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PERFORMANCE-BASED SEISMIC DESIGN PROVISIONS NEWLY INTRODUCED TO THE BUILDING STANDARD LAW OF JAPAN

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ABSTRACT: The seismic design provisions of buildings in Japan was revised toward a performance-based structural engineering framework in 2000. The provisions provide 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 to take account of the soil conditions and soil-structure interaction effects as properly as possible. The seismic performance shall be verified by comparing the predicted response values with the estimated limit values of a building.

Key Words: performance-based provision, seismic design, building, site-dependant response spectrum, amplification by surface soil layers, response spectrum analysis, equivalent single-degree-of-freedom modeling, equivalent linearization

INTRODUCTION

The 1995 Hyogoken-nanbu Earthquake caused a lot of loss of human lives and extensive damage or collapse of buildings (BRI 1996). Many lessons among practitioners and scientists were learnt about earthquake preparedness, disaster response, seismic design, upgrading of existing buildings and introduction of new technologies. As a result, the need for a new generation of seismic design was recognized and led to the development of the performance-based engineering (Yamanouchi 2000), whose framework explicitly address life-safety, reparability and functionality issues.

The seismic provisions of the Building Standard Law of Japan (BSLJ) were significantly revised in 2000 from existing prescriptive into performance-based type to enlarge the alternatives of structural design, particularly for the application of newly developed materials, structural elements, structural systems and construction. It is further expected to encourage structural engineers to develop and apply new construction technology. In the revised provisions, the precise definitions for structural performance requirements and verification procedures are specified on the basis of definite response and limit values. The provisions should be applicable to any kind of materials and any type of buildings on condition that the material properties are clearly defined and the behavior of a building structure is properly predicted.

This paper presents the concept and framework of new verification procedures of seismic structural performance against major earthquake motions in the performance-based building provisions of BSLJ (Hiraishi 1999, 2000; Midorikawa 2000; Miura 2000). The basic concept for seismic design spectra of earthquake motions in the verification procedures is 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.

REQUIREMENTS FOR SEISMