

# Performance-Based Seismic Design Code for Buildings in Japan

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## ABSTRACT

The seismic design code of buildings in Japan was revised in June 2000 to implement a performance-based structural engineering framework. The code provides two performance objectives: life safety and damage limitation of a building at two corresponding levels of earthquake motions. The design earthquake motions are defined in terms of the acceleration response spectra specified at the engineering bedrock in order to take into consideration the soil conditions and soil-structure interaction effects as accurately as possible. The seismic performance shall be verified by comparing the predicted response values with the building's estimated limit values. The verification procedures of seismic performance in the new code are in essence a blend of the equivalent single-degree-of-freedom modeling of a building and the site-dependent response spectrum concepts, which make possible the prediction of the maximum structural response against earthquake motions without using time history analysis.

## INTRODUCTION

The 1995 Hyogoken-nanbu Earthquake caused much loss of human lives and severe damage or collapse of buildings [1]. Scientists and engineers learned many lessons regarding earthquake preparedness, disaster response, seismic design, upgrading of existing buildings and introduction of new technologies, which assure high safety levels of buildings during destructive earthquakes. As a result, the need for a new generation of seismic design was recognized and this recognition led to the development of performance-based engineering [2], whose framework explicitly addresses life-safety, reparability and functionality issues.

The seismic provisions of the building code of Japan were significantly revised in 2000 from an existing prescriptive format into a performance-based framework in order to expand design alternatives. In particular, these revisions allowed for the application of newly developed materials, structural elements, structural systems and construction. They are further expected to encourage structural engineers to develop and apply new construction technology. In the revised code, the precise definitions for structural performance requirements and verification procedures are specified on the basis of clear response and limit values. Assuming that material properties are well defined and the behavior of a building structure is properly predicted, the code should be applicable to any

kind of material and any type of building including seismic isolation systems.

This paper presents the concept and framework of the new verification procedures of seismic structural performance against major earthquake motions in the performance-based building code of Japan [3~6]. The verification procedure for developing the seismic design spectra includes (1) the basic design spectra defined at the engineering bedrock, and (2) the evaluation of site response from geotechnical data of surface soil layers. The verification procedures apply the equivalent linearization technique using an equivalent single-degree-of-freedom (ESDOF) system and the response spectrum analysis, while the previous procedures are based on the estimation of the ultimate capacity for lateral loads of a building.

### **SEISMIC PERFORMANCE REQUIREMENTS**

The new procedures deal with the evaluation and verification of structural performance at a set of limit states under dead and live loads, snow loads, and wind and earthquake forces. Two limit states should be considered for building structures to protect the life and property of the occupants against earthquake motions; life safety and damage limitation. To satisfy the life safety limit state, the engineer should design the building such that neither the entire building nor an individual story collapses. The damage limit state aims to prevent and control damage to the building, though some permanent deformation to energy dissipating devices is acceptable. Even if some damage occurs, the building must still satisfy the life-safety limit state during a subsequent earthquake.

Two sets of earthquake motions; maximum earthquake motions and once-in-a-lifetime earthquake motions are considered, each having different probability of occurrence. The effects of the design earthquake motions were maintained at the same levels as the design seismic forces in the previous code.

The level of maximum earthquake motions to be considered corresponds to the category of requirement for life safety and is assumed to produce the maximum possible effects on the structural safety of a building. The possible level of a maximum earthquake is determined on the basis of historical earthquake data, past recorded strong motions, seismic and geologic tectonics, active faults and other factors. This earthquake motion level corresponds approximately to that of the highest earthquake forces used in the conventional seismic design practice, representing the horizontal earthquake forces induced in buildings in case of major seismic events.

The level of once-in-a-lifetime events corresponds with the category of requirement for damage limitation of a building and is assumed to be experienced at least once during the lifetime of the building. A return period interval of 20 ~ 50 years is expected to cover these events. This earthquake motion level corresponds approximately to the middle level earthquake forces used in the conventional seismic design practice, representing the horizontal earthquake forces induced in buildings in case of moderate earthquakes.

