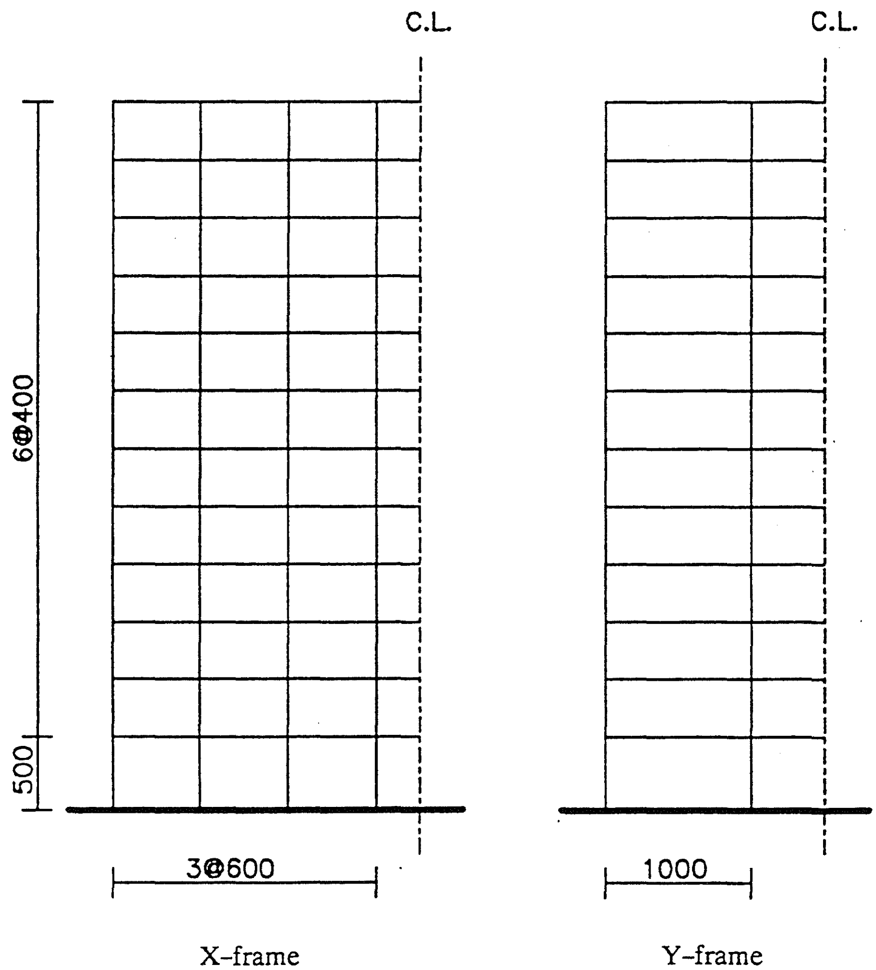
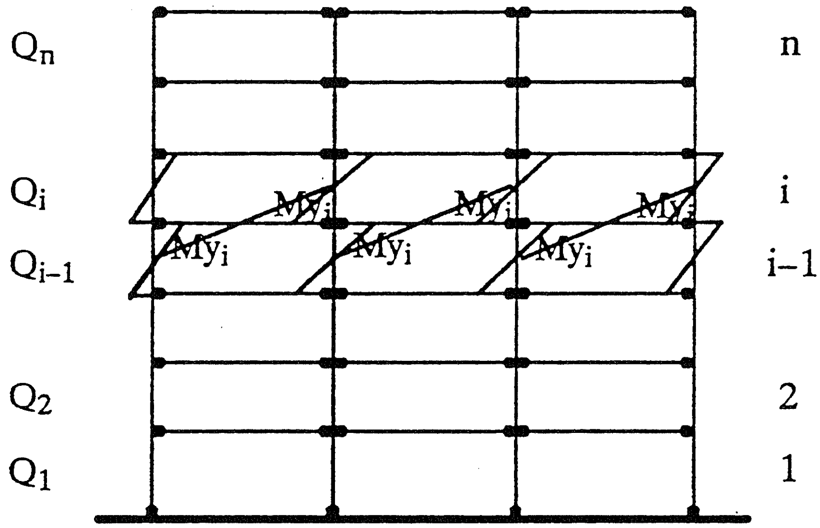
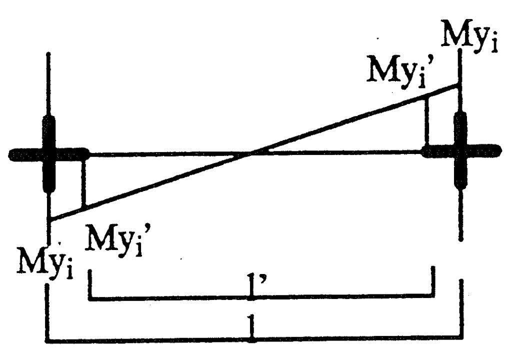


Figure 3.9 Elevation view (12–story building)



$$6M_{yi} = Q_{i-1}\,h_{i-1}/2 + Q_i\,h_i/2$$



$$My_i' = \frac{l'}{l}\,My_i$$

Figure 3.10 Schematic representation of the design process for yielding members

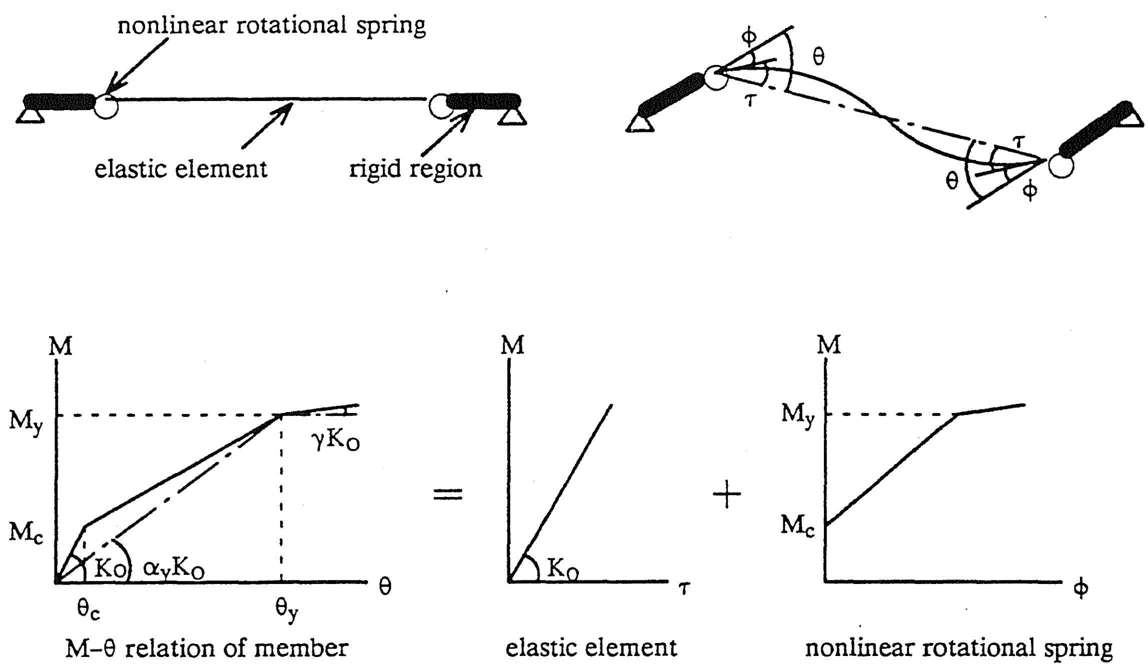


Figure 3.11 Model used for inelastic members

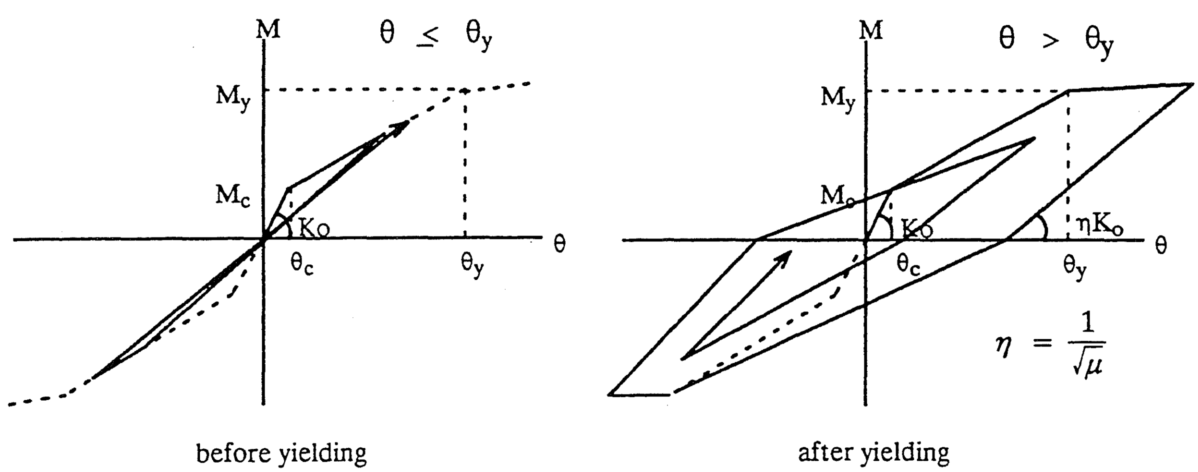


Figure 3.12 Modified Takeda model

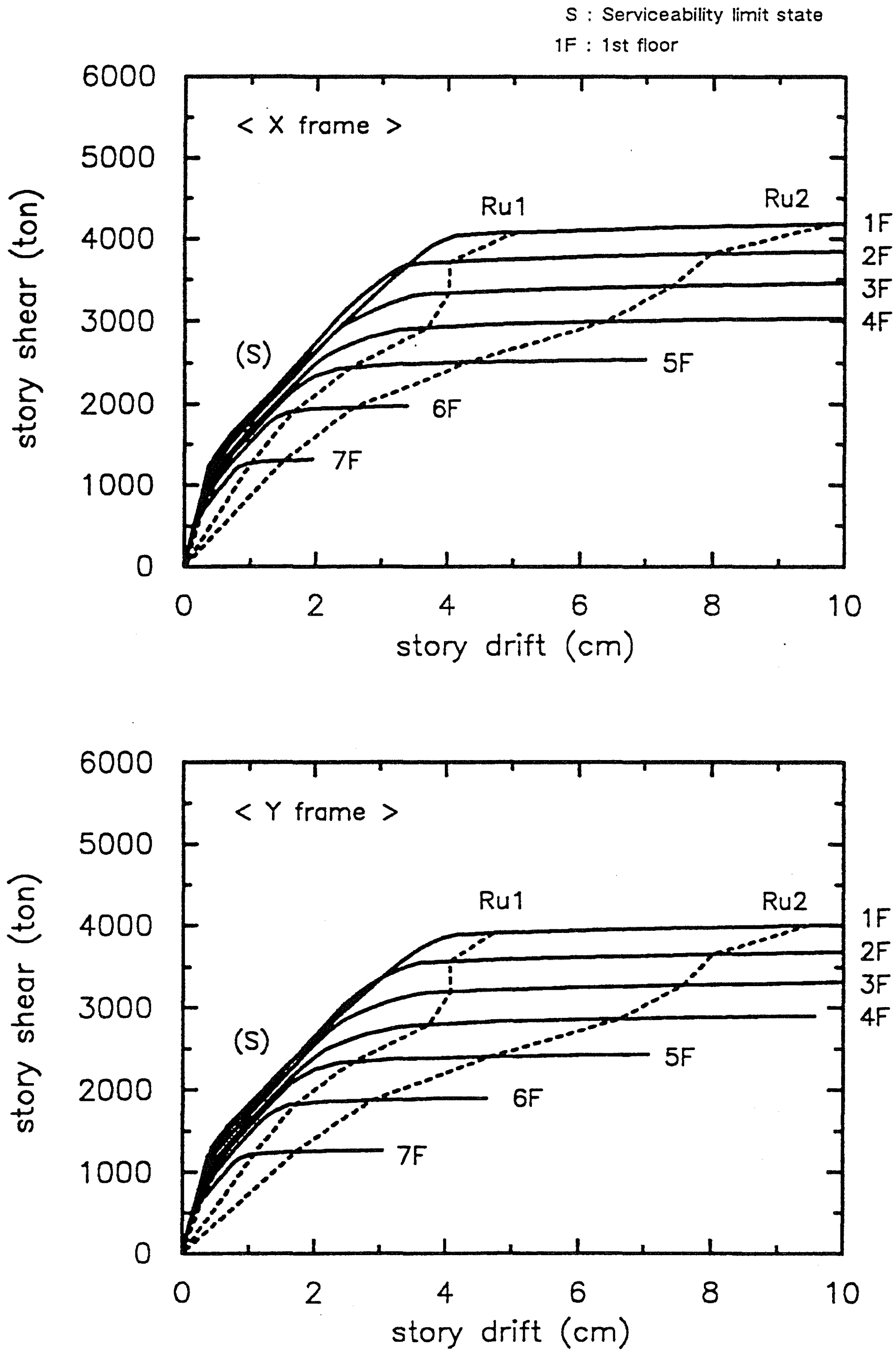


Figure 3.13 Relation between story shear force and story drift (7–story building)

