Earthquake Damage and Seismic Code for Buildings in Japan

Yuji Ishiyama, Dr. Eng.

Professor Emeritus, Hokkaido University, to-yuji@nifty.com

Summary

Major earthquakes that have forced to make development and modifications to seismic code are shown. The current seismic code for buildings in Japan includes allowable stress calculation using linear analysis and ultimate lateral capacity calculation using nonlinear analysis. The code consists of six seismic design routes from $\boxed{0}$ to $\boxed{5}$ and each design route is a sequence of some designated structural requirements among (a) to (j).

1 Introduction

Many big earthquakes have occurred in and around Japan since ancient times and have caused severe damage, but the scientific research on seismology or earthquake engineering began only after the Meiji Restoration in 1868. The Yokohama Earthquake occurred in 1880, and the Seismological Society of Japan was established. Since then, every big earthquake forced to make some development in earthquake resistant technology and seismic codes. The 1891 Nobi Earthquake led to the formation of the Earthquake Investigation Committee, the 1923 Great Kanto Earthquake brought about the introduction of the seismic coefficient, the 1968 Tokachi-oki Earthquake prompted the revision of shear reinforcement of reinforced concrete columns, the 1978 Miyagi-ken-oki Earthquake led to the adoption of a new seismic design method, the 1995 Great Hanshin-Awaji Earthquake made the revision of shape factor and introduction of Seismic Retrofitting Promotion Law, and the 2011 Great East Japan Earthquake brought about a new notification for tsunami evacuation buildings, etc.

This is to report the major earthquakes, development of seismic code and the current seismic code in Japan.

2 Major Earthquakes and Develpoment of Seismic Code

2.1 1880 Yokohama Earthquake and 1891 Nobi Earthquake

After the Meiji Restoration in 1868, the government of Japan abandoned the 230 year isolation from foreign countries and resumed overseas trade. The government invited many foreign researchers, scientists and engineers to Japan in order to develop Japanese industries as well as to absorb Western culture as soon as possible. The Yokohama Earthquake (M=5.5) occurred in 1880 and caused moderate damage. The earthquake was not very severe, but was strong enough to frighten resident researchers from overseas countries. In the same year, the Seismological Society of Japan was founded and scientific research on seismology and earthquake engineering began.

The 1891 Nobi Earthquake (M=7.9, 7,273 deaths) caused very severe damage. In the following year (1892), the Earthquake Investigation Committee was founded and research on seismology was accelerated. The Earthquake Investigation Committee was rechartered to the Earthquake Research Institute in 1925 which now belongs to the University of Tokyo.

2.2 1923 Great Kanto Earthquake and Seismic Coefficient

After the 1923 Great Kanto Earthquake (M=7.9, more than 105,000 deaths or missing), the provision that “the horizontal seismic coefficient shall be at least 0.1” was added in 1924 to the Urban Building Law which had been enforced since 1919.

The seismic coefficient was specified to be 0.1, because the seismic coefficient of ground surface at Tokyo was estimated to be 0.3 of the acceleration due to gravity during the Great Kanto Earthquake, and the safety factor of material strength to allowable stress was assumed to be 3.

2.3 Building Standard Law after World War II

Since the enforcement of the Urban Building Law, one allowable stress had been specified for each structural material. After World War II, the Building Standard Law replaced the old regulations in 1950. Then the concept of "permanent" (long term) and "temporary" (short term) was introduced to load combinations and allowable stresses. Since the temporary allowable stress became twice of the old allowable stress (equivalent to the permanent allowable stress), the horizontal seismic coefficient became 0.2 which is twice of the old regulations.

2.4 1968 Tokachi-oki Earthquake and Strengthening of Shear Reinforcement

In the 1968 Tokachi-oki Earthquake (M=7.9, 52 deaths), many reinforced concrete buildings suffered severe damage which had been believed to have enough earthquake resistant capacity. The Building Standard Law was revised and shear reinforcement of reinforced concrete columns was strengthened. Incidentally, the effectiveness of this revision was proven in the 1995 Great Hanshin-Awaji Earthquake.