### **DESIGN EARTHQUAKE MOTIONS**

The design seismic forces in the previous code were specified in terms of story shear forces as a function of building period and soil conditions without the apparent definition of earthquake ground motions. Therefore, the previous design seismic forces were easily applied to the seismic design. However, they become inconsistent. The estimated earthquake ground motions were not equal among the different soil conditions, since the previous design seismic forces were specified as the response values of representative buildings. It is also difficult to apply the design seismic forces to new structural systems and construction techniques, such as seismic isolation and structural-control buildings, and to take into account the seismic behavior of surface soil deposits. Considering this inconsistency, it was concluded that the seismic design should start with defining the input

earthquake ground motions. This methodology coincides with the performance-based structural engineering framework, which aspires toward flexible design. Consequently, new seismic design procedures including the design earthquake response spectrum [3~6] have been introduced to replace previous procedures.

**Design Response Spectrum at Engineering Bedrock**

The earthquake ground motion used for the seismic design at the life-safety limit state is the site-specific motion of an extremely rare earthquake, which is expected to occur once in approximately 500 years. The engineering bedrock is assumed to be a soil layer whose shear wave velocity is equal to or more than about 400m/s. The basic design earthquake acceleration response spectrum,  $S_0$ , of the seismic ground motion at the exposed (outcrop) engineering bedrock is shown in Fig. 1 and given in Eq. (1).

$$\begin{aligned}
 S_0(T) &= (3.2 + 30 T) && \text{for } T < 0.16 \\
 S_0(T) &= 8.0 && \text{for } 0.16 \leq T < 0.64 \\
 S_0(T) &= \frac{5.12}{T} && \text{for } 0.64 \leq T
 \end{aligned} \tag{1}$$

where,

- $S_0$ : basic design acceleration response spectrum at the exposed (outcrop) engineering bedrock ( $m/s^2$ ), and,
- $T$ : natural period (s).

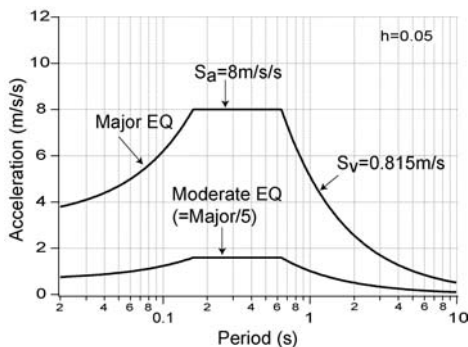


Fig. 1 Basic design earthquake acceleration response spectra at exposed engineering bedrock

The level of the earthquake ground motion used for the seismic design at the damage-limitation limit state should be reduced to a fifth of that for life safety. These response spectra at the engineering bedrock are applied in the design of all buildings, including conventionally designed buildings and seismically isolated buildings.

**Design Response Spectrum at Ground Surface**

Multiplying the response spectrum at the engineering bedrock by the surface soil layer amplification factor,  $G_s$ , as shown in Fig. 2, the design earthquake response spectrum at the ground surface,  $S_a$ , is obtained as shown in Fig. 3 and expressed by Eq. (2).

$$S_a(T) = G_s(T) Z S_0(T) \tag{2}$$

where,

- $S_a$ : design acceleration response spectrum at ground surface ( $m/s^2$ ),
- $G_s$ : surface soil layer amplification factor,
- $Z$ : seismic zone factor of 0.7 to 1.0, and,
- $T$ : natural period (s).

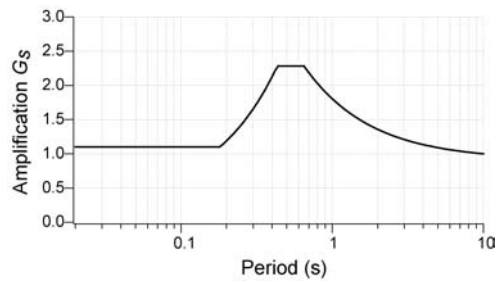


Fig. 2 Amplification factor of surface soil layers

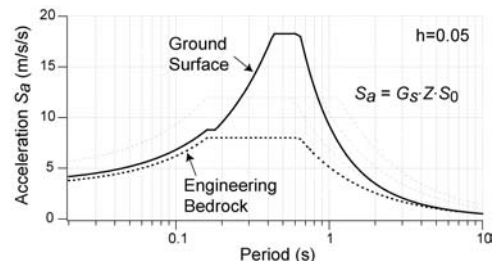


Fig. 3 Design earthquake acceleration response spectrum at ground surface

The calculation procedures of the amplification factor,  $G_s$ , are given by the accurate or simplified procedures [6,7].  $G_s$  to be determined here is the ratio of response spectra. Practically, the accurate procedures considering the strain-dependent properties of soils are available for most of soil conditions.  $G_s$  is calculated based on the strain-dependent shear stiffness and damping ratio of soil [8~10].  $G_s$  is given by Eq. (3):