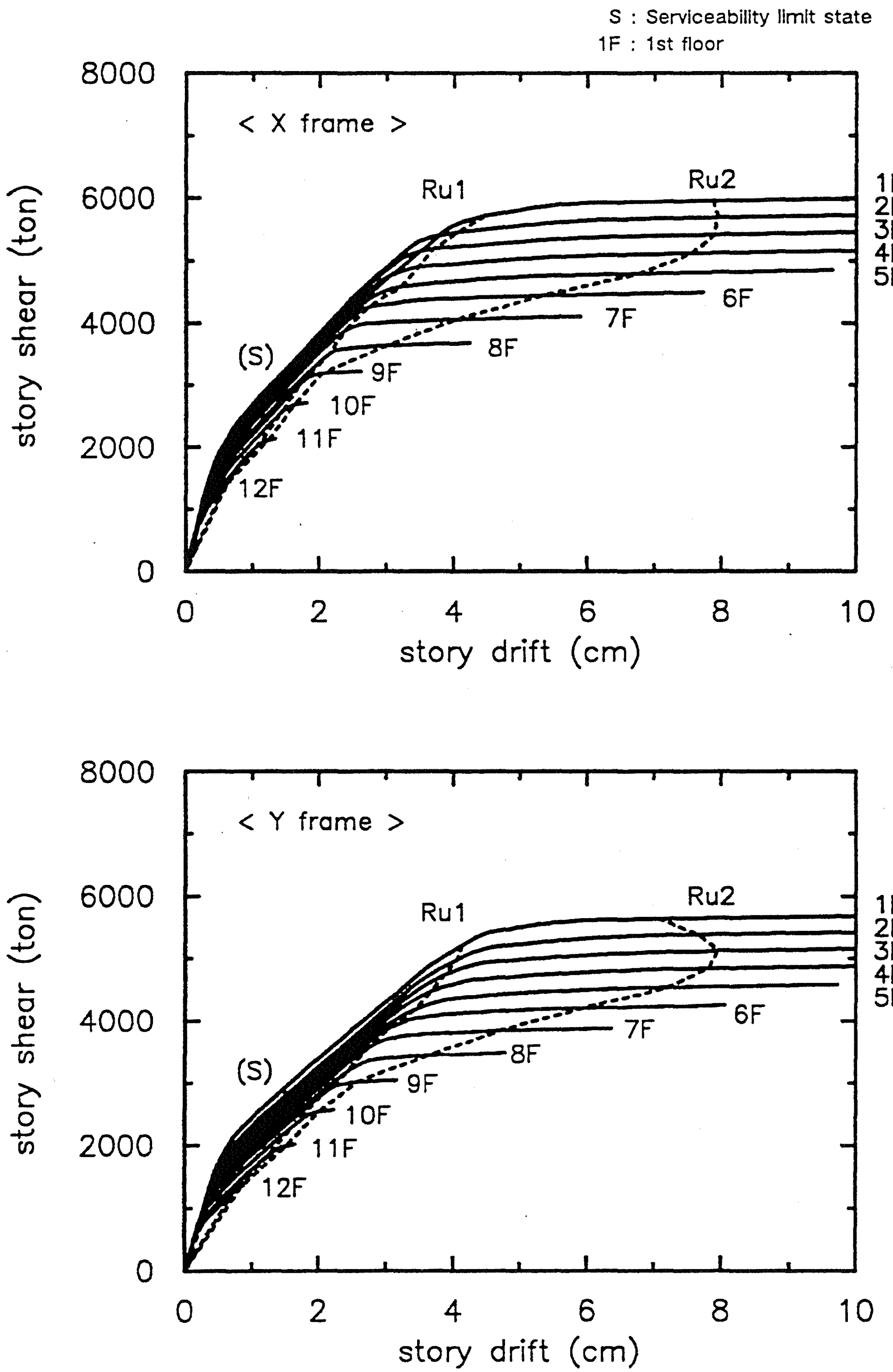
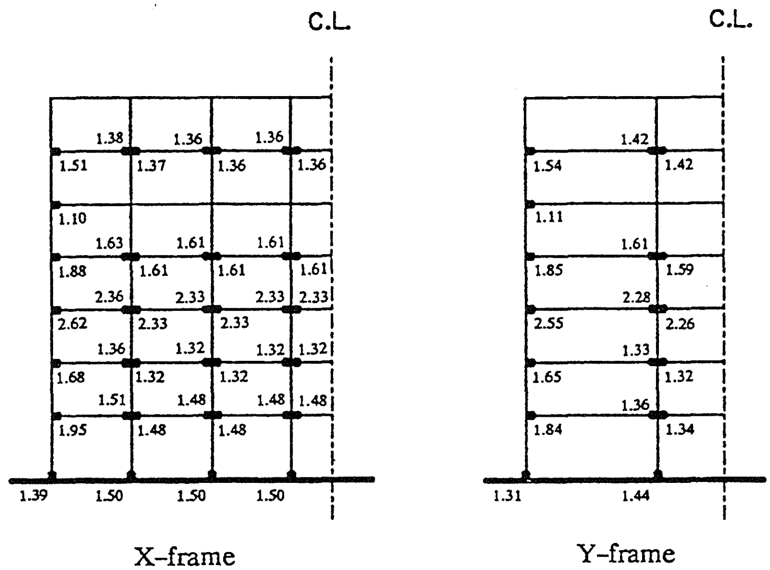
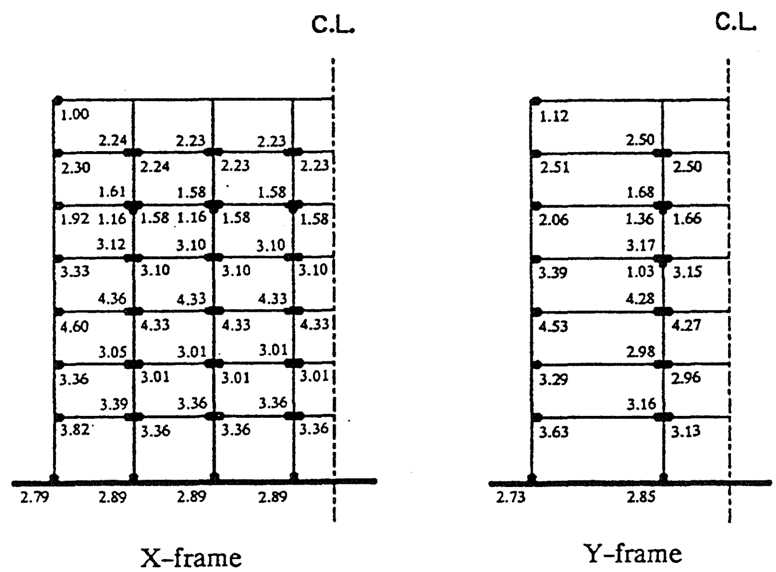


Figure 3.14 Relation between story shear force and story drift (12–story building)

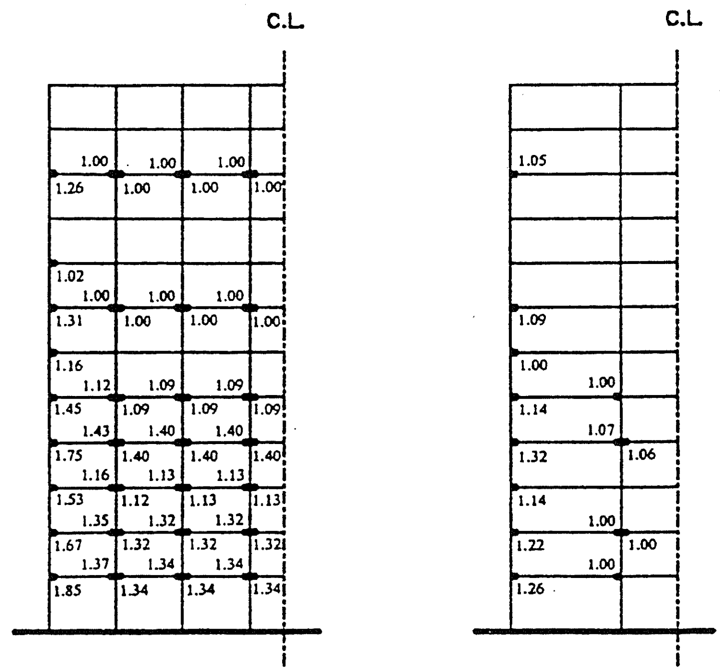


(a) Design limit deformation R_{u1}

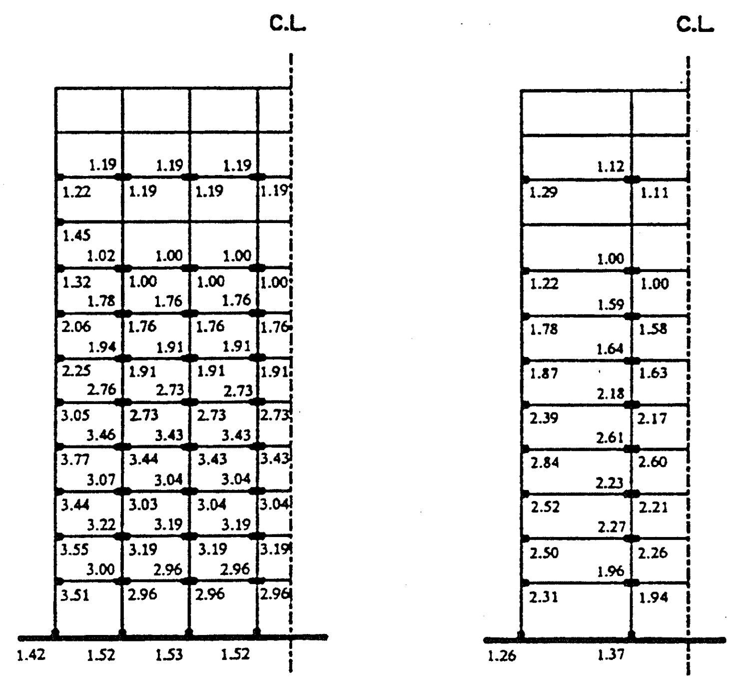


(b) Design proof deformation R_{u2}

Figure 3.15 Location of yield hinges and associated ductility factors (7-story building)



(a) Design limit deformation R_{u1}



(b) Design proof deformation R_{u2}

Figure 3.16 Location of yield hinges and associated ductility factors (12–story building)

CHAPTER 4
EVALUATION OF STRUCTURAL SAFETY

4.1 Introduction

The primary objective of seismic design of structures is to protect life safety during and following an earthquake. To achieve this, most current design codes impose limits on the response of structures when subjected to "design earthquake conditions", e.g., the maximum story drift criteria under static design earthquake loads. However, there is much uncertainty in the selection of the design earthquake because of the uncertainty in predicting the magnitude and frequency of occurrence of future earthquakes. There are also uncertainties in the design and analysis procedures. Consequently, the seismic safety of a building is also uncertain and only justified by experience and engineering judgment.

Probabilistic methods can be used to better quantify the uncertainties associated with earthquake loads and structural responses. The safety of a structure can be quantified in terms of exceedance probabilities of structural responses during the structure's lifetime.

One relatively simple approach for estimating the probability that a response will exceed some threshold value is based on a Poisson process model. Let X be a random variable representing the response quantity of interest, and let $F_X(x)$ be the cumulative distribution function of X. If the probability of X exceeding some threshold value, x_c, is associated with the occurrence of some precursory event, then $(1-F_X(x_c))$ represents the probability of X exceeding x_c given the occurrence of the precursory event. If the occurrence of the precursory event is assumed to follow a Poisson process with mean occurrence rate, ν_0, then the probability of X exceeding x_c over a lifetime t is

$$p_f = P(X>x_c \text{ in } [0,t]) = 1 - \exp(-\nu_0 t\,[1-F_X(x_c)]) \tag{4.1}$$

This formula can be applied to the determination of seismic safety as follows. First, it can be assumed that the occurrence of earthquakes follows a Poisson process. Second, the variable X can be considered as either earthquake intensity or some structural response quantity. With these assumptions and definitions, the exceedance probability over the structural lifetime can be estimated by the above equation.

In this chapter, the exceedance probabilities of two measures of earthquake intensity (peak ground acceleration and peak ground velocity) and of two different maximum structural response quantities (linear elastic acceleration response spectrum and the maximum story drift) are examined for both the Sendai and Tokyo sites. The buildings considered in this study are reinforced concrete moment–resisting frames designed according to the AIJ design guideline. Based on the historical earthquake data discussed in Chapter 2, the earthquake occurrence rates are determined to be 10.8 and 7.8 at the Sendai and Tokyo sites, respectively. The procedures used to evaluate the cumulative distribution function, $F_X(x)$, are described in the following section. Based on the exceedance probabilities calculated using Equation (4.1), the safety level implied in the Japanese design guideline is also discussed.

4.2 Seismic Risk Analysis in Terms of Exceedance Probability

Using standard simulation techniques, 100 sample values of magnitude M, epicentral distance R, and response spectrum uncertainty factor α are obtained. (See Chapter 2 for discussion of these parameters.) It is assumed that M, R, and α are mutually independent. For each set (M_i, R_i, α_i) (i=1,...,100), the response spectrum $RS_i(\omega)$ is obtained by Katayama's attenuation formula (Equation (2.5)), and a ground acceleration record, $a_i(t)$, is generated using the method developed by Yeh and Wen. (The details of this procedure have been presented in Chapter 2.) With this information, three different risk analyses are performed as described below.

4.2.1 Risk Analysis in Terms of the Exceedance Probability of Ground Accelerations and Velocities

The new Japanese design guideline provides two different design criteria associated with two different earthquake load levels ; one is the criteria for moderate earthquakes and the other is the criteria for strong earthquakes. In the guideline, there is no clear definition of "moderate" and "strong" earthquakes ; however, the guideline does state the following :

- A moderate earthquake is an earthquake which may occur several times during the lifetime of a building. Moderate earthquakes can generate maximum ground accelerations of 100–120 gal (1 gal = 1 cm/sec^2) or maximum ground velocities of 15–20 kine (1 kine = 1 cm/sec).
- A strong earthquake is an earthquake which may possibly occur once during the lifetime of a building. Strong earthquakes can generate maximum ground accelerations of 300–400 gal or maximum ground velocities of 40–50 kine.

The UBC code defines the design earthquake level based on an estimated 10% probability of being exceeded in 50 years. This definition can be used for other natural hazards such as wind and snow.

This section examines the probabilities of exceeding two measures of earthquake intensity (ground acceleration and ground velocity) during the lifetime of a building. Instead of using the actual peak ground acceleration and the actual peak ground velocity obtained from the time history of earthquake ground motion, the effective peak acceleration (EPA) and the effective peak velocity (EPV) are used. These "effective" quantities provide a better measure of the damage potential of an earthquake (Newmark and Hall, 1987).