2.5 1978 Miyagi-ken-oki Earthquake and New Seismic Design Method

The fundamental revision of the seismic design method became necessary because of the development of the earthquake response analysis technology and the damage caused by the 1968 Tokachi-oki Earthquake and other earthquakes.

A five year national research project for establishing a new seismic design method was carried out from 1972 to 1977 in Japan. The 1978 Miyagi-ken-oki Earthquake (M=7.4, 28 deaths), hit the Sendai area and its occurrence accelerated the adoption of the new seismic design method. The new seismic design method, which had been already proposed, was reviewed and evaluated for use as a practical design method. The Building Standard Law was revised in 1980, and the new seismic design method has been used since 1981.

The new seismic design method was developed introducing up-to-date knowledge of earthquake engineering at that time. The major changes were (1) the introduction of two levels of earthquake motions (severe and moderate earthquake motions), (2) simple formulae to evaluate seismic shear forces and their distribution along the height of buildings with shorter natural period as well as longer natural period, (3) seismic shear coefficient instead of seismic coefficient, (4) consideration of structural balance in plan and in elevation (story drift and shape factor to consider stiffness and eccentricity), (5) structural characteristic factor to consider strength and ductility, etc. The new seismic design method for buildings is now called as "Allowable Stress and Lateral Shear Capacity Method" and it is summarized as Routes $\boxed{1}$, $\boxed{2}$ and $\boxed{3}$ in the next section.

2.6 1995 Great Hanshin-Awaji Earthquake and Seismic Diagnoses

Although the validity of the new seismic design method was proven during the 1995 Great Hanshin-Awaji Earthquake (M=7.2, 6,430 deaths), the Building Standard Law was revised in 1998. The revision was to make minor amendments to the shape factor, and to introduce performance-based "Response and Limit Capacity Method". This method is referred as Route $\boxed{4}$ in the next section.

2.7 2011 Great East Japan Earthquake and Tsunami

The damage caused by the 2011 Great East Japan Earthquake (M=9.0, 19,000 deaths or missing) was mainly due to huge tsunami. The Bulding Standard Law has not been revised yet, but a new notification was issued for tsunami evacuation buildings.

3 Current Seismic Code for Buildings

3.1 Structural Requirements

The buildings shall satisfy the designated structural requirements accoding to structural type, floor area, height, etc. as shown in Fig.1.

(a) Structural specifications shall be fulfilled for any buildings. (Some of them can be alleviated for Route $\boxed{3}$, and most of them except for duarability can be alleviated for Routes $\boxed{4}$ and $\boxed{5}$.)

(b) Allowable stress calculation is to verify that the stress caused by the lateral seismic shear for moderate earthquake motions shall not exceed the allowable stress for temporary loads. (This is applied to Routes $\boxed{1}$, $\boxed{2}$ and $\boxed{3}$.)

(c) Height, strength, etc. limitation (along with allowable stress calculation and structural specifications) is to realize seismic safety for small scale buildings. (This is applied to Route $\boxed{1}$.)

(d) Story drift limitation is to prevent non-structural elements from earthquake damage. (This is applied to Routes $\boxed{2}$, $\boxed{3}$ and $\boxed{4}$.)

(e) Stiffness and eccentricity limitation is to prevent earthquake damage concentration to a part of buildings. (This is applied to Route $\boxed{2}$.)

(f) Strength and ductulity limitation is to give adequate strength and ductility against severe earthquake motions. (This is applied to Route $\boxed{2}$.)

(g) Ultimate lateral capacity limitation is to verify seismic performance against severe earthquake motions, calculating ultimate lateral capacity of the building. (This is applied to Route $\boxed{3}$.)

(h) Damage limit is to verify that the damage of the building shall be within the allowable level against rare earthquake motions. (This is applied to Route $\boxed{4}$.)

(i) Safety limit is to verify that the building may not collapse against very rare earthquake motions. (This is applied to Route $\boxed{4}$.)