$$G_s = G_{s2} \frac{T}{0.8 T_2} \quad \text{for } T \leq 0.8 T_2$$

$$G_s = G_{s2} + \frac{G_{s1} - G_{s2}}{0.8(T_1 - T_2)}(T - 0.8 T_2) \quad \text{for } 0.8 T_2 < T \leq 1.2 T_1$$

$$G_s = G_{s1} \quad \text{for } 0.8 T_1 < T \leq 1.2 T_1$$

$$G_s = G_{s1} + \frac{G_{s1} - 1.0}{\frac{1}{1.2 T_1} - 0.1} \left( \frac{1}{T} - \frac{1}{1.2 T_1} \right) \quad \text{for } 1.2 T_1 < T \quad (3)$$

where,

$G_s$  : surface-soil-layer amplification factor,

$G_{s1}$  :  $G_s$  value at the period of  $T_1$ ,

$G_{s2}$  :  $G_s$  value at the period of  $T_2$ ,

$T$  : natural period (s),

$T_1$  : predominant period of surface soil layers for the first mode (s), and,

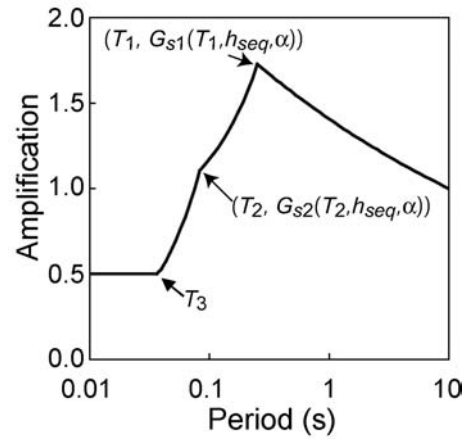
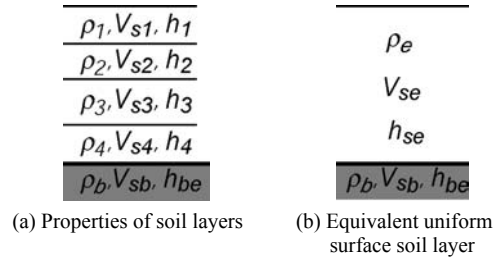
$T_2$  : predominant period of surface soil layers for the second mode (s).

Minimum value of  $G_s$ : 1.5 for  $T \leq 1.2 T_1$  and 1.35 for  $1.2 T_1 < T$  at the damage-limitation limit state, and 1.2 for  $T \leq 1.2 T_1$  and 1.0 for  $1.2 T_1 < T$  at the life-safety limit state.

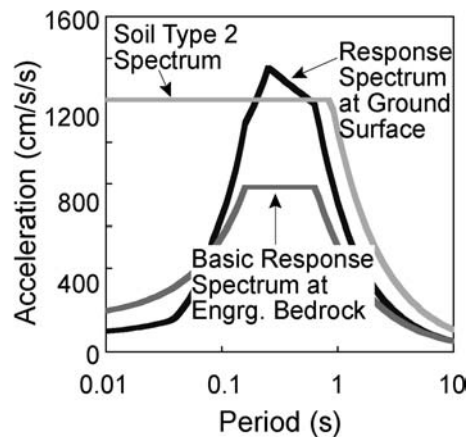
The factors of 0.8 and 1.2 in the period classification such as  $0.8 T_1$ ,  $0.8 T_2$  and  $1.2 T_1$  in Eq. (3) are introduced to consider the uncertainties included in the soil properties and the simplified calculation.

### Amplification Factor for Surface Soil Layers

The calculation procedures of the surface soil layer amplification factor,  $G_s$ , in surface soil layers according to the provision [7] are illustrated in Fig. 4. The iteration is required in the calculation procedures because of soil nonlinearity.