The 1991 NEHRP (National Earthquake Hazards Reduction Program) Provisions define EPA and EPV as

$$\mathrm{EPA} = S_A / 2.5 \tag{4.2}$$

$$\mathrm{EPV} = S_V / 2.5 \tag{4.3}$$

where S_A = a 5 percent damped acceleration response spectrum ordinate of a ground motion in the period range of 0.1 sec to 0.5 sec.

S_V = a 5 percent damped velocity response spectrum ordinate at a period of about 1.0 sec.

The simulated earthquake ground acceleration records generated in Chapter 2 have significant high frequency content as indicated by the power spectral density function shown in Figure 2.11. A sample simulated record is shown in Figure 4.1. The peak acceleration for this record is 56.9 gal. The ground velocity record was calculated from the acceleration record using numerical integration, and the peak ground velocity was found to be 10.9 kine. The EPA and EPV of this sample ground motion were calculated based on the 5 percent damped response spectrum as shown in Figure 4.2. The EPA is calculated using the average of the acceleration response spectrum ordinates (S_A) in the range of 0.1 sec to 0.5 sec, and the EPV is calculated using the average velocity response spectrum ordinates (S_V) in the range of 1.0 sec to 2.0 sec. Both the EPA and the EPV are much smaller than the actual peak ground acceleration and the actual peak ground velocity.

The EPA and EPV for each of the 100 sample waves were plotted on a lognormal probability paper as shown in Figures 4.3 and 4.4, respectively. The data appears to follow a lognormal distribution. The median, xm, and the standard deviation, ς, of ln(EPA) or ln(EPV) were obtained from the plots. The parameters, xm and ς, are defined as

$$xm = \frac{\mu}{\sqrt{1 + \delta^2}} \quad , \quad \zeta = \sqrt{\ln(1 + \delta^{2)}} \tag{4.4}$$

where μ is the mean value of the data and δ is the coefficient of variation of the data.

Assuming the cumulative distributions of EPA and EPV as lognormal distributions, the exceedance probabilities, p_f, in 5 and 50 years were evaluated using Equation (4.1) and are listed in Table 4.1. The results are summarized as follows :

- Although both Sendai and Tokyo are located in the same design seismic zone, the EPA and the EPV for the Tokyo site are 1.2–1.5 times higher than those for the Sendai site.

- The values of EPA which have a 10% chance of being exceeded in 50 years (corresponding to the design earthquake in the UBC code) are 381.4 gal (39% of gravity) at the Sendai site and 572.2 gal (58% of gravity) at the Tokyo site. These values are equal to or higher than the maximum earthquake acceleration levels for strong earthquakes mentioned in the AIJ guideline.

- The values of EPV which have a 10% chance of being exceeded in 50 years are 34.4 kine at the Sendai site and 50.3 kine at the Tokyo site. These values are comparable to the maximum earthquake velocity levels for strong earthquakes mentioned in the AIJ guideline.

4.2.2 Risk Analysis in Terms of the Exceedance Probability of Response Spectra

The design response spectrum is a basic piece of information used for the seismic design of structures. Using statistical theory, the exceedance probability of response spectrum ordinates is examined as described below.

Response spectrum ordinates for each of the 100 sample waves were plotted on a lognormal probability paper at 18 different periods. Note that these spectrum ordinates were obtained by the attenuation formula developed by Katayama(1978) for each set (M_i, R_i, α_i) (i=1,...,100). As shown in Figure 4.5 (for a period of 0.2 sec), the distribution of the data appears to follow a lognormal distribution. With this cumulative distribution of the response spectrum ordinates, the exceedance probability, p_f, in t years can be evaluated using Equation (4.1).

In order to compare the results in this study with the results presented by Katayama (1978), the response spectrum corresponding to $p_f = 0.632$ and $t = 75$ years was calculated and is presented in Figure 4.6. Although the seismic environment was simplified in this study as compared to the seismic environment considered by Katayama, there is generally good agreement between the results of the two studies.

4.2.3 Risk Analysis in Terms of the Exceedance Probability of Story Drift Ratio

The nonlinear earthquake response of the reinforced concrete buildings was calculated using the computer program FRAME–D. The details of the building design are described in Chapter 3. Only the Y–frame was analyzed for each building. The modeling of structural members (columns and beams) is the same as that used in the nonlinear static analysis (see Section 3.3.2). Newmark's β–method (β=0.25) was used to integrate the equations of motion. The damping matrix was derived from the mass and the initial elastic stiffness matrices by Rayleigh's method with damping factors of 0.05 for the first and second natural frequencies. This damping matrix remained unchanged throughout the analysis. The natural periods of the frames are 0.67 sec for the 7–story building and 1.05 sec for the 12–story building. The uncertainties in structural properties were assumed to be small compared to the uncertainties in ground motion parameters and thus were not considered in this study.

The model frames were analyzed for each of the 100 simulated earthquake records. The maximum story drift ratios from each analysis were plotted on lognormal probability paper. Figures 4.7 and 4.8 present these plots for the 1st story of each frame model. It appears that the data follow the log-normal distribution. Using this distribution, the story drift ratios corresponding to various exceedance probabilities in 50 years were calculated using Equation (4.1) and are listed in Tables 4.2 and 4.3. The results are summarized as follows :

- The story drifts of the buildings at the Tokyo site are generally larger than those at the Sendai site. The drifts are 2.1–2.5 times larger for the 7–story building and 1.2–1.7 times larger for the 12–story building.

- At the Sendai site, the story drifts of the 12–story building are about 1.2 times larger than those of the 7–story building. On the other hand, at the Tokyo site, there are no large differences between the story drifts of the 7–story building and the 12–story building.

- The maximum story drift ratio is observed in the second story in all cases, and the story drift decreases as one goes to higher stories.

It seems that the building at the Tokyo site is more susceptible to damage from earthquakes than the building at the Sendai site. However, as shown in Figures 4.7 and 4.8, the median values of the data are not so different between the two sites, but the standard deviations of the data (which correspond to the slope of the data plot in each case) are different. Since the estimation of the story drift corresponding to a small exceedance probability is significantly affected by a slight change in the standard deviation, it is difficult to conclude which site is more susceptible to damage. More sample data are required to make such a determination.

Table 4.1 Effective peak acceleration (EPA) and effective peak velocity (EPV) corresponding to various exceedance probabilities in 5 and 50 years

(a) EPA (gal)

	SENDAI		TOKYO	
P_f	T = 5 years	T = 50 years	T = 5 years	T = 50 years
0.50	115.6	235.7	160.7	344.7
0.25	155.1	297.8	220.3	441.5
0.15	184.5	344.0	266.0	514.0
0.10	209.7	381.4	304.5	575.2
0.05	255.8	451.5	376.1	686.9

(b) EPV (kine)

	SENDAI		TOKYO	
P_f	T = 5 years	T = 50 years	T = 5 years	T = 50 years
0.50	11.6	22.2	12.9	29.1
0.25	15.2	27.5	18.1	37.9
0.15	17.8	31.3	22.1	44.6
0.10	20.0	34.4	25.5	50.3
0.05	23.9	40.0	32.0	60.8

Table 4.2 Story drift ratio (% of story height) corresponding to various exceedance probabilities in 50 years (7–story building model)

(a) SENDAI

p_f	story drift ratio (%) (T=50 years)						
	1F	2F	3F	4F	5F	6F	7F
0.50	0.272	0.293	0.262	0.238	0.185	0.136	0.077
0.25	0.342	0.369	0.330	0.302	0.233	0.170	0.097
0.15	0.394	0.425	0.381	0.349	0.268	0.196	0.111
0.10	0.435	0.471	0.422	0.388	0.297	0.217	0.123
0.05	0.513	0.556	0.499	0.460	0.351	0.256	0.144

(b) TOKYO

p_f	story drift ratio (%) (T=50 years)						
	1F	2F	3F	4F	5F	6F	7F
0.50	0.586	0.639	0.592	0.543	0.415	0.277	0.153
0.25	0.793	0.867	0.807	0.741	0.562	0.370	0.203
0.15	0.954	1.046	0.976	0.897	0.676	0.442	0.242
0.10	1.095	1.201	1.123	1.034	0.776	0.504	0.275
0.05	1.358	1.494	1.401	1.292	0.964	0.620	0.336

Table 4.3 Story drift ratio (% of story height) corresponding to various exceedance probabilities in 50 years (12–story building model)

(a) SENDAI

p_f	story drift ratio (%) (T=50 years)											
	1F	2F	3F	4F	5F	6F	7F	8F	9F	10F	11F	12F
0.50	0.398	0.494	0.459	0.434	0.394	0.339	0.315	0.284	0.223	0.168	0.117	0.059
0.25	0.502	0.627	0.581	0.549	0.497	0.426	0.397	0.360	0.281	0.210	0.145	0.073
0.15	0.579	0.726	0.671	0.634	0.574	0.490	0.458	0.416	0.323	0.241	0.166	0.082
0.10	0.642	0.807	0.745	0.704	0.636	0.542	0.507	0.462	0.358	0.266	0.183	0.090
0.05	0.759	0.957	0.882	0.834	0.752	0.639	0.600	0.548	0.422	0.312	0.215	0.105

(b) TOKYO

p_f	story drift ratio (%) (T=50 years)											
	1F	2F	3F	4F	5F	6F	7F	8F	9F	10F	11F	12F
0.50	0.556	0.676	0.616	0.568	0.537	0.491	0.442	0.384	0.300	0.229	0.165	0.082
0.25	0.750	0.914	0.828	0.761	0.720	0.657	0.588	0.507	0.390	0.294	0.212	0.104
0.15	0.902	1.100	0.993	0.911	0.862	0.786	0.700	0.600	0.458	0.343	0.247	0.120
0.10	1.033	1.261	1.137	1.040	0.985	0.897	0.796	0.680	0.516	0.385	0.276	0.133
0.05	1.280	1.566	1.405	1.283	1.215	1.105	0.976	0.829	0.622	0.461	0.330	0.157

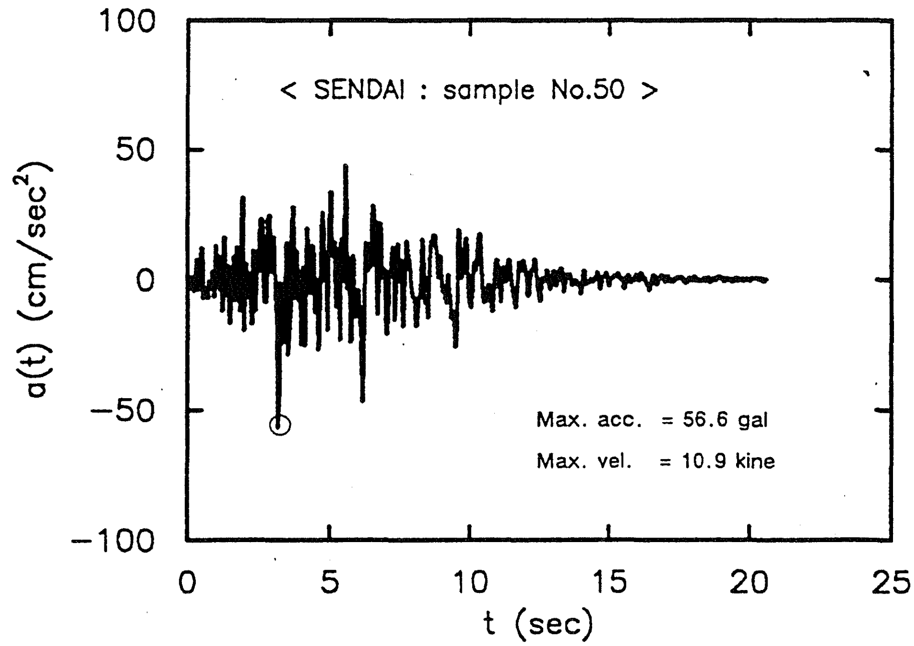


Figure 4.1 Example of a simulated earthquake ground motion record

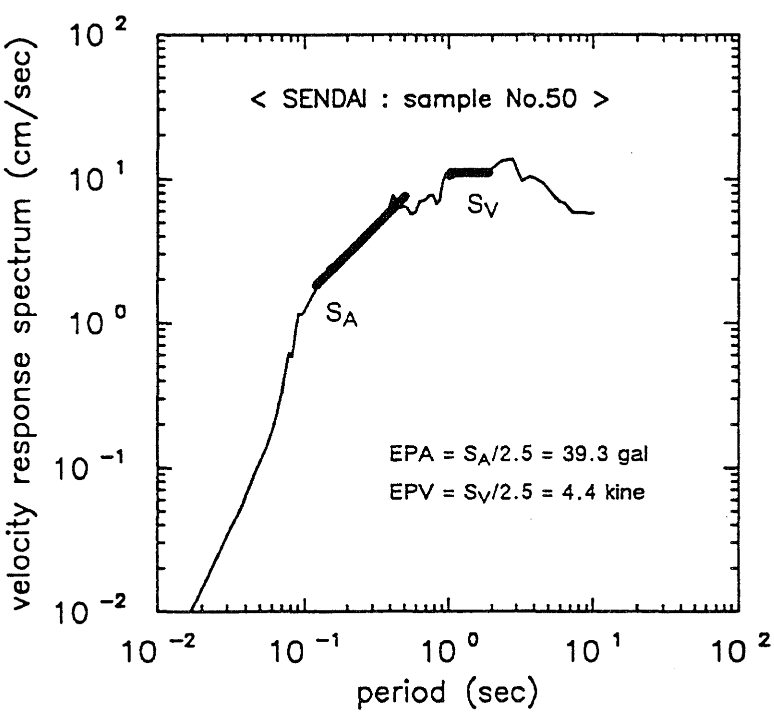


Figure 4.2 Velocity response spectrum (5% damping)