(j) Time history response analysis is to verify the seismic safety of the building through time history response analysis. (This is applied to Route $\boxed{5}$.)

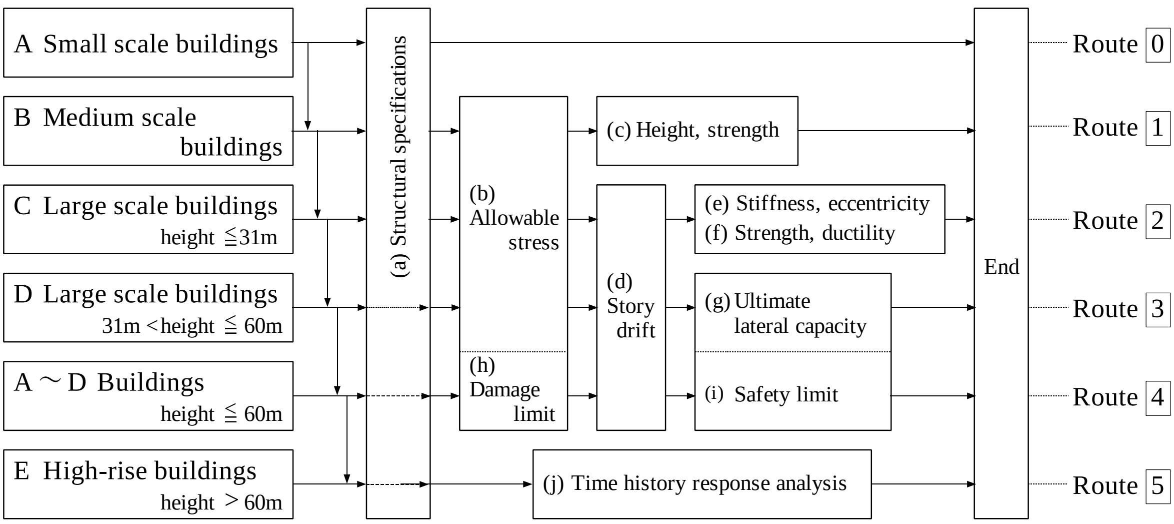


Figure 1: Design routes for buildings

3.2 Seismic Design Routes

Each sequence of some designated structural requirements among (a) to (j) forms a seismic design route, from $\boxed{0}$ to $\boxed{5}$, as follows (see Fig.1).

Route $\boxed{\mathbf{0}}$ requires no structural calculation. The seismic safety is realized by (a) structural specifictions. (This route is applied to small scale buildings.)

Route $\boxed{\mathbf{1}}$ requires (b) allowable stress calculation, and some limitations of (c) height, strength, etc. The seismic safety is realized with the supplement of (a) structural specifications. (This route is applied to small and medium scale buildings.)

Route $\boxed{\mathbf{2}}$ requires (b) allowable stress calculation, and limitations of (d) story drift, (e) stiffness and eccentricity, and (f) strength and ductility. The seismic safety is realized with the supplement of (a) structural specifications. (This route is applied to buildings, whose height does not exceed 31 m.)

Route $\boxed{\mathbf{3}}$ requires (b) allowable stress calculation, (d) story drift limitation, and (g) ultimate lateral capacity calculation. The seismic safety is realized with the supplement of (a) structural specifications (some of them can be alleviated). (This route is applied to buildings, whose height does not exceed 60 m.)

Route $\boxed{\mathbf{4}}$ requires verifications for (h) damage limit, (i) safety limit and (d) story drift limitation. The seismic safety is realized with the supplement of (a) structural specifications that can be alleviated except for durability. (This route is applied to buildings, whose height does not exceed 60 m.)