(c) Amplification factor of equivalent surface soil layer



(d) Design acceleration response spectrum at ground surface

Fig. 4 Amplification factor of surface soil layers

The amplification of ground motion by surface soil layers is estimated using the geotechnical data at the site, the equivalent single soil layer modeled from surface soil layers, and the equivalent linearization technique. The nonlinear amplification of ground motion by a uniform soil layer above the engineering bedrock is evaluated by applying the one-dimensional wave propagation theory.

The surface soil layers are reduced to an equivalent single soil layer. Consequently, the soil layers including the engineering bedrock are reduced to the equivalent two-soil-layer model. The characteristic values of the equivalent surface soil layer are expressed by Eqs. (4) to (7):

$$V_{se} = \frac{\sum V_{si} d_i}{H} \quad (4)$$

$$\rho_e = \frac{\sum \rho_i d_i}{H} \quad (5)$$

$$h_{se} = \frac{\sum h_i W_{si}}{\sum W_{si}} \quad (6)$$

$$H = \sum d_i \quad (7)$$

where,

$V_{se}$  : equivalent shear wave velocity of surface soil layers (m/s),

$\rho_e$  : equivalent mass density of surface soil layers ( $t/m^3$ ),

$h_{se}$  : equivalent damping ratio of surface soil layers,

$H$  : total thickness of surface soil layers (m),

$V_{si}$  : shear wave velocity of soil layer  $i$  (m/s),

$d_i$  : thickness of soil layer  $i$  (m),

$\rho_i$  : mass density of soil layer  $i$  ( $t/m^3$ ),

$d_i$  : thickness of soil layer  $i$  (m),

$h_i$  : viscous damping ratio of soil layer  $i$ , and,

$W_{si}$  : potential energy of soil layer  $i$ .

Equation (6) represents the averaged value of the equivalent viscous damping ratio of the equivalent surface soil layer. The value of  $h_i$  in Eq. (6) is estimated from geotechnical data at the

site or the relationships of viscous damping ratio and shear strain of soils given in the provision [7]. Finally, the viscous damping ratio,  $h_{seq}$ , of the equivalent surface soil layer is estimated by Eq. (8) at the final step of iteration in the calculation, considering the scattering of geotechnical data for estimating damping ratios.

$$h_{seq} = 0.8 \frac{\sum h_i W_{si}}{\sum W_{si}} \quad (8)$$

The first and second predominant periods,  $T_1$  and  $T_2$ , and amplification factors,  $G_{s1}$  and  $G_{s2}$ , of the equivalent surface soil layer are obtained by Eqs. (9) to (12):

$$T_1 = \frac{4H}{V_{se}}, \quad T_2 = \frac{T_1}{3} \quad (9)$$

$$G_{s1} = \frac{1}{1.57 h_{seq} + \alpha} \quad (10)$$

$$G_{s2} = \frac{1}{4.71 h_{seq} + \alpha} \quad (11)$$

$$\alpha = \frac{\rho_e V_{se}}{\rho_b V_{sb}} \quad (12)$$

where,

$\alpha$  : wave impedance ratio,

$\rho_b$  : mass density of engineering bedrock ( $t/m^3$ ),  
and,

$V_{sb}$  : shear velocity of engineering bedrock (m/s).

Minimum value of  $G_{s1}$ : 1.5 at the damage-limitation limit state and 1.2 at the life-safety limit state.

Equations (10) and (11) are obtained from previous studies [11,12].

## VERIFICATION OF SEISMIC PERFORMANCE

### Verification Procedures for Major Earthquake Motions

The new verification procedures involve the application of the equivalent linearization

technique using an equivalent single-degree-of-freedom (ESDOF) system and the response spectrum analysis, while the previous procedures were based on the estimation of the ultimate capacity for lateral loads of a building. A variety of linearization techniques have already been studied [e.g., 13]. Several applications of linearization techniques have also been published [14~17].

Various response and limit values are considered for use in the performance verification procedures in accordance with each of the requirements prescribed for building structures. The principle of the verification procedures is that the predicted response values caused by earthquake motions should not exceed the estimated limit values. In the case of a major earthquake, the maximum strength and displacement response values should be smaller than the ultimate capacity for strength and displacement.