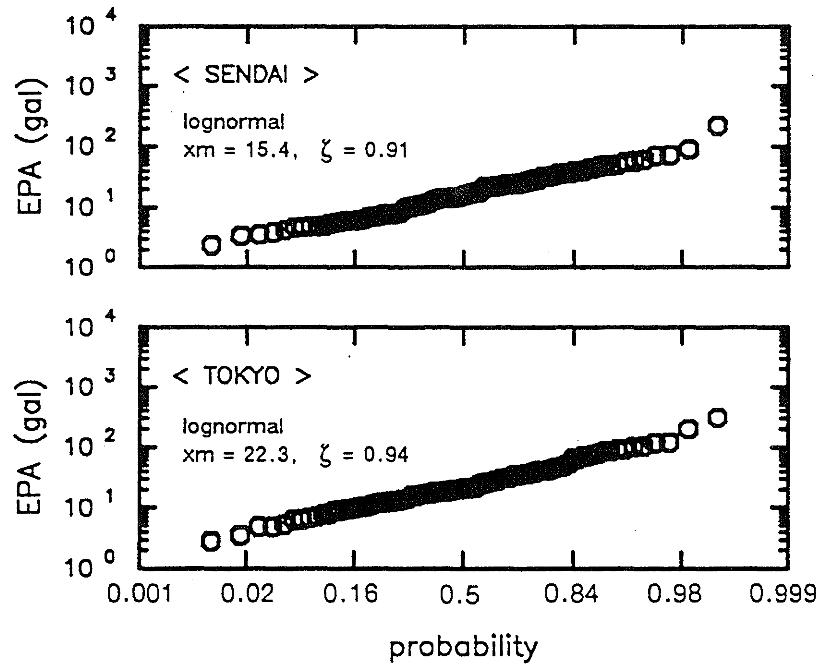


Figure 4.3 Distribution of effective peak acceleration (5% damping)

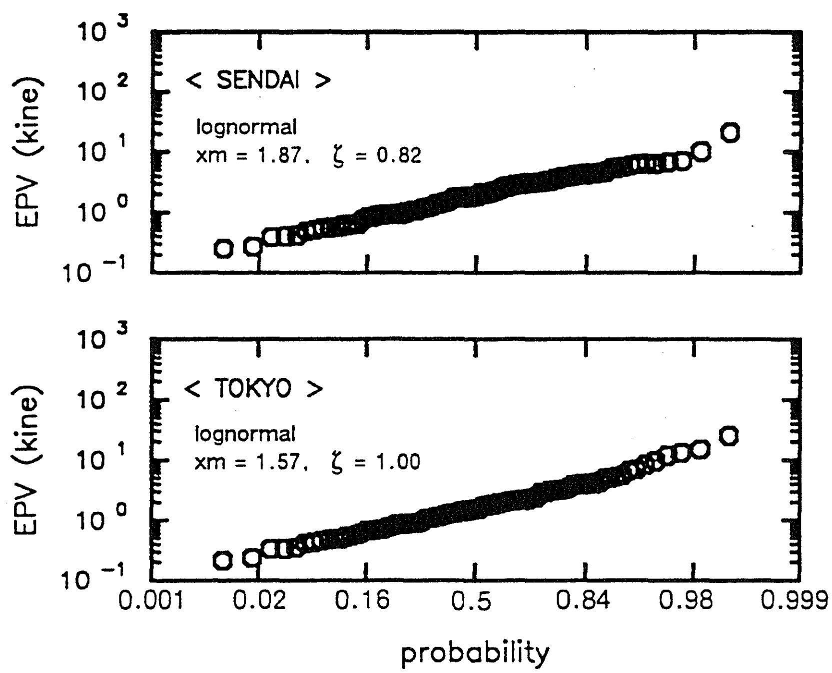


Figure 4.4 Distribution of effective peak velocity (5% damping)

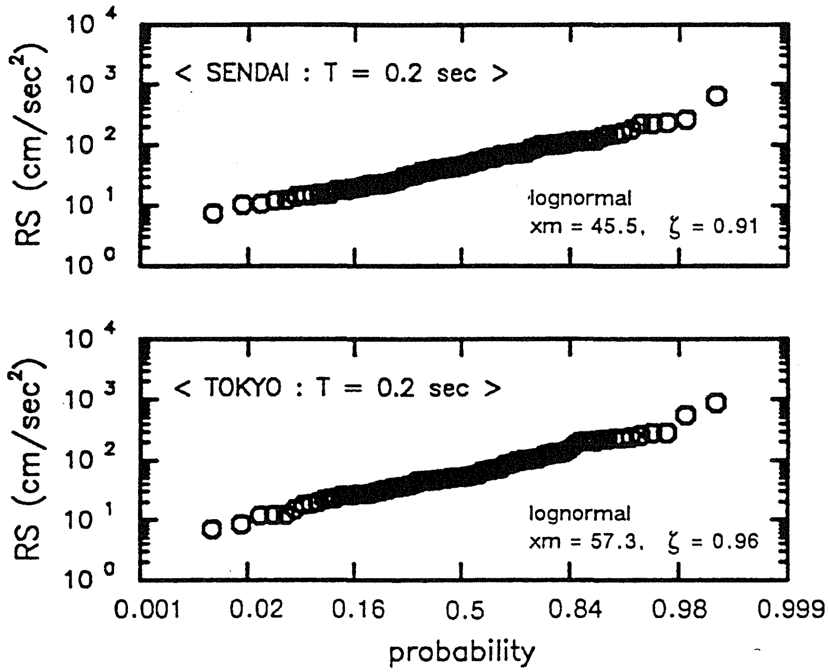


Figure 4.5 Distribution of acceleration response spectrum (5% damping)

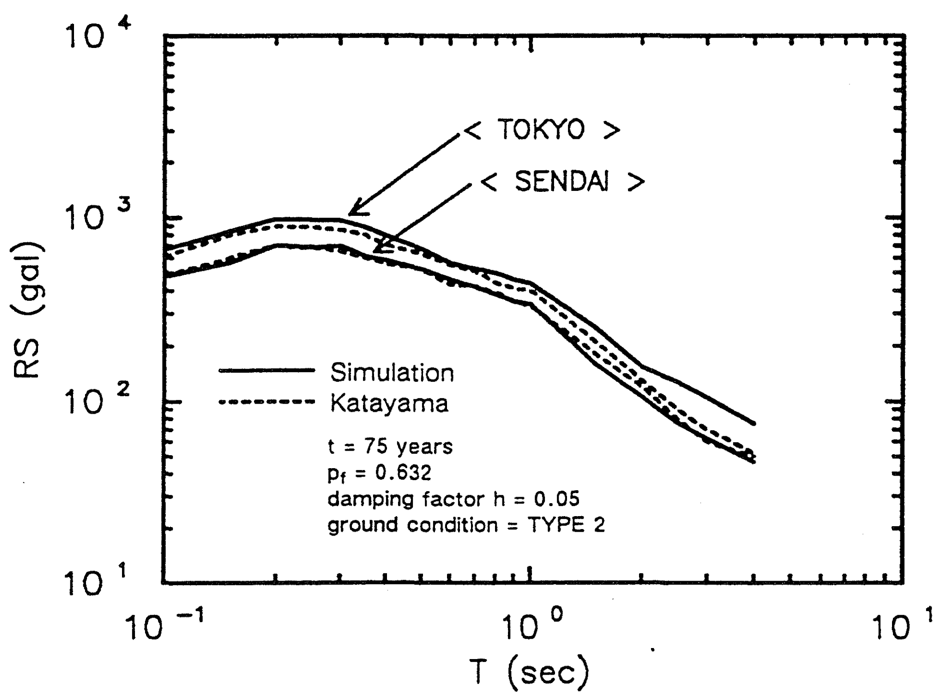


Figure 4.6 Comparison of acceleration response spectra corresponding to an exceedance probability of 0.632 in 75 years

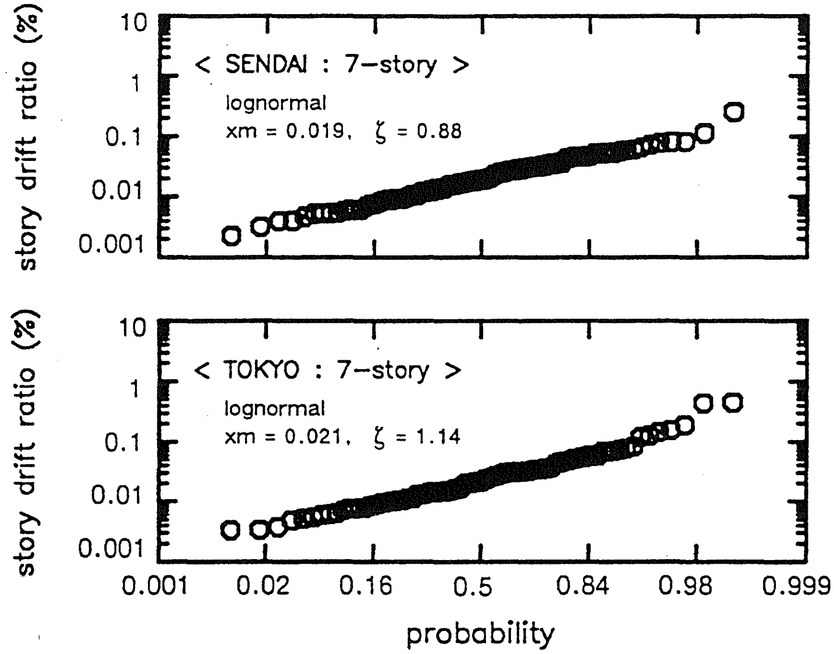


Figure 4.7 Distribution of 1st story drift ratio (7–story building)

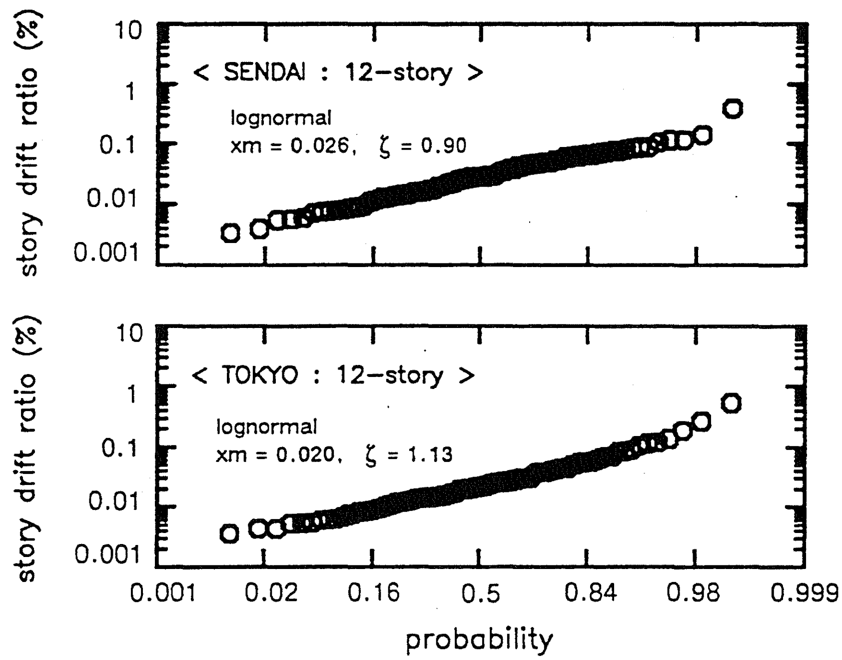


Figure 4.8 Distribution of 1st story drift ratio (12–story building)

CHAPTER 5
SUMMARY AND CONCLUSIONS

5.1 Summary

The seismic safety of reinforced concrete moment–resisting frames designed according to a new Japanese design guideline are evaluated using probabilistic methods.

Two cities in active seismicity areas in Japan, Sendai and Tokyo, are assumed as the sites of the designed buildings. The historical earthquake data available at the two sites are used to construct the probability distribution models of earthquake magnitude and empirical distance. From information on magnitude, source distance, and site soil conditions, the acceleration response spectrum at each site is obtained using an attenuation formula. This response spectrum is then used to obtain a compatible power spectrum by iterative calculations.

Earthquake ground motions are simulated by a nonstationary stochastic process model which accounts for the time–varying characteristics of earthquake intensity and frequency content. The model is described by the intensity function, the frequency modulation function, and the Clough–Penzien power spectrum. The parameters of the Clough–Penzien spectrum are identified based on the power spectrum which is compatible with the response spectrum for the site. The other functions are determined based on actual earthquake records measured at the site. A total of 100 simulated earthquake ground motion records are generated for each site.

Two reinforced concrete buildings, 7–story and 12–story moment–resisting frames, are designed according to the recently proposed Japanese design guideline. The performance of the designed building is examined by a nonlinear frame analysis with static lateral incremental forces. Two design criteria are then checked ; one is the criteria for the serviceability limit state, and the other is the criteria for ultimate limit state. The locations of yield hinges and associated ductility factors are also examined at the design limit deformation and the design proof deformation specified in the guideline.

The seismic safety of each building is evaluated based on the probabilities of exceeding certain threshold levels of response during the lifetime of the building. The intensities of earthquake are measured by the effective peak acceleration (EPA) and the effective peak velocity (EPV). The cumulative distributions of EPA and EPV are assumed as lognormal distributions. The EPA and EPV corresponding to various exceedance probabilities over the periods of 5 and 50 years are then calculated assuming a Poisson process model for earthquake occurrence. The exceedance probability of the acceleration response spectrum is also examined and the results are compared with those of another study. Nonlinear dynamic frame analyses are carried out using the 100 simulated earthquake records. The computer program FRAME–D is used for the analysis. The nonlinear behavior of constituent structural members are appropriately modeled by nonlinear member models available in FRAME–D. The maximum story drifts in each designed building are calculated and the exceedance probability of story drift during an assumed lifetime of the buildings is estimated.