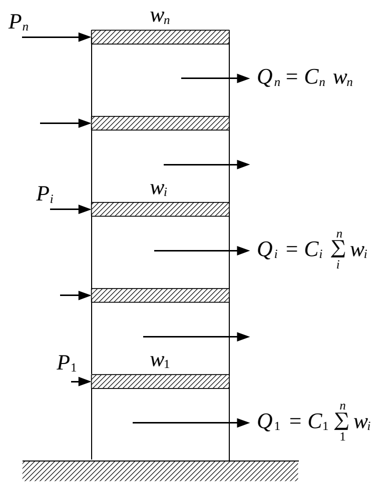
Route $\boxed{\mathbf{5}}$ requires (j) time history response analysis to verify the seismic safety, and (a) structural specifications can be alleviated except for durability. (This route is applied to all buildings, including whose height exceeds 60 m.)

3.3 Design Seismic Shears

For (b) allowable stress calculation and (g) ultimate capacity calculation, the seismic shear Q_i of the i-th story is calculated as follows (see Fig.2).

$$Q_i = C_i W_i \tag{1}$$

where, C_i is the lateral seismic shear coefficient of the i-th story that shall be determined as Eq.(2) and W_i is the weight of the building above the i-th story.



P_i: lateral seismic force of the i-th story
Q_i: lateral seismic shear of the i-th story
C_i: lateral seismic shear coefficient of the i-th story
w_i: weight of the i-th story

Figure 2: Lateral seismic force P_i and shear Q_i of the i-th story

$$C_i = Z\,R_t\,A_i\,C_0 \tag{2}$$

where, Z is the seismic hazard zoning factor (see Fig.3), R_t is the design spectral factor (see Table 1 and Fig.4) which shall be determined according to the type of soil profile (see Table 2) and the fundamental natural period T of the building, A_i is the lateral shear distribution factor (see Fig.5 a), and C_0 is the standard shear coefficient that shall be as follows.

$$\begin{aligned} &C_0 \geqq 0.2\ :\ \text{for moderate earthquake motions} \\ &C_0 \geqq 1.0\ :\ \text{for severe earthquake motions} \end{aligned} \tag{3}$$

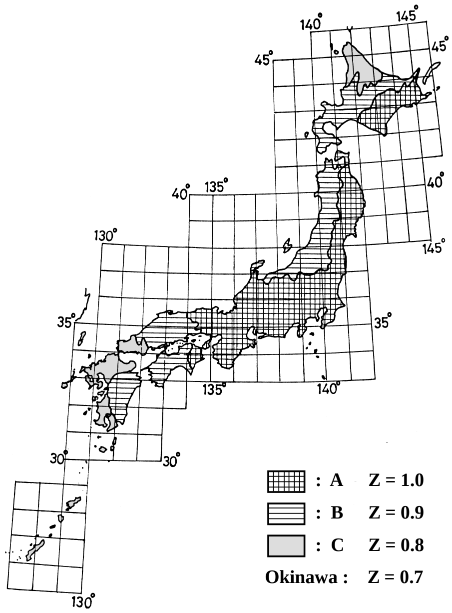


Figure 3: Seismic hazard zoning factor Z

Table 1: Design spectral factor* R_t

T	$T < T_c$	$T_c \leqq T < 2\,T_c$	$T \geqq 2\,T_c$
R_t	1	$1 - 0.2\left(\dfrac{T}{T_c} - 1\right)^2$	$\dfrac{1.6\,T_c}{T}$

* : R_t can also be calculated by other methods, but the calculated value shall not be less than 0.75 of the value given by this table.
T : Fundamental natural period (s) of the building (see Fig.(6)).
T_c : Critical period (s) of the soil (see Table 2).

The lateral shear distribution factor A_i is given as follows (see Fig.5 a).

$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i\right)\frac{2T}{1+3T} \tag{4}$$

where, α_i is the normalized weight of the i-th story, which is calculated as the weight above the i-th story divided by the weight above the ground level as follows.

$$\alpha_i = \frac{\sum_{j=i}^{n} w_j}{\sum_{j=1}^{n} w_j} = \frac{W_i}{W} \tag{5}$$

where w_j is the weignt of the j-th story.