The focus is hereafter placed on the verification procedures for major earthquakes. The analytical method used for predicting the structural response applies the equivalent linearization technique using an ESDOF system and the response spectrum analysis. A flow of the procedures is illustrated in Fig. 5.

According to the verification procedures, the steps to be followed are:

- (1) Confirm the scope of application of the procedures and the mechanical properties of materials and/or members to be used in a building.
- (2) Determine the design response spectra used in the procedures.
  - (a) For a given basic design spectrum at the engineering bedrock, draw up the acceleration,  $S_a$ , and displacement response spectra,  $S_d$ , at ground surface for the different damping levels.
  - (b) In the estimation of the free-field site-dependent acceleration and displacement response, consider the strain-dependent soil deposit characteristics.
  - (c) If needed, present the relation of  $S_a$ - $S_d$  for

the different damping levels (see Fig. 5(c)).

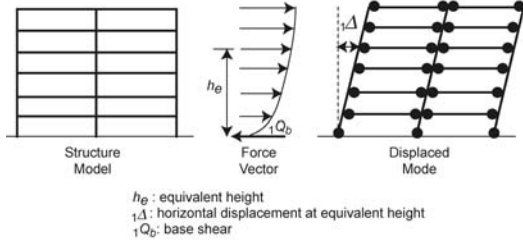
- (3) Determine the hysteretic characteristics, equivalent stiffness and equivalent damping ratio of the building.
  - (a) Model the building as an ESDOF system and establish its force-displacement relationship (see Fig. 5(a)).
  - (b) Determine the design limit strength and displacement of the building corresponding to the ESDOF system.
  - (c) The soil-structure interaction effects should be considered if necessary.
  - (d) If needed, determine the equivalent stiffness in accordance with the limit values.
  - (e) Determine the equivalent damping ratio on the basis of the viscous damping ratio, hysteretic dissipation energy and elastic strain energy of the building (see Fig. 5(b)).
  - (f) If the torsional vibration effects are predominant in the building, these effects should be considered when establishing the force-displacement relationship of the ESDOF system.
- (4) Examine the safety of the building. In this final step, it is verified whether the response values predicted on the basis of the response spectra determined according to step 2 satisfy the condition of being smaller than the limit values estimated on the basis of step 3 (see Fig. 5(c)).

In order to determine the design limit strength and displacement of the building, it is necessary to assume a specific displaced mode for its inelastic response (see Figure 5(a)). Basically, any predominant or potential displaced mode should be considered.

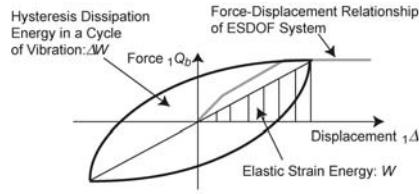
#### **Estimation of Ultimate Deformation of a Member**

The seismic performance of a building is evaluated at the two limit states under the two levels of design earthquake motions.

The limit state of damage limitation is attained when the working stress increases to the allowable stress of materials in any member or when the

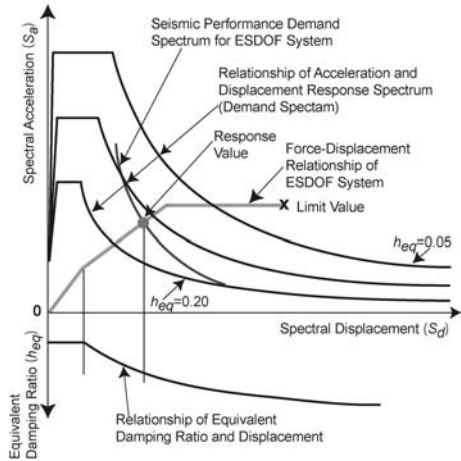


(a) Reduction of building to ESDOF system by pushover analysis



Energy for Equivalent Damping Ratio

(b) Equivalent damping ratio using hysteretic energy dissipation



(c) Performance criteria using demand spectra and force-displacement relationship of ESDOF system in  $S_a$ - $S_d$  relations

Fig. 5 Verification procedures for major earthquake motions

story drift reaches 1/200 of the story height at any story. The limit state of life safety is reached when the building cannot sustain the gravity loads at any story under additional lateral drift, that is, a structural member has reached its ultimate deformation capacity. The ultimate deformation of a member should be estimated by Eq. (13).

$$R_u = R_b + R_s + R_x \tag{13}$$

where,

$R_u$ : ultimate deformation of a member,  
 $R_b$ : flexural deformation of a member,  
 $R_s$ : shear deformation of a member, and,  
 $R_x$ : deformation resulting from the deformation in the connection to adjacent members and others.