5.2 Conclusions

The conclusions drawn from this study are as follows :

- A methodology of modeling earthquake ground motion at sites in Japan is developed. The occurrence rate of earthquake magnitude can be estimated using a truncated Gutenberg–Richter formula which accounts for the upper and lower bounds of magnitude. The probability distribution of epicentral distance can be modeled by a simple triangular distribution. The earthquake ground motion is generated as a nonstationary stochastic process taking into consideration the large uncertainties in the assumed attenuation equation (Equation (2.4)).

- The concept of the AIJ guideline providing two levels of design criteria associated with two levels of design earthquake forces is quite rigorous. However, the seismic safety implied in the guideline is unknown. The procedures of risk analysis presented in this study can be used to evaluate the reliability implied in the design guideline.

- Although the two sites, Sendai and Tokyo, are located in a same seismic zone specified in the design guideline, the evaluated seismic risk at the Tokyo site is much higher than the seismic risk at the Sendai site. This suggests that probabilistic methods should be used to define new seismic zones based on seismic risk.

- The effective peak ground acceleration (EPA) and the effective peak ground velocity (EPV) can be used to characterize the intensity of earthquake ground motion. The level of strong earthquake mentioned in the Japanese guideline corresponds approximately to an earthquake with 10% exceedance probability in 50 years.

- The acceleration response spectrum at each site corresponding to an exceedance probability of 0.632 in 75 years is estimated. The estimated response spectrum compares well with the response spectrum given by Katayama (1978).

- The story drift ratios corresponding to various exceedance probabilities in 50 years are estimated for the designed buildings at each site. For the 7–story building, the estimated maximum story drift at the Tokyo site is more than twice the estimated maximum drift at the Sendai site. However, the median values of story drift for both sites are almost identical. This result suggests that the risk evaluation at small exceedance probabilities is quite sensitive to the variance of the data.

5.3 Future Study

This study is part of the on–going research to develop reliability–based seismic design procedures. The methodology presented in this study can be used to evaluate the seismic safety level implied in a structural design code. In particular, the results of this study should be helpful in formulating future design guidelines for reinforced concrete buildings in Japan. Such efforts to formulate a new design guideline are currently in progress.

The suggestions for future study are summarized below :

- To estimate the seismic risk associated with realistic buildings, a large number of simulations (including generating earthquake ground motions and performing nonlinear dynamic structural response analyses) are required. An efficient simulation technique must be developed to streamline the risk estimation process.

- Destructive earthquakes are often associated with return periods of more than 100 years. However, high quality historical earthquake data are available only for recent earthquakes. Therefore, methods of combining additional information (such as active fault data) with the historical data need to be developed in order to obtain better predictions of future earthquakes.

- Further studies are also needed to extend the method presented herein to take into consideration the uncertainties associated with structural properties. For example, the uncertainty associated with concrete strength is quite large, and the effect of this uncertainty on structural safety is unknown.

APPENDIX

A COPY OF

"ULTIMATE STRENGTH DESIGN GUIDELINES FOR REINFORCED CONCRETE BUILDINGS"

(This document is included for information only.)

ULTIMATE STRENGTH DESIGN GUIDELINES

FOR

REINFORCED CONCRETE BUILDINGS

by

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March 31, 1993

ABSTRACT

This paper introduces "Ultimate Strength Design Guidelines for Reinforced Concrete Buildings," developed as a part of U.S.–Japanese Coordinated PRESSS (Precast Seismic Structural System) project. The guidelines has been drafted by Guidelines Drafting Working Group, and discussed in Design Guidelines Committee of Japanese PRESSS. Extensive commentary will accompany the guidelines to explain the concept behind requirements and to suggest methods of calculation.

LIST OF CONTENTS

CHAPTER 1 SCOPE

1.1 General Requirements

(1) This design guidelines provides minimum requirements for the design of cast–in–situ reinforced concrete (RC) buildings and reinforced concrete (PCaRC) buildings which use precast reinforced concrete (PCa) members for earthquake resistant elements.

(2) The application of some requirements can be exempted if a structure, designed on the basis of special studies, can be demonstrated to posses structural performance as good as or superior to those buildings designed under this guidelines.

1.2 Building Height

The total height of a building shall be not more than 60 m.

1.3 Structural System

A structure, in each principal direction, shall consist of ductile moment–resisting frames, or of ductile moment–resisting frames combined with continuous structural walls, or of independent structural walls. The structural wall shall be continuous from the foundation to the roof level.

1.4 Structural Configuration

(1) Eccentricity ratio and rigidity ratio of a structure, defined in Article 82–3 of Building Standard Law Enforcement Order, shall be not more than 0.15 and not less than 0.6, respectively.

(2) Height–to–width ratio (aspect ratio) at any level of a building shall be, as a general rule, not more than 4.

1.5 Yield Mechanism

The structure of a building shall be clearly planned to form a specified total yield mechanism, in which flexural yield hinges shall develop, as a general rule, at the ends of all floor beams and at the base of the first story columns and structural walls.

1.6 Site Geology

Soil types at a construction site shall be, as a general rule, Type 1 or Type 2, defined in Notification No. 1793 of Ministry of Construction.

1.7 Definitions

Some technical terms, used specifically in the guidelines, are defined below;

PCa connection: Connection between two PCa elements or between a PCa element and an

RC member;

Major earthquake resisting elements: Out of major structural parts, defined in Article 3–1 of Building Standard Law Enforcement Order, columns, girders, structural walls, foundation girders, foundation slabs and foundation piles;

Yield hinge (region): The location (region) to develop plastic deformation by flexural yielding under the action of bending;

Special yield hinge: Yield hinge of columns and structural walls where special confining reinforcement is required to resist high axial loading;

Non–yield hinge (region): The location (region) where yield hinge does not form;

Serviceability limit state design: Design of a structure under long–term loading and small to medium intensity earthquake motion to ensure serviceability of the structure;

Ultimate limit state design: Design of a structure under a strong earthquake motion, that may occur once during the life time, to ensure the safety and reuse of the structure with extensive repair work;

Design limit deformation R_{u1}: Deformation of a structure or members expected to develop under an intense earthquake motion;

Design proof deformation R_{u2}: Deformation of a structure to which the performance of the structure is assured in design.

CHAPTER 2 DESIGN PRINCIPLES

2.1 Structural Performance

2.1.1 Building

(1) A building shall resist gravity loads and medium intensity ground motions, wind pressure and snow loads without disturbing serviceability.

(2) The structural part of a building above the ground level shall be designed to develop total yield mechanism under an intense earthquake motion, and shall be provided with stiffness and lateral resistance sufficient to limit the lateral deformation within a specified value.

Foundation structure and the structural part in the basement shall safely transfer the vertical and lateral loads from the super–structure to the soil, and major earthquake resisting elements in the basement, foundation girders, foundation slabs and piles shall not, as a general rule, yield even under an intense earthquake motion.

2.1.2 PCa Members and Connections

(1) PCa connections shall be provided with strength sufficient to transfer member actions caused by specified design loads.

(2) PCa connections shall be designed to limit the strength deterioration and slippage deformation, inherent to PCa connections, to satisfy "Evaluation Criteria for PCa Connection Performance".

(3) PCa members and connections shall satisfy required serviceability, durability and fire resistance.

2.1.3 Non–structural Elements

(1) Non–structural elements shall be connected to structural members to ensure service–ability during gravity, snow loading, wind pressure, and during medium intensity earthquake motions, and also not to influence the development of the specified yield mechanism of a structure during an intense earthquake motion.

(2) Non–structural elements and attachments shall be fastened to structural members so that their falls will not damage the function and safety of the building.

2.2 Method of Structural Design

2.2.1 Design Principle

Design of major structural members of a building shall be designed for gravity loads, earthquake loads, wind pressure and snow loads to satisfy the structural performance defined in Chapter 1 and Section 2.1. Structural calculation of PCa members may follow the method for RC members.

2.2.2 Design for Gravity Loads

(1) Stress in every part of major structural members under dead load, specified in Article 84 of Building Standard Law Enforcement Order, live load, specified in Article 85, and snow load, specified in Article 86, in heavy snow zones designated by specific administrative office, shall not exceed allowable stresses for the long–term loading specified in Section 3.3.

Long term loads shall include soil pressure, water pressure, vibration, impact, temperature, shrinkage, uneven ground settlement, if applicable, to the structure.

(2) Structural members and PCa connections shall not develop excessive cracking, deflection or vibration for serviceability and durability under the loads defined above.

2.2.3 Design for Earthquake Loads

(1) Serviceability and ultimate limit state design of structural members in the super–structure under earthquake loads shall conform to the provisions of Chapter 4. Those buildings not taller than 31 m can be designed by the provisions of Chapter 5.

(2) Serviceability and ultimate limit state design of structural members in the basement and foundation under earthquake loads shall conform to the provisions of Chapter 6.

2.2.4 Design for Wind Pressure

Stress in every part of structural members under wind pressure and combined loads, stipulated in Paragraph 2 of Article 82–1 of Building Standard Law Enforcement Order, shall not exceed allowable stresses for the short–term loading specified in Section 3.3.

2.2.5 Design for Snow Loads

Stress in every part of structural members under snow loads and combined loads, stipulated in Paragraph 2 of Article 82–1 of Building Standard Law Enforcement Order, shall not exceed allowable stresses for the short–term loading specified in Section 3.3.

Material constants for mechanical properties of reinforcing bars shall be as follows:

Young's modulus: 2.1×10^6 kgf/cm^2 (3.4)

Coefficient of thermal expansion: 1×10^{-5} (/deg C) (3.5)

(3) Mortar and Grout

Young's modulus of mortar shall be assumed equal to the smaller value specified for RC and PCa members at the PCa connection.

(4) Steel

Material constants for mechanical properties of steel shall be as follows:

Young's modulus: 2.1×10^6 kgf/cm^2 (3.6)

Coefficient of thermal expansion: 1×10^{-5} (/deg C) (3.7)

3.3 Allowable Stresses and Material Strength

(1) Concrete

Allowable stresses and material strength of concrete shall be taken as specified in Tables 3.2 and 3.3. Material strength for bond may be determined by experimental or analytical study.

(2) Reinforcing Bars

Allowable stresses and material strength of reinforcing bars shall be taken from Table 3.4.

(3) PC Steel Bars

Allowable stresses and material strength of prestressed concrete steel bars, steel wire and strand shall be equal to the values specified in "AIJ Standard for Design and Construction of Pre-stressed Concrete."

(4) Steel

(a) Allowable stress of steel shall be as specified in Article 90 of Building Standard Law Enforcement Order.

(b) Material strength of steel shall be as specified in Article 96 of Building Standard Law Enforcement Order.

(5) Mortar and Grout

Allowable stresses and material strength of mortar and grout shall be equal to the smaller value of RC and PCa members.

"Uncoated Stress–Relieved Steel Wires and Strand for Prestressed Concrete."

(c) Welded wire fabric shall meet the provisions of JIS G–3551 "Welded Steel Wire Fabric." Nominal diameter of steel wire shall be not less than 4 mm.

(5) Grout and Mortar

(a) Compressive strength of mortar used at PCa connections shall be as high as or superior to that of concrete for RC and PCa members.

(b) Types, quality, mix, production and materials shall meet the provisions of "PRESSS PCa Construction Guidelines."

(6) Steel Materials

Quality of steel elements for PCa members shall be specified in design specifications. The shape and dimensions of steel elements shall be specified in design specifications and drawings.

(7) Joint of Reinforcing Bars and Steel

(a) Reinforcing bars may be jointed by gas pressured welding, flare welding, or lap splicing.

(b) Lap splice shall meet the requirements of "AIJ Standards for Structural Calculation of Reinforced Concrete Structures."

(c) Work of gas pressured welding shall meet "Standard Specification for Gas Pressured Welding Work for Reinforcing Bars" by Japan Gas Pressured Welding Institute.

(d) Steel plates may be jointed by welding or high tension bolts.

(e) Work of welding and high tension bolt friction joint of steel plates and work of flare welding of reinforcing bars shall meet the provisions of JASS–10 "Precast Concrete Work" and JASS–6 "Steel Work."

3.2 Material Constants

(1) Concrete

Material constants for mechanical properties of concrete for RC, PCa members and PCa connections shall be as follows:

Young's modulus: $2.1 \times 10^5 (r/2.3)^{1.5} (Fc/200)^{0.5}$ (3.1)

Poison's ratio: 1/6 (3.2)

Coefficient of thermal expansion: 1×10^{-5} (/deg C) (3.3)

where, r: weight per unit volume (tonf/m^3), and Fc: specified concrete compressive strength (kgf/cm^2).

(2) Reinforcing Bars

Table 3.4 : Allowable Stresses and Material Strength for Reinforcement (kgf/cm²)

Steel Grade	Long-term Loading Tension Compres.	Long-term Loading Shear Reinf.	Short-term Loading Tension Compres.	Short-term Loading Shear Reinf.	Material Strength Tension Compres.	Material Strength Shear Reinf.
SR235	1,600	1,600	2,400	2,400	2,400 x 1.1	2,400
SR295	1,600	2,000	3,000	3,000	3,000 x 1.0	3,000
SD295A SD295B	2,000	2,000	3,000	3,000	3,000 x 1.1	3,000
SD345	2,200 (2,000)[1]	2,000	3,500	3,500	3,500 x 1.1	3,500[2]
SD390	2,200 (2,000)[1]	2,000	4,000	4,000	4,000 x 1.1	4,000[2]
Welded Wire	2,000	2,000	-----	3,000	3,000 x 1.1	----

Note: Values in parenthesis for deformed bars D29 and larger.