Table 2: Classification of soil

Soil Profile	Ground Characteristics	T_c (s)
Type 1 (Hard soil)	Ground consisting of rock, hard sandy gravel, etc., classified as tertiary or older, or Ground whose period, estimated by calculation or by other investigation, is equivalent to that of the above.	0.4
Type 2 (Medium soil)	Other than Type 1 or 2.	0.6
Type 3 (Soft soil)	Alluvium consisiting of soft delta deposits, topsoil, mud, or the like (including fills, if any), whose depth is 30 m or more, land obtained by reclamation of marsh, muddy sea bottom, etc., where the depth of the reclaimed ground is 3 m or more and where 30 years have not yet elapsed since the time of reclamation, or Ground whose period, estimated by calculation or by other investigation, is equivalent to that of the above.	0.8

Critical period T_c is used in Table 1.

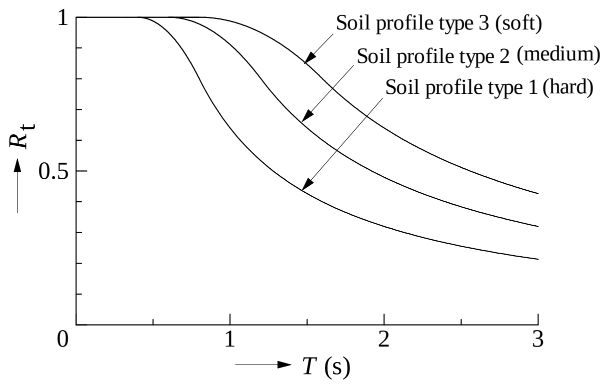


Figure 4: Design spectral factor R_t

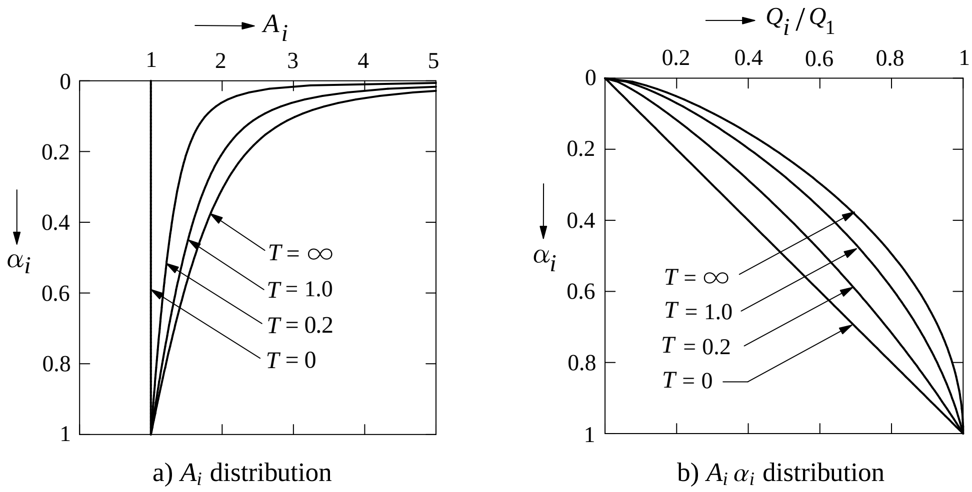


Figure 5: Lateral shear distribution factor A_i and its shear distribution $A_i\,\alpha_i$

The fundamental natural period of the building T (s) can be calculated as follows.

$$T = h\,(0.02 + 0.01\lambda) \tag{6}$$

where, h (m) is the height of the building, and λ is the ratio of the total height of stories of wooden or steel construction to the height of the building.

The period can also be calculated by eigenvalue analysis considering the mass and stiffness of the building. The value of R_t, however, can not be less than 0.75 of the value given by Table 1.