The ultimate flexural deformation,  $R_b$ , should be calculated as follows:

$$R_b = \frac{\phi_y a}{3} + (\phi_u - \phi_y) l_p \left(1 - \frac{l_p}{2a}\right) \tag{14}$$

where,

$\phi_y$  : curvature of a member when the allowable stress is first reached in the member,  
 $\phi_u$  : curvature of a member at the maximum resistance,  
 $l_p$  : length of plastic region, and,  
 $a$  : shear span or a half of clear length of a member.

### Modeling of Multi-Degree-of-Freedom System into ESDOF System

In estimating the seismic response of a multi-story building structure, the building is modeled as an ESDOF system as shown in Fig. 5. This modeling is based on the result of the nonlinear pushover analysis under the horizontal forces at each floor level, of which the distribution along the height should be proportional to the first mode shape of vibration or the  $A_i$  distribution prescribed in the provision [8]. The modeling is discussed in detail elsewhere [18].

The deflected shape resulting from the pushover analysis is assumed to represent the first mode shape of vibration. As the deflected shape does not change very much with the distribution of horizontal forces along the height, the fixed force distribution is used during the pushover analysis.

The modal analysis is applied to relate the seismic response of the multi-degree-of-freedom and ESDOF systems. For spectral response acceleration,  ${}_1S_a$ , and displacement,  ${}_1S_d$ , at the

first-mode period and damping, the first-mode inertia force vector,  ${}_1\{f\}$ , and displacement vector,  ${}_1\{\delta\}$ , are expressed in the following:

$${}_1\{f\} = [m] {}_1\beta {}_1\{u\} {}_1S_a \quad (15)$$

$${}_1\{\delta\} = {}_1\beta {}_1\{u\} {}_1S_d \quad (16)$$

where,

${}_1S_a$  : spectral response acceleration for the first mode,

${}_1S_d$  : spectral response displacement for the first mode,

${}_1\{f\}$  : inertia force vector for the first mode,

${}_1\{\delta\}$  : displacement vector for the first mode,

${}_1\beta$  : modal participation factor for the first mode,

${}_1\{u\}$  : mode shape vector for the first mode (normalized to the roof displacement,  ${}_1\beta {}_1S_d$ ), and,

$[m]$  : lumped floor mass matrix.

The modal participation factor is expressed as follows:

$${}_1\beta = \frac{{}_1\{u\}^T [m] \{1\}}{{}_1\{u\}^T [m] {}_1\{u\}} \quad (17)$$

where,

$\{1\}$  : unit vector.

The force-displacement relationship of the ESDOF system is given by Eqs. (18) and (19), when the force corresponds to the base shear,  ${}_1Q_b$ , and its displacement,  ${}_1\Delta$ , corresponds to the displacement at the equivalent height,  $h_e$ , where the modal participation function,  ${}_1\beta {}_1\{u\}$ , is equal to unity.

$${}_1Q_b = \{1\}^T {}_1\{f\} = \{1\}^T [m] {}_1\beta {}_1\{u\} {}_1S_a = {}_1M_e {}_1S_a \quad (18)$$

$${}_1\Delta = {}_1S_d \quad (19)$$

where,

${}_1Q_b$  : base shear corresponding to the first mode,

${}_1\Delta$  : displacement at the equivalent height corresponding to the first mode, and,

${}_1M_e$  : effective modal mass corresponding to the first mode given as follows:

$${}_1M_e = \{1\}^T [m] {}_1\beta {}_1\{u\} \quad (20)$$

According to the provision [7], the effective mass should not be less than 0.75 times the total mass of the building.