Table 3.2 : Allowable Stresses and Material Strength for Concrete (kgf/cm^2)

Loading Type	Action	Normal Concrete	Light–weight Concrete
Long–term Loading	Compression	Fc / 3	Fc / 3
	Tension	------	------
	Shear	Fc / 30 and (5 + Fc / 100)	0.9 times the value for normal concrete
Short–term Loading	Compression	2 times the values for long–term loading	
	Tension	------	------
	Shear	1.5 times the values for long–term loading	
Material Strength	Compression	Fc	Fc

Fc : Specified compressive strength of concrete

Table 3.3 : Allowable Bond Stresses between Reinforcement and Concrete (kgf/cm^2)

Bar Type	Long–term Loading		Short–term Loading
	Top Reinforcement	Bottom Reinforcement	
Round	0.04 Fc and 9.0	0.06 Fc and 13.5	1.5 times the values for long–term loading
Deformed	Fc / 15 and (9 + 2 Fc / 75)	Fc / 10 and (13.5 + Fc / 25)	

a) Top reinforcement: Horizontal reinforcement with more than 30 cm depth of concrete below in a flexural member;
b) For a deformed bar with concrete cover less than 1.5 time bar diameter, allowable bond stress shall be reduced by the ratio of cover depth to the length of 1.5 times bar diameter.

4.3 Ultimate Limit State Design

4.3.1 Performance Criteria at Design Limit Deformation

Story shear resistance at each story, calculated at maximum story drift angle reaching design limit deformation, shall be greater than 90 percent of the required lateral load resisting capacity.

The required lateral load resisting capacity Q_{uni} of story i shall be

$$Q_{uni} = C_{uni} W_i \qquad (4.3)$$

$$C_{uni} = Z R_t A_i C_{unB} \qquad (4.4)$$

where, Z, R_t, W_i, and A_i are defined in Section 4.2.1. Design limit deformation R_{u1} and standard base shear coefficient for required lateral load resisting capacity C_{unB} are specified in Table 4.3.1.

Table 4.3.1: Standard base shear coefficient for required lateral load resisting capacity C_{unB}, design limit deformation R_{u1}, and design proof deformation R_{u2}

Ratio b_w of OTM resisted by walls	C_{unB}	R_{u1}	R_{u2}
$0.0 < b_w < 0.3$	0.30	1/100	1/50
$0.3 < b_w < 0.7$	0.35	1/120	1/60
$0.7 < b_w < 1.0$	0.40	1/150	1/75

where, b_w: the ratio of base overturning moment (OTM) resisted by structural walls at design limit deformation;

$$b_w = S\, {}_wQ_i H_i / (S\, Q_i H_i) \qquad (4.5)$$

where, H_i: story height at i–th story, ${}_wQ_i$: sum of story shear carried by structural walls at i–th story, Q_i: total story shear at i–th story, S: sum from the first to the top story. ${}_wQ_i$ and Q_i are evaluated when the maximum story drift at a story reaches the design limit deformation.

4.3.2 Performance Criteria at Design Proof Deformation

The structure and its members shall satisfy the following conditions when the maximum story drift at a story reaches the design proof deformation R_{u2}, specified in Table 4.3.1;

(1) The story resistance at each story shall be greater than the required lateral load resisting capacity at the story,

CHAPTER 4 EARTHQUAKE RESISTANT DESIGN (NONLINEAR ANALYSIS PROCEDURE)

4.1 Design Principles

4.1.1 Serviceability and Ultimate Limit State Design

The performance of super–structure of a building shall be examined for serviceability limit state under small to medium intensity earthquake motions and for ultimate limit state under an intense earthquake motion.

4.1.2 Method of Earthquake Resistant Design

(1) Earthquake resistant design shall be based on a static nonlinear analysis of a building under monotonically increasing lateral loading taking into account realistic elastic and inelastic characteristics of constituent structural members.

(2) The analysis shall be carried out in the longitudinal and transverse directions, separately.

(3) Lateral load shall be increased monotonically in the analysis under dead load, specified in Article 84 of Building Standard Law Enforcement Order, and live load for earthquake load calculation, specified in Article 85 of the order.

4.2 Serviceability Limit State Design

4.2.1 Design Earthquake Load

Design story shear Q_i at story i under the action of an earthquake motion shall be

$$Q_i = C_i W_i \tag{4.1}$$

$$C_i = Z R_t A_i C_B \tag{4.2}$$

where, C_i: story shear coefficient, W_i: sum of dead and live loads at and above the i–th story, Z: seismic zone coefficient, R_t: vibration characteristic coefficient, A_i: coefficient for story shear distribution, C_B: standard base shear coefficient of 0.2. Coefficients Z, R_t, and A_i are defined in Notification No. 1793 of Ministry of Construction, issued in 1980.

4.2.2 Earthquake Performance Criteria

The super–structure of the building shall satisfy the followings at the design earthquake load;

(1) No flexural yielding shall occur in structural members, and

(2) Story drift angle at each story shall be less than 1/200 rad.

4.4 Nonlinear Incremental Lateral Load Analysis

4.4.1 Modeling of Building

(1) A building structure may be idealized as a series of plane frames in a principal direction if the effect of torsion and transverse frames can be neglected.

(2) If the effect of torsion and transverse frames cannot be neglected, a building must be analyzed as a three dimensional structure. If the three-dimensional effect can be considered in a plane structural model, such plane structural model may be used.

(3) A structure shall, as a general rule, be analyzed including the super-structure, basement and foundation structure.

4.4.2 Lateral Load Distribution

The distribution of lateral loads shall be the same as the one assumed in the serviceability limit state design (Section 4.2.1); the distribution in the basement shall meet the requirements of Notification No. 96 of Bureau of Housing.

4.4.3 Lateral Loading Analysis

(1) Horizontal loads shall be assumed to act at the floor level of each floor.

(2) The analysis may be terminated when the maximum story drift angle of a story reaches the design proof drift angle R_{u2}.

4.4.4 Modeling of Structural Members

(1) A column and girder shall be represented by a line member considering the following deformations:

Column: Bending, shear and axial deformations,

Girder: Bending and shear deformations.

(2) A beam-column connection may be assumed to deform in shear, or to be rigid in a region specified in Commentary of Article 8.2 in "AIJ Standard for Structural Calculation of Reinforced Concrete Structures."

(3) Inelastic deformation of a column and girder may be assumed to concentrate at the member end, represented by rotation of a rigid-plastic rotational spring.

(4) Shear, flexural and axial deformations of a structural wall shall be included.

4.5 Stiffness and Strength of Structural Members

(1) Restoring Force Characteristic Model

Stiffness change at cracking and flexural yielding shall be considered in restoring characteristics of a member.

(2) Ultimate Resistance of Member

Ultimate resistance of a member shall be evaluated by using the material strength specified in Chapter 3.

(2) Flexural yielding shall not take place at the location and region where yield hinges are not permitted in Section 1.5,

(3) Shear and flexural strength of members, that yield at one or both ends at the design proof deformation, shall be greater than corresponding action of the member magnified by the amplification factors specified in Table 4.3.2.

(a) The bending moment and shear actions of a structural wall shall be taken from Figs. 4.3.1 and 4.3.2, respectively.

(b) Shear action in a girder yielding at one end shall be calculated by assuming simultaneous yielding at the two ends and gravity loads.

(c) Bending strength shall be examined at a member end where flexural yielding does not take place at the design proof deformation.

(d) Bond resistance of longitudinal reinforcement in a girder and column shall be examined for the reinforcement actually arranged in the member.

(4) Shear and flexural strength of members, that does not yield at either end at the design proof deformation, shall be greater than corresponding action of the member magnified by the amplification factors specified in Table 4.3.3.

(a) The bending moment and shear actions of a structural wall shall be taken from Figs. 4.3.1 and 4.3.2, respectively.

(b) Shear action in a girder yielding shall be calculated by assuming simultaneous yielding at the two ends and gravity loads.

(c) Bond resistance of longitudinal reinforcement in a girder and column shall be examined for the reinforcement actually arranged in the member.

(5) Limit of Column Axial Load

Column axial load, calculated for all earthquake loading directions, shall be greater than (3/4) Nt in tension, and less than (2/3) Nu, where,

$$Nu = Ac\ Fc \qquad (4.6)$$

$$Nt = Ag\ fy \qquad (4.7)$$

Ac: column cross sectional area, Fc: specified concrete strength, Ag: gross sectional area of column longitudinal reinforcement, and fy: material strength of column longitudinal reinforcement.

4.3.3 Design of PCa Connections

Design actions at a PCa connection shall be evaluated by the existing forces calculated at design proof deformation magnified by the amplification factors specified in Section 4.3.2. Design of PCa connections shall conform to the provisions of "Design Manual for PCa Connections."

Beam: Bending and shear deformations.

(2) A beam–column connection may be assumed to be rigid in a region. If a deep girder is connected at the connection, shear deformation shall be considered in the connection. The rigid zone at a column and girder end shall be determined as specified in Commentary of Article 8.2 of "AIJ Standard for Structural Calculation of Reinforced Concrete Structures."

(3) Shear, flexural and axial deformations of a structural wall shall be included in the model.

5.3.3 Stiffness of Structural Members

(1) The stiffness of a column, girder and structural wall shall, as a general rule, be linearly elastic.

(2) The effect of orthogonal elements shall be considered in the flexural stiffness of a member.

5.3.4 Stiffness Reduction of Structural Members

(1) In structural members, subjected to high local stresses in the linear analysis, the elastic stiffness may be reduced.

(2) Those members whose stiffness may be reduced shall be limited to short–span girders and girders connected to a structural wall.

5.3.5 Structural Walls with Opening

(1) In a structural wall with an opening, the stiffness shall be properly reduced.

(2) The allowable location of an opening in a structural wall shall be specified in Chapter 7.

5.4 Ultimate Limit State Design

Super–structure shall satisfy the conditions (1) and (2) listed below.

(1) Lateral Load Resisting Capacity

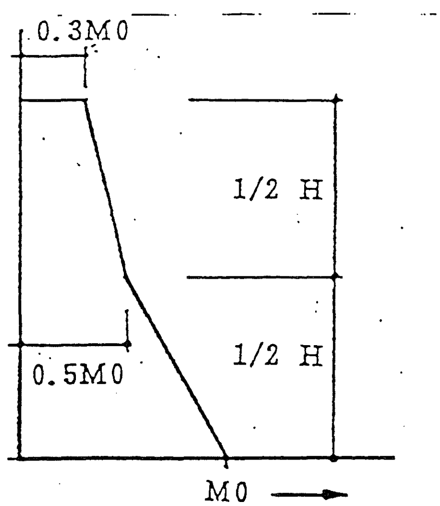
The lateral load resisting capacity of a structure shall be larger than the required lateral load resisting capacity. The required lateral load resisting capacity Quni of story i is defined by Eq. (5.4.1):

$$Q_{uni} = C_{uni}\, W_i \tag{5.1}$$

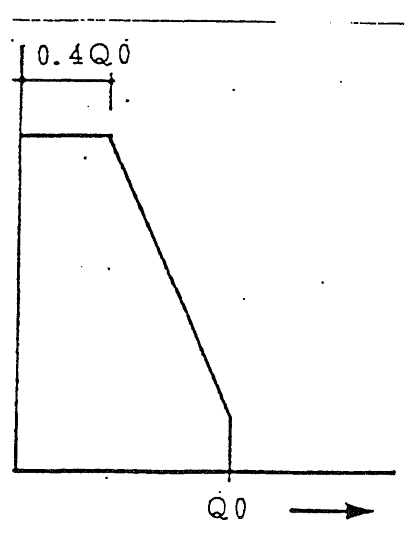
$$C_{uni} = Z\, R_t\, A_i\, C_{uns} \tag{5.2}$$

where, Cuni: Story shear coefficient for ultimate limit state design of story i, Wi: Sum of dead and live (corresponding to earthquake load) loads supported at story i, Cuns: standard base shear coefficient for ultimate limit state design, and shall be not less than the value specified below:

for $0.0 < bs < 0.3$, Cuns = 0.30



Mo: Base bending moment calculated at design proof deformation;
H: Total height of a structural wall.
Fig. 4.3.1: Bending moment in a structural wall



Qo: Maximum story shear calculated at design proof deformation.
Fig. 4.3.2: Story shear in a structural wall

Table 5.6.1: Amplification factors of design actions in member yielding at one or both ends

Member	Actions	Amplification Factors
(a) Girders	Bending	a_1
(b) Columns	Bending	$a_2 + 0.1$
	Shear	a_3
(c) Walls	Bending	a_4
	Shear	a_5

a_1 to a_5 are the same as ones defined in Table 4.3.2.

Table 5.6.2: Amplification factors of design actions in member without yielding

Member	Actions	Amplification Factors
(a) Columns	Bending	$a_2 + 0.1$
	Shear	a_3
(b) Walls	Bending	a_4
	Shear	a_5
(c) Beam–column Joint	Shear	a_6

a_2 to a_6 are the same as ones defined in Table 4.3.3.

5.6.2 Limitation of Column Axial Load

Column axial load N, calculated at the formation of collapse mechanism, shall stay within the region specified below:

$$(3/4)\ Nt < N < (2/3)\ Nu \qquad (5.3)$$

where, Nu = Ac Fc, Nt = Ag fy, Ac: column cross sectional area, Fc: specified concrete strength, Ag: gross sectional area of column longitudinal reinforcement, and fy: material strength of column longitudinal reinforcement.

5.6.3 Design of PCa Connection

Design actions at a PCa connection shall be evaluated by the existing forces calculated at the formation of collapse mechanism magnified by the amplification factors specified in Section

CHAPTER 5 EARTHQUAKE RESISTANT DESIGN (ELASTIC ANALYSIS PROCEDURE)

5.1 Design Principles

5.1.1 Scope of Buildings

This chapter may be used for the design of buildings less than 31 m in height.

5.1.2 Serviceability and Ultimate Limit State Design

The earthquake resistant design of super–structure of a building shall be examined for serviceability and ultimate limit states. The serviceability limit state design examines the performance criteria, specified in Section 5.2, by a linearly elastic analysis. The ultimate limit state design examines the performance criteria, specified in Section 5.3, at the formation of a collapse mechanism.

5.2 Serviceability Limit State Design

5.2.1 Design Earthquake Loads

Design earthquake load shall be the same as the one specified in 4.2.1.