3.4 Story drift (Structural requirement (d) for Routes $\boxed{2}$, $\boxed{3}$ and $\boxed{4}$)

The drift of each story of the building (except the basement) caused by the lateral seismic shear for moderate earthquake motions or by the lateral seismic force at damage limit shall not exceed 1/200 of the story height. This value can be increased to 1/120, if the nonstructural elements shall have no severe damage at the increased story drift limitation.

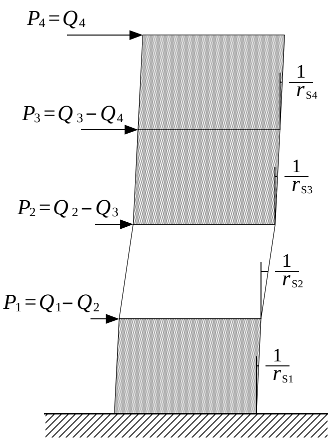
3.5 Lateral stiffness ratio (Structural requirement (e) for Route $\boxed{2}$)

The lateral stiffness ratio R_s of each story (except the basement) shall be equal to or greater than 0.6.

$$R_s = \frac{r_s}{\overline{r_s}} \geqq 0.6 \tag{7}$$

where, r_s is the lateral stiffness, which is defined as the story height divided by the story drift caused by the lateral seismic shear for moderate earthquake motions, and $\overline{r_s}$ is the mean lateral stiffness that is defined as the arithmetic mean of r_s's (see Fig.6).

In case the lateral stiffness ratio R_s becomes less than 0.6, the ultimate lateral capacity (Route $\boxed{3}$) shall be calculated and satisfy Eq.(9).



$$\overline{r_s} = \frac{\sum_{i=1}^{n} r_{si}}{n}$$

$$R_s = \frac{r_{si}}{\overline{r_s}} \geqq 0.6$$

Figure 6: Lateral stiffness ratio R_s of the i-th story

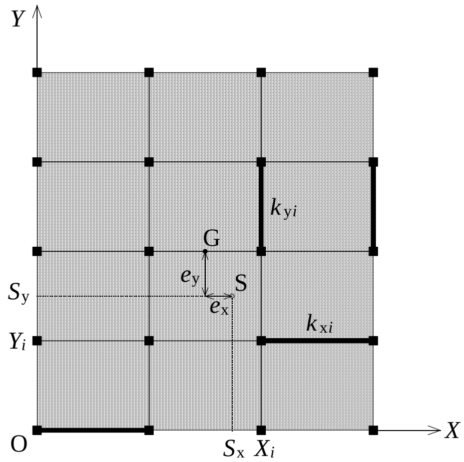
3.6 Stiffness eccentricity ratio (Structural requirement (e) for Route $\boxed{2}$)

The stiffness eccentricity ratio R_e of each story (except the basement) shall be equal to or less than 0.15.

$$R_e = \frac{e}{r_e} \leqq 0.15 \tag{8}$$

where, e is the eccentricity of the center of stiffness from the center of mass, and r_e is the elastic radius that is defined as the square root of the torsional stiffness divided by the lateral stiffness (see Fig.7).

In case the stiffness eccentricity ratio R_e becomes more than 0.15, the ultimate lateral capacity (Route $\boxed{3}$) shall be calculated and satisfy Eq.(9).



G: center of mass
S: center of stiffness
K_{xi}, K_{yi}: lateral stiffness of element i in X and Y directions

$$S_x = \frac{\sum (K_{yi} X_i)}{\sum K_{yi}} \qquad S_y = \frac{\sum (K_{xi} Y_i)}{\sum K_{xi}}$$

$$K_r = \sum \left\{ K_{xi} (Y_i - S_y)^2 \right\} + \sum \left\{ K_{yi} (X_i - S_x)^2 \right\}$$

$$r_{ex} = \sqrt{\frac{K_r}{\sum K_{xi}}} \qquad r_{ey} = \sqrt{\frac{K_r}{\sum K_{yi}}}$$

$$R_{ex} = \frac{e_y}{r_{ex}} \qquad R_{ey} = \frac{e_x}{r_{ey}}$$

Figure 7: Stiffness eccentricity ratio R_e in X and Y directions

3.7 Ultimate lateral capacity (Structural requirement (g) for Route $\boxed{3}$)

The ultimate lateral capacity (lateral load bearing capacity) of each story (except the basement) shall be equal to or greater than the specified lateral shear Q_{un} determined as follows.