### Force-Displacement Curve in $S_a$ - $S_d$ Relations

Assuming that the first-mode displacement and inertia force vectors are equal to the floor displacement and external force distributions, respectively obtained from the pushover analysis, the force-displacement relationship of an ESDOF system is expressed in spectral acceleration and displacement ( $S_a$ - $S_d$ ) relations as follows:

$${}_1S_a = \frac{{}_1Q_b}{{}_1M_e} = \frac{{}_1\{\delta\}^T [m] {}_1\{\delta\}}{({}_1\{\delta\}^T [m] \{1\})^2} {}_1Q_b \quad (21)$$

$${}_1S_d = {}_1\Delta = \frac{{}_1S_a}{{}_1\omega_e^2} = \frac{{}_1\{\delta\}^T [m] {}_1\{\delta\}}{{}_1\{\delta\}^T [m] \{1\}} {}_1S_a \quad (22)$$

where,  ${}_1\omega_e$ : effective circular frequency for the first mode.

The effective first-mode circular frequency of the building at each loading step is approximately estimated by Eq. (23).

$${}_1\omega_e = \sqrt{\frac{{}_1K_e}{{}_1M_e}} = \sqrt{\frac{{}_1\beta {}_1\{u\}^T [k] {}_1\beta {}_1\{u\}}{\{1\}^T [m] {}_1\beta {}_1\{u\}}} = \sqrt{\frac{{}_1\{\delta\}^T {}_1\{f\}}{({}_1\{\delta\}^T [m] {}_1\{\delta\})}} \quad (23)$$

where,

${}_1K_e$ : effective modal stiffness corresponding to the first mode, and,

$[k]$ : stiffness matrix of the building.

Consequently, using Eqs. (21) and (22), the external forces and displacements at each floor level, and the base shear at each loading step obtained from the nonlinear pushover analysis, the force-displacement relationship of the ESDOF system in  $S_a$ - $S_d$  relations may be plotted as illustrated in Fig. 5(c). This relation is



sometimes called the capacity curve of the building.

### Estimation of Equivalent Damping Ratio

The equivalent damping ratio is defined by the viscous damping, hysteretic dissipation energy, elastic strain energy of a building and the radiation damping effects of the ground.

The equivalent damping ratio for the first mode is prescribed to be 0.05 at the damage-limitation limit state because the behavior of a building is basically elastic.

The equivalent viscous damping ratio at the life-safety limit state is defined by equating the energy dissipated by hysteretic behavior of a nonlinear system and the energy dissipated by viscous damping under stationary vibration in resonance. The equivalent damping ratio of an ESDOF system,  ${}_{st}h_{eq}$ , is defined as follows (see Fig. 5(b)).

$${}_{st}h_{eq} = \frac{1}{4\pi} \frac{\Delta W}{W} \quad (24)$$

where,

- ${}_{st}h_{eq}$  : equivalent damping ratio of an ESDOF system under resonant stationary vibration,
- $\Delta W$  : dissipation energy of an ESDOF system, and,
- $W$  : potential energy of an ESDOF system ( ${}_1Q_b \cdot \Delta/2$ ).

The dissipation energy of a stationary hysteretic loop at the assumed maximum response of a building is either estimated by calculating the area of the supposed cyclic loop of the building in the nonlinear pushover analysis, or determined based on the equivalent damping ratio of each structural element considered.

Equation (24) does not hold in the response under nonstationary excitations such as earthquake motions. The equivalent damping ratio under stationary vibration must be reduced to correlate the maximum response of an equivalent linear system and a nonlinear system under earthquake motions. According to the analytical results [5], the equivalent damping ratio is

reduced to approximately 80 percent of that calculated by Eq. (24).

The equivalent damping ratio,  $h_{eq}$ , of an ESDOF system should be in principle estimated as the weighted average with respect to strain energy of each member according to the provision [7]:

$$h_{eq} = \frac{\sum_m h_{eqi} {}_mW_i}{\sum_m {}_mW_i} + 0.05 \quad (25)$$

where,

- $h_{eq}$  : equivalent damping ratio of an ESDOF system,
- ${}_{m}h_{eqi}$  : equivalent damping ratio of member  $i$ , and,
- ${}_mW_i$  : strain energy stored in member  $i$  at ultimate deformation.

The equivalent damping ratio,  ${}_{m}h_{eqi}$ , of member  $i$  is estimated as follows:

$${}_{m}h_{eqi} = \gamma \left( 1 - \frac{1}{\sqrt{\mu}} \right) \quad (26)$$

where,

- $\mu$ : ductility factor of a member reached at the ultimate state of a building.