5.2.2 Performance Criteria

The super–structure of a building shall satisfy the following conditions under the design earthquake load;

(1) No flexural yielding shall develop in structural members, and

(2) Story drift angle at each story shall not exceed the following limiting values;

for $0.00 < b_s < 0.30$: 1/600 rad

for $0.30 < b_s < 0.70$: 1/800 rad

for $0.70 < b_s < 1.00$: 1/1000 rad

where, b_s: the ratio of base overturning moment resisted by structural walls under design earthquake load.

5.3 Linearly Elastic Analysis

5.3.1 Modeling of Building

A building structure may be idealized as specified in Section 4.4.1.

5.3.2 Modeling of Structural Members

(1) A column and beam shall be represented by a line member considering the following deformations:

Column: Bending, shear and axial deformations,

CHAPTER 6 DESIGN OF FOUNDATION AND BASEMENT

6.1 Method of Design

6.1.1 Design Principle

Foundation and basement structures shall be designed to satisfy the requirement of Chapter 1 and the performance criteria specified in Section 2.1 for the loading under gravity, high winds, earthquakes and snow falls.

Foundation and basement structures shall be designed with care if special design conditions, such as ground settlement and liquefaction, need to be considered.

6.1.2 Design Actions

Design stresses for foundation and basement structures shall be determined by (a) and (b) below;

(1) Design stresses for basement structures shall include the effect of soil and hydraulic pressure in addition to the effect of long-term gravity load and short-term snow, wind pressure and earthquake loads. Stresses caused by uneven settlement and lifting or deflection of the soil and piles shall be considered if appropriate.

(2) Design stresses for foundation structures shall include the effect of surrounding soil in addition to the effect of long-term gravity load and short-term snow load and wind pressure and earthquake loads. Stresses caused by uneven settlement and lifting or deflection of the soil and piles shall be considered if applicable.

6.1.3 Design of Structural Members

(1) Design of structural members in the foundation and basement shall satisfy (a) to (d) below:

(a) Design stresses due to the long-term gravity load and short-term snow load and wind pressure shall not exceed allowable stresses of materials specified in Section 3.3.

(b) At an earthquake load level for the ultimate limit state of the super-structure, no flexural yielding nor shear failure shall, as a general rule, occur in structural members in the foundation and basement. The location and members, where flexural yielding is permitted at the ultimate limit state, shall not yield at the serviceability limit state.

(c) Axial force in piles at the long-term gravity loading shall not exceed the allowable vertical bearing capacity of the soil for the long-term loading, and stress in the pile shall not exceed the allowable stress of the pile for the long-term loading. Axial force in piles at an earthquake load level for the serviceability limit state of the super-structure and the short-term snow and wind loading shall not exceed the allowable bearing capacity of soil for the short-term loading, and stresses in piles shall not exceed the allowable stresses of piles for the short-term loading.

At an earthquake load level for the ultimate limit state of the super-structure, axial force in piles shall not exceed the ultimate bearing capacity of soil, and the piles shall not, as a general rule, develop flexural yielding nor shear failure.

(d) Ground contact pressure of direct foundation under the long-term gravity loading shall

for 0.3 < bs < 0.7, Cuns = 0.35
for 0.7 < bs < 1.0, Cuns = 0.40

where, bs: the ratio of overturning moment resisted by structural walls at the level of first story floor to the total overturning moment evaluated at the formation of collapse mechanism.

(2) Collapse Mechanism

In order to assure the formation of a total collapse mechanism of the structure, the locations and members of planned yield hinges and also the locations and members not allowed to form yield hinges shall be designed for the action at the formation of collapse mechanism magnified by the amplification factor of design member action.

5.5 Ultimate Strength of Members

The ultimate strength of members shall be evaluated as specified in Section 4.5.2.

5.6 Amplification Factors of Design Member Action

5.6.1 Amplification Factors

The amplification factors of design member actions shall satisfy either (1) or (2) below.

(1) Members with Planned Yield Hinge

(a) The amplification factors for each design member action shall be taken from Table 5.6.1.

(b) Design bending moment shall be used for a region other than the yield hinge.

(c) Design bond stress shall be calculated using the amplified actions.

(2) Members without Planned Yield Hinge

(a) The amplification factors for each design member action shall be taken from Table 5.6.2.

(b) Ultimate flexural strength of columns without a planned yield hinge shall be evaluated using the specified yield strength of longitudinal reinforcement.

(c) Ultimate flexural strength of walls without a planned yield hinge shall be evaluated using the specified yield strength of longitudinal reinforcement.

(d) Shear strength of members shall be evaluated using the specified yield strength of transverse reinforcement.

(e) Design bond stress shall be calculated using the amplified actions.

6.2.4 Soil beneath Foundation Slab

Foundation slab of direct foundation shall rest on stable soil which shall not result in volume change or liquefaction under gravity and earthquake loading.

6.3 Design of Pile Foundation

6.3.1 Principles

(a) Vertical load capacity of pile foundation shall, as a general rule, be axial load bearing capacity of the pile itself.

(b) Design force for pile foundation shall be horizontal and vertical actions transmitted from the floor immediately above the foundation and the load acting on the foundation. Impact, cyclic, eccentric, and inclined loads shall be included if applicable.

(c) If the ground may become unstable due to ground settlement, lateral movement and liquefaction during an earthquake, the effect shall be considered in design.

6.3.2 Design for Vertical Loads

(a) Vertical force on a pile due to the long–term loads, specified in Section 2.2.2, shall not exceed the allowable bearing force of the pile for the long–term loading. If pull–out force acts on a pile under the long–term loading, the force shall not exceed the allowable pull–out force of the pile for the long–term loading.

(b) Vertical force on a pile due to loads for the short–term loading by the snow and wind pressure, specified in Sections 2.2.4 and 2.2.5, shall not exceed the allowable bearing force of the pile for the short–term loading. If pull–out force acts on a pile under the short–term loading, the force shall not exceed the allowable pull–out force of the pile for the short–term loading.

(c) Vertical force of a pile at an earthquake load level for the serviceability limit state of the super–structure shall not exceed the allowable bearing force of the pile for the short–term loading. If pull–out force acts on a pile, the force shall not exceed the allowable pull–out force of the pile for the short–term loading.

(d) Vertical force on a pile at an earthquake load level for the ultimate limit state of the super–structure shall not exceed the bearing capacity of the pile. If pull–out force acts on a pile, the force shall not, as a general rule, exceed the ultimate pull–out strength of the pile. The vertical force caused by maximum earthquake loading in any direction shall not exceed the bearing capacity of the pile.

6.3.3 Design for Horizontal Loads

(a) If a horizontal force acts on a pile under the long–term loading, the stress developed in the pile shall not exceed the allowable stresses of materials for the long–term loading.

(b) Stress in a pile under the short–term loading by snow and wind pressure as well as under earthquake loads for the serviceability limit state of the super–structure shall not exceed the allowable stresses of materials for the short–term loading.

(c) At an earthquake load for the ultimate limit state of the super–structure, a pile shall, as a general rule, be provided with a required horizontal strength against shear failure in the pile and

5.6.1. Design of PCa connections shall conform to the provisions of "PRESSS Design Manual for PCa Connections."

(e) Stresses developed in a foundation slab, described in (a) to (d) above, shall be transmitted to foundation girders.

(3) If PCa members are to be used in foundation slab, the PCa members shall be ensured to develop structural performance specified in Section 2.1.2. The action at the PCa connection shall be estimated under different loadings, and PCa connections shall be designed in accordance with "PRESSS Design Manual for PCa Connection."

6.4.2 Design of Foundation Girder

(1) Design stresses for a foundation girder shall be calculated for stresses from ground bearing pressure in direct foundation and actions in pile in pile foundation, in addition to stresses transmitted from connecting columns and structural walls. Stresses due to soil and hydraulic pressure and due to out-of-plane actions by piles shall be considered.

(2) Design of a foundation girder shall satisfy (a) to (d) below:

(a) Stresses in a foundation girder under the long-term loading shall not exceed the allowable stresses of materials, specified in Section 3.3, for the long-term loading.

(b) Stresses in a foundation girder under the short-term loading by the snow load and wind pressure shall not exceed the allowable stresses of materials, specified in Section 3.3, for the short-term loading.

(c) At an earthquake load level for the ultimate limit state of the super-structure, a foundation girder shall not, as a general rule, yield in flexure nor fail in shear and bond splitting. If a foundation girder is permitted to yield at an earthquake load level for the ultimate limit state of the super-structure, the girder shall not yield under the serviceability limit state.

(3) If PCa members are to be used in a foundation girder, the PCa members shall be ensured to develop structural performance specified in Section 2.1.2. The action at the PCa connection shall be estimated under different loadings, and PCa members shall be designed in accordance with "PRESSS Design Manual for PCa Connection."

6.4.3 Connection of Foundation Slab with Foundation Girder

Connection between foundation slab and foundation girders shall be provided with sufficient rigidity and strength to transfer the action developed in the foundation slab to the foundation girder.

6.5 Design of Members in Basement

(1) Design for structural members in basement shall satisfy (a) to (d) below:

(a) Stresses in members in basement due to the long-term loading shall not exceed the allowable stresses of materials, specified in Section 3.3, for the long-term loading.

(b) Stresses in members in basement due to the short-term loading by the snow load and wind pressure shall not exceed the allowable stresses of materials, specified in Section 3.3, for the short-term loading.

not exceed the allowable bearing pressure of the soil for the long–term loading; ground contact pressure at an earthquake load level for the serviceability limit state of the super–structure and under the short–term snow and wind pressure shall not exceed the allowable bearing stress of soil for the short–term loading.

At an earthquake load level for the ultimate limit state of the super–structure, the bearing pressure shall not exceed the bearing capacity of soil.

(2) The reduction of vertical and horizontal load resistance in sandy soil and clay soil below the ground water level shall be properly considered under earthquake loading.

6.1.4 Examination of Foundation Uplifting

Foundation shall not be uplifted under the earthquake load level for the serviceability limit state of the super–structure.

6.2 Design of Direct Foundation

6.2.1 Design for Ground Contact Pressure

(a) Ground contact pressure due to loads for the long–term loading, specified in Section 2.2.2, shall not exceed the allowable bearing stress of soil for the long–term loading.

(b) Ground contact pressure due to loads for the short–term loading by snow load and wind pressure, specified in Sections 2.2.4 and 2.2.5, shall not exceed the allowable bearing stress of soil for the short–term loading.

(c) Ground contact pressure at an earthquake load level for the serviceability limit state of the super–structure shall not exceed the allowable bearing stress of soil for the short–term loading.

(d) Ground contact pressure at an earthquake load level for the ultimate limit state of the super–structure shall not exceed the bearing capacity of soil. Ground contact pressure caused by maximum earthquake loading in any direction shall not exceed the bearing capacity of soil.

6.2.2 Allowable Bearing Stresses and Ultimate Bearing Strength

(1) The allowable bearing stress for direct foundation shall be determined not to exceed the allowable bearing capacity of soil, and not to cause uneven settlement which affects the serviceability of a structure.

(2) The ultimate bearing stress for direct foundation shall be determined not to cause excessive tilting of a structure due to settlement, not to cause yielding in members other than the planned members, and not to cause brittle failure, such as shear and bond splitting failure, of foundation and principal structural members.

6.2.3 Design for Horizontal Loading

If horizontal force acts on the lower face of the direct foundation, the safety against sliding shall be examined.

CHAPTER 7 STRUCTURAL REQUIREMENTS

7.1 General Requirements

Nominal bar size, spacing, clearance, cover depth, standard bend of reinforcement, if not specified in this guidelines, shall conform to "Building Standard Law Enforcement Order," "Architectural Institute of Japan Standard for Structural Calculation of Reinforced Concrete Structures, Japan Architectural Standard Specification (JASS) and its Commentary on Reinforced Concrete Work (JASS–5)," "Reinforcement Arrangement Guidelines for Reinforced Concrete Structures and its Commentary," and "PRESSS Guidelines for Construction and Quality Control of PCa Construction."

7.2 Columns

(1) Column Dimensions

The shorter dimension of a column section shall be not less than 40 cm. A ratio of section dimensions of long side to short side shall be not more than 2.

(2) Longitudinal Reinforcement

(a) Longitudinal reinforcement shall be deformed bars of size equal to or larger than D19.

(b) Gross reinforcement ratio of longitudinal reinforcement shall be not less than 0.008.

(3) Lateral Reinforcement

Lateral reinforcement shall be deformed bars of size equal to or larger than D10. Lateral reinforcement shall be arranged to effectively confine the longitudinal reinforcement and concrete. The spacing of lateral reinforcement shall satisfy the values specified in Table 7.2.1.

Table 7.1: Minimum Spacing of Column Lateral Reinforcement
(Unit: mm)

Special yield hinge	Yield hinge	Non–yield hinge
Spacing not more than yield hinge region Use sub–ties All long. bars be supported[3)]	(1) D / 5 (2) 150 (3) 6 d_b Use sub–ties[1)] Intermediate long. bars be supported[2)]	(1) D / 3 (2) 200 (3) 8 d_b

where, d_b: size of longitudinal reinforcement in mm.

at the connection to the pile cap. Excessive horizontal deflection of a pile shall not occur. If flexural yielding is permitted in a pile under an earthquake loading level at the ultimate limit state of the super–structure, required horizontal resistance shall be maintained in the pile, and shear failure shall not occur in the pile and at the connection to the pile cap.

6.3.4 Connection of Piles

(1) Connection of a pile to a pile cap and to a foundation girder shall be designed by the same criteria as the pile foundation.

(2) Splicing of a pile shall be provided with resistance sufficient to transmit actions developed at the locations.

6.4 Design of Foundation Slab and Girder

6.4.1 Principles

(1) Design of a foundation slab for direct foundation shall satisfy (a) to (e) below:

(a) Stresses in a foundation slab due to ground bearing pressure under the long–term loading shall not exceed the allowable stresses of materials, specified in Section 3.3, for the long–term loading.

(b) Stresses in a foundation slab due to ground bearing pressure under the short–term loading by the snow load and wind pressure shall not exceed the allowable stresses of materials, specified in Section 3.3, for the short–term loading.