$$Q_{un} = D_s F_{es} Q_{ud} \tag{9}$$

where, Q_{ud} is the lateral seismic shear for severe earthquake motions defined as Q_i in Eq.(1) with Eq.(2) where $C_0 \geqq 1.0$, D_s is the structural characteristic factor in Table 3, and F_{es} is the shape factor which shall be determined as follows.

$$F_{es} = F_e F_s \tag{10}$$

where, F_e and F_s are given in Table 4 and Table 5, respectively (see Fig.8).

The ultimate lateral capacity can be calculated as the shear of the story when the buiding come to yield subjected to horizontal forces, using structural mechanics, e.g. method of virtual work. Recently the lateral capacity Q is usually calculated using computer program of so-called “pushover analysis” (see Fig.9).

Table 3: Structural characteristic factor* D_s

Behavior of members		Type of frame		
		(1) Ductile moment frame	(2) Frame other than listed in (1) & (3)	(3) **
A	Members of excellent ductility	0.3	0.35	0.4
B	Members of good ductility	0.35	0.4	0.45
C	Members of fair ductility	0.4	0.45	0.5
D	Members of poor ductility	0.45	0.5	0.55

* The values can be decreased by 0.05 for steel frame reinforced concrete or steel buildings.
** Frames with shear walls or braces for reinforced concrete and steel frame reinforced concrete buildings, and frames with compressive braces for steel buildings.

3.8 Acceleration Response Spectra (Structural requirements (h), (i), (j) for Routes $\boxed{4}$, $\boxed{5}$)

The acceleration response specra are used for Route $\boxed{4}$ (Response and limit capacity method) and Route $\boxed{5}$ (Time history response analysis).

Table 4: Shape factor F_e and stiffness eccentricity ratio R_e

R_e of Eq.(8)	F_e
$R_e \leqq 0.15$	1.0
$0.15 < R_e < 0.3$	linear interpolation
$0.3 \leqq R_e$	1.5

Table 5: Shape factor F_s and lateral stiffness ratio R_s

R_s of Eq.(7)	F_s
$R_s \geqq 0.6$	1.0
$0.6 > R_s$	$2.0 - R_s/0.6$

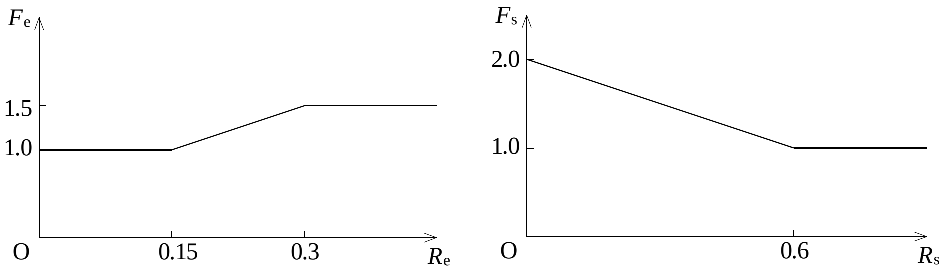
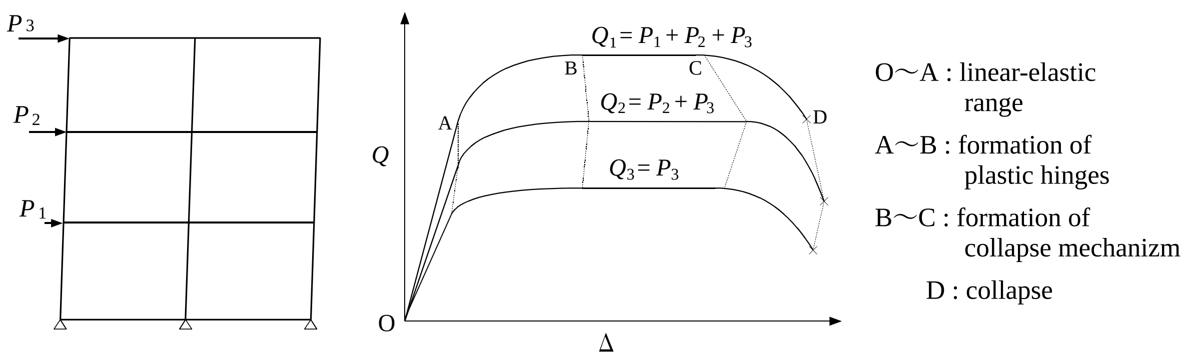


Figure 8: Shape factor $F_{es} = F_e F_s$



a) Pushover analysis b) Lateral capacity Q

Figure 9: Pushover analysis and lateral capacity Q

S_{Ad} (m/s^2) is the acceleration response at damage limit on the engineering bedrock that is given as follows (see Fig.10).

$$S_{Ad} = \begin{cases} (0.64 + 6\,T_d) & \text{for } T_d < 0.16 \\ 1.6 & \text{for } 0.16 \leqq T_d < 0.64 \\ 1.024/T_d & \text{for } 0.64 \leqq T_d \end{cases} \tag{11}$$

where, T_d (s) is the response period of the building at damage limit.

S_{As} (m/s^2) is the acceleration response at safety limit on the engineering bedrock that is given as follows (see Fig.10).

$$S_{As} = \begin{cases} (3.2 + 30\,T_s) & \text{for } T_s < 0.16 \\ 8.0 & \text{for } 0.16 \leqq T_s < 0.64 \\ 5.12/T_s & \text{for } 0.64 \leqq T_s \end{cases} \tag{12}$$

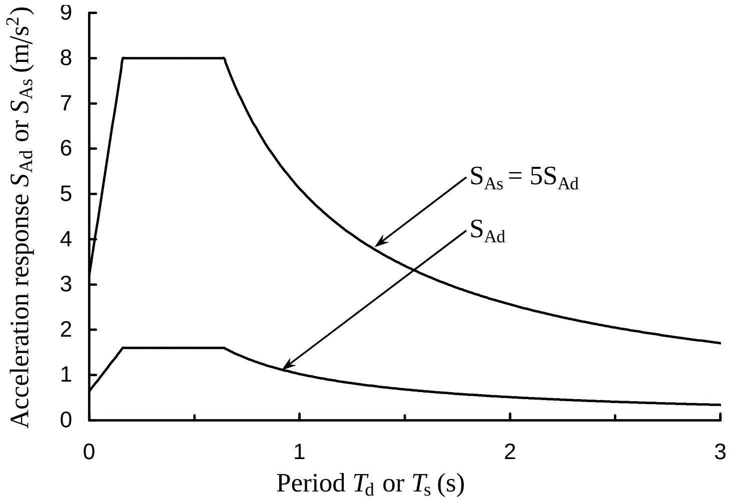


Figure 10: Acceleration response spectra at damage and safety limits

where, T_s (s) is the response period of the building at safety limit.

The earthquake motions at the ground level, for time history response analysis, shall conform to the designated response spectra, that shall be estimated, using the product of the acceleration response at engineering bedrock and the seismic hazard zoning factor Z, and considering the amplification of earthquake motions through surface soil above the engineering bedrock.

4 Conclusions

Major earthquakes, development of sesimic code and the current seismic code for buildings in Japan have been summarized. It can be seen that the earthquake damage forced to make modifications to seismic code. Recent major earthquakes have shown that it is important to improve seismic capacity not only for structural members but also for non-structural components as well. Furthermore, recent huge tsunami damage causes other serious problems to be solved.

Reference

Yuji Ishiyama: Introduction to Earthquake Engineering and Seismic Codes in the World, July 2012, http://iisee.kenken.go.jp/