The factor of  $\gamma$  is the reduction factor considering the damping effect for the transitional seismic response of the building [13]. It takes the values of 0.25 for ductile members and 0.2 for non-ductile ones. When all structural members of a building structure have the same hysteretic characteristics, the equivalent damping ratio of a whole building can be estimated by Eq. (26).

### Soil-Structure Interaction Effects

The effective period and equivalent damping ratio should be modified by the following equations taking into consideration the effects of soil-structure interaction if necessary in case of major earthquake motions. A sway-rocking analytical model is assumed in the modeling of soil-structure system.

$$r = \sqrt{1 + \left(\frac{T_{sw}}{T_e}\right)^2 + \left(\frac{T_{ro}}{T_e}\right)^2} \quad (27)$$

$$h_{eq} = \frac{1}{r^3} \left\{ h_{sw} \left(\frac{T_{sw}}{T_e}\right)^3 + h_{ro} \left(\frac{T_{ro}}{T_e}\right)^3 + h_b \right\} \quad (28)$$

where,

- $r$  : period modification factor,
- $T_e$  : effective period of a fixed-base super-structure at ultimate state,
- $T_{sw}$  : period of sway vibration at ultimate state,
- $T_{ro}$  : period of rocking vibration at ultimate state,
- $h_{sw}$  : damping ratio of sway vibration of surface soil layers corresponding to shear strain level considered, but the value is limited to 0.3,
- $h_{ro}$  : damping ratio of rocking vibration or surface soil layers corresponding to shear strain level considered, but the value is limited to 0.15, and,
- $h_b$  : equivalent damping ratio of a super-structure at ultimate state.

#### Demand $S_a$ - $S_d$ Spectrum and Response Spectrum Reduction Factor

Response spectral displacement,  $S_d(T)$ , is estimated from the linearly elastic design acceleration response spectrum,  $S_a(T)$ , at the free surface by Eq. (29). The demand  $S_a$ - $S_d$  spectra for different damping ratios are constructed using Eq. (29) as illustrated in Fig. 5(c).

$$S_d(T) = \left(\frac{T}{2\pi}\right)^2 S_a(T) \quad (29)$$

The demand  $S_a$ - $S_d$  spectra are prepared for the damping ratio of 0.05 up to the yield displacement, and for the estimated equivalent damping ratio up to the ultimate displacement. Beyond the yield displacement, the response spectral acceleration and displacement are reduced by the following factor:

$$F_h = \frac{1.5}{1 + 10 h_{eq}} \quad (30)$$

where,  $F_h$ : response spectrum reduction factor.

#### Seismic Performance Criteria

The seismic performance of a building under the design earthquake motion is examined by

comparing the force-displacement relationship of the building and the demand spectrum of the design earthquake motion in  $S_a$ - $S_d$  relations. The intersection of the force-displacement relationship and the demand spectrum for the appropriate equivalent damping ratio represents the maximum response under the design earthquake motion as shown in Fig. 5(c).

In the provision [7], spectral acceleration of a building, defined by Eq. (21), at a limit state should be equal to or higher than the corresponding acceleration of the demand spectrum using the effective period, corresponding to Eq. (23), and equivalent damping ratio, expressed by Eqs. (25) or (26), at the limit state.

## CONCLUSIONS

This paper presents the seismic design code of buildings in Japan revised in June 2000 toward a performance-based structural engineering framework. The code provides two performance objectives: life safety and damage limitation of a building at corresponding levels of earthquake motions. The design earthquake motions are defined as the acceleration response spectra specified at the engineering bedrock in order to take into consideration the soil conditions and soil-structure interaction effects as accurately as possible. Design earthquakes with return periods of approximately 500 years and 50 years are used to evaluate the seismic performance at the life-safety and damage-limitation levels, respectively. The seismic performance shall be verified by comparing the predicted response values with the estimated limit values of both the overall building and structural components.

The verification procedures for seismic performance against the design earthquake motions in the new code are in essence a blend of the ESDOF modeling of a building and the site-dependent response spectrum concepts, and the application of a nonlinear pushover analysis and the modal analysis. The new procedures make possible the prediction of the maximum structural response against earthquake motions without using time history analysis.

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