(c) At an earthquake load level for the ultimate limit state of the super–structure, a foundation slab shall not yield in flexure nor fail in a brittle manner, such as in shear, due to ground bearing pressure.

(d) A foundation slab shall not yield in flexure nor fail in a brittle manner, such as in shear, due to ground bearing pressure caused by maximum earthquake loading in any direction.

(e) Stresses developed in a foundation slab, described in (a) to (d) above, shall be transmitted to foundation girders.

(2) Design of a foundation slab for pile foundation and foundation slab shall satisfy (a) to (e) below:

(a) Stresses in a foundation slab due to the action in pile under the long–term loading shall not exceed the allowable stresses of materials, specified in Section 3.3, for the long–term loading.

(b) Stresses in a foundation slab due to the action in pile under the short–term loading by the snow load and wind pressure shall not exceed the allowable stresses of materials, specified in Section 3.3, for the short–term loading.

(c) At an earthquake load level for the ultimate limit state of the super–structure, a foundation slab shall not yield in flexure not fail in a brittle manner, such as in shear and punching shear, due to the action in pile.

(d) A foundation slab shall not yield in flexure not fail in a brittle manner, such as in shear and punching shear, due to the action in pile caused by maximum earthquake loading in any direction.

(2) Longitudinal Reinforcement

(a) Longitudinal reinforcement shall be deformed bars of size equal to or larger than D16.

(b) Area ratio of compressive to tensile reinforcement shall be not less than 0.5.

(c) Tensile reinforcing bars shall, as a general rule, be placed in not more than two layers in the section.

(3) Anchorage of Second–Layer Longitudinal Reinforcement

The cut–off location of beam longitudinal reinforcement from the critical section shall be determined considering the anchorage length sufficient for the stress in the longitudinal reinforcement to be safely transferred by bond.

(4) Lateral Reinforcement in Beam

Lateral reinforcement in beam shall be deformed bars of size equal to or greater than D10. Lateral reinforcement shall be arranged to effectively confine the longitudinal reinforcement and concrete. The spacing of lateral reinforcement shall satisfy the values specified in Table 7.3.1.

Table 7.3.1: Minimum Spacing of Beam Lateral Reinforcement
(Unit: mm)

Yield hinge region	Non–yield hinge region
(1) D / 3	(1) D / 3
(2) 200	(2) 300
(3) 8 d_b	

where, d_b: size of longitudinal reinforcement in mm, D: depth.

(5) Yield Hinge Region

A yield hinge region is defined as a region where flexural yielding takes place, and shall be equal to 1.5 times beam depth from the orthogonal column face.

(6) Small Openings

(a) The diameter of an opening shall be not more than one–third of the beam depth.

(b) Center–to–center spacing of two adjacent openings shall be more than three times the diameter of the larger opening.

(c) The distance from the column face to the edge of an opening shall be, as a general rule, more than the beam depth. However, the distance requirement may not be satisfied if the safety is proven by a special study, such as experiment.

7.4 Structural Walls

(1) Sectional Shape

(a) A structural wall, as a general rule, shall be provided with boundary columns at both

(c) At an earthquake load level for the ultimate limit state of the super–structure, structural members shall not, as a general rule, yield in flexure nor fail in shear and bond splitting modes.

(d) Axial force in a column under earthquake loading in any direction shall be less than 2/3 Nu (Nu: compressive strength of column), and greater than 3/4 Nt (Nt: tensile strength of column).

(2) If a PCa member is to be used in a part of basement to contact the ground, the PCa member shall satisfy (1) above, and also shall be ensured to develop structural performance speci–fied in Section 2.1.2. PCa connections shall be designed in accordance with "PRESSS Design Manual for PCa Connection."

story wall base. However, the height of a yield hinge region may not be taken higher than the bottom face of the third floor beam.

(9) Special Yield Hinge Region

A special yield hinge region is defined as a yield hinge region where axial force N_w at the design proof deformation in Chapter 4 or at the formation of the collapse mechanism in Chapter 5 falls in a region specified in Eq.(7.4.1).

$$2/3\, A_{core}\, F_c - A_{ws}\, f_{ws} < N_w < A_{core}\, F_c - A_{ws}\, f_{wy} \qquad (7.4.1)$$

where, A_{core}: core area of a compressive side boundary column, A_{ws}: total area of longitudinal reinforcement in wall panel, F_c: specified compressive strength of concrete, f_{ws}: material strength of longitudinal reinforcement in wall panel.

(10) Lateral Reinforcement in Special Yield Hinge Region

Lateral reinforcement of a boundary column within a special yield hinge region shall satisfy the requirements in Section 7.2 (3) for spacing, and Section 7.2 (6) for detailing.

7.5 Beam–Column Connections

(1) Lateral Reinforcement

Lateral reinforcement ratio in a beam–column connection shall be not less than 0.003. Spacing of lateral reinforcement shall satisfy the values specified in Table 7.5.1.

Table 7.5.1: Minimum Spacing of Lateral Reinforcement in Beam–column Connection (unit: mm)

Deformed bar D10[1)]	150
Deformed bar beyond D10[2)]	200 and 8 d_b

where, d_b: nominal diameter of longitudinal reinforcement.

Note 1): including high strength bars of nominal diameter of not less than 6 mm and less than 11 mm;

2): including high strength bars of nominal diameter of not less than 11 mm.

(2) Anchorage of Beam and Column Reinforcement

(a) Anchorage Method

Beam longitudinal reinforcement shall, as a general rule, pass through or anchored with 90–degree bend in the column core of a beam–column connection. Beam longitudinal reinforcement in a yield hinge region shall be placed inside the column longitudinal reinforcement. Column

1) Lateral reinforcement placed on intermediate longitudinal reinforcement.

2) Intermediate longitudinal reinforcement, placed more than 300 mm apart, shall be laterally supported by a corner of closed shape lateral reinforcement or 135 degree bend.

3) All longitudinal reinforcement, as a general rule, shall be laterally supported by a corner of closed shape lateral reinforcement or 135-degree bend. However, longitudinal reinforcement, within 200 mm between the two adjacent supported longitudinal reinforcement, may not be supported.

(4) Yield Hinge Region

A region, where flexural yielding may take place at a yield hinge, shall be equal to 1.5 times column depth from the orthogonal beam face.

(5) Special Yield Hinge Region

A special yield hinge region is defined as a yield hinge region where design axial force N_c at the design proof deformation specified in Chapter 4 or at the formation of a collapse mechanism specified in Chapter 5 fall in a region of Eq.(7.1.1).

$$1/3\, A_c F_c < N_c \qquad (7.1.1)$$

where, Ac: column sectional area, and Fc: specified concrete compressive strength.

(6) End of Lateral Reinforcement in Special Yield Hinge Region

End of lateral reinforcement within a special yield hinge region shall conform to (a) to (c) below where d_b: bar diameter of lateral reinforcement;

(a) The end of lateral reinforcement other than closed shape welded lateral reinforcement and spiral reinforcement shall be anchored with 135-degree hook, and with extension of more than 8 d_b,

(b) The end of lateral reinforcement other than of closed shape shall be bent more than 135 degrees. The extension shall be more than 8 d_b for 135 degree bend, and more than 4 d_b for 180-degree bend.

(c) The end of spiral reinforcement shall be anchored into the confined core concrete with 1-1/2 extra turns. The end of spiral reinforcement shall be provided with hooks of 135-degree bend with extension of more than 8 d_b or hooks of 90-degree bend with extension of more than 14 d_b.

7.3 Beams and Girders

This section specifies requirements for beams and girders other than sub-beams.

(1) Sectional Shape

Width of a beam shall be not less than 25 cm. Width of a beam in a yield hinge region shall be not less than one-quarter of beam depth.

(c) A foundation girder shall be, as a general rule, cast–in–situ reinforced concrete construction. If requirements in Section 6.4.2 (c) are satisfied, PCa members may be used for the foundation girder.

(2) Reinforcement in foundation girders shall be determined by structural calculation and shall satisfy (a) to (d) below:

(a) Longitudinal reinforcement shall be placed at the top and bottom of a section.

(b) Longitudinal reinforcement shall be, as a genral rule, placed not more than two layers at the top and at the bottom of a section.

(c) Spacing of longitudinal reinforcement in the vertical direction shall be, as a genral rule, not more than 300 mm. The amount of longitudinal reinforcement in a foundation girder supporting a structural wall shall be provided sufficient to resist the tensile strength of horizontal wall reinforcement.

(d) Spacing of lateral reinforcement shall, as a genral rule, be not more than 300 mm. The amount of lateral reinforcement in a foundation girder supporting a structural wall shall be provided sufficient to resist the tensile strength of wall vertical reinforcement.

(3) Reinforcement and structure of a foundation slab shall conform to the requirements of Chapter 6.4 and (a) to (c) below:

(a) Size and depth of a foundation slab shall be determined taking into consideration soil bearing pressure, pile size and number, and reaction from piles.

(b) Reinforcement in a foundation slab shall be determined by design calculation, and shall satisfy the minimum reinforcement specified in "AIJ Standard for Structural Calculation of Reinforced Concrete Structures."

(c) A foundation slab shall be, as a general rule, cast–in–situ reinforced concrete construction. If the requirements in Section 6.4.1 (3) are satisfied, PCa members may be used for the foundation slab.

edges.

(b) Thickness of a structural wall shall be not less than 150 mm, and not less than 0.05 times clear story height.

(2) Reinforcement

(a) Reinforcement shall be deformed bars of size not less than D10.

(b) Reinforcement shall be placed in double layers in a yield hinge region.

(3) Opening

An opening shall not be placed, as a general rule, in a yield hinge region. If an opening is to be placed in a non-yield hinge region, the opening shall be placed near the center portion of the wall span. The size of the opening shall be selected not to alter the structural characteristics of the structural wall.

(4) Minimum Reinforcement

Lateral reinforcement ratio shall be not less than 0.0025. The ratio shall be not less than 0.003 in a yield hinge region.

(5) Lateral Reinforcement

The spacing of vertical reinforcement shall be not more than 300 mm, and the spacing of lateral reinforcement shall satisfy the values specified in Table 7.4.1.

Table 7.4.1: Minimum Spacing of Wall Lateral Reinforcement
(Unit: mm)

Yield hinge region	Non-yield hinge region
200	300

(6) Sub-Ties

Sub-ties using deformed bars of size not less than D10 shall be placed at 300 mm spacing over 1/5 clear span and over the entire height within a yield hinge region.

(7) Anchorage of Wall Reinforcement

All wall reinforcement, as a general rule, shall be anchored either in boundary columns or in boundary beams.

(8) Yield Hinge Region

A yield hinge region is defined as a region where flexural yielding takes placed and shall be equal to the larger of 1/6 times the wall height or the horizontal wall dimension from the first

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longitudinal reinforcement, except at the base of the base story and at the top of the highest story, shall pass through a beam–column joint. The anchorage length shall be measured from the column face for beam longitudinal reinforcement, and from the beam face for column longitudinal reinforcement.

(b) Anchorage with Bend

If beam reinforcement is to be anchored with bend, horizontal lead length of longitudinal reinforcement shall be not less than 16 times the nominal bar diameter, and shall be sufficient to develop design anchorage force. The angle of bend shall be 90 degrees, and the bend shall start outside the column center line.

The extension shall be not less than 10 times nominal bar diameter placed within the beam–column joint.

(c) Bars Passing through Connection

If yield hinges are formed at the both faces of a beam–column connection at the Design Proof Deformation (at the formation of a collapse mechanism in Chapter 5), and if beam or column reinforcement passes through the connection, the column width (beam depth) shall, as a general rule, be not less than 25 times the nominal bar diameter.

7.6 Slabs and Sub–Beams

(1) Thickness of floor and roof slabs shall be not less than 130 mm.

(2) Slab Reinforcement

Reinforcing bars in slab shall be deformed bars of size not less than D10 or slab reinforcement shall be welded wire mesh of nominal diameter not less than 6 mm.

(3) Slab reinforcement ratio in both directions shall, as a general rule, be not less than 0.002 of concrete cross section.

(4) Full PCa slabs and half PCa slabs shall, as a general rule, satisfy the performance criteria on vibration characteristics and deformation of RC slabs..

(5) Thickness of cast–in–situ concrete in half PCa slabs shall be not less than 70 mm.

(6) Design of a connection between PCa slabs and a connection between a PCa sub–beam and its support shall conform to "PRESSS Design Manual for PCa Connection."

(7) PCa slab shall, as a general rule, be supported along an edge over not more than 30 mm.

(8) Minimum reinforcement ratio in a sub–beam shall conform to the requirements of "AIJ Standard for Structural Calculation of Reinforced Concrete Structures."

7.7 Foundation Girders and Foundation Slabs

(1) Structural requirements for foundation girders shall conform to the requirements of Section 6.4 and (a) to (c) below:

(a) Base of bottom story columns and structural walls, as a general rule, shall be connected effectively by foundation girders.

(b) Width of a foundation girder shall be not smaller than the thickness of a connected structural wall, and shall be, as a general rule, not smaller than the depth of a connected column.

CHAPTER 8 NON-STRUCTURAL ELEMENTS

8.1 Method of Design

(1) If a reinforced concrete or similar non–structural wall is to be installed in a structure, structural joints shall, as a general rule, be place along appropriate edges to separate the non–structural wall from structural members to ensure the structural performance at the ultimate limit state specified in Chapters 4 and 5.

(2) Non–structural elements shall be provided with strength sufficient to resist inertia force developed by an earthquake motion, and shall follow the structural deformation.

8.2 Connection of Non–Structural Elements

(1) Structural joints for non–structural walls shall, as a general rule, be either of a complete separate type or of a shear failure type.

(2) Stresses developed in a non–structural element and its connection by the inertia force of an earthquake motion shall be less than the allowable stress of the material for the short–term loading.

(3) A non–structural element and its connection shall not fail nor fall by the forced deformation expected at the ultimate limit state design.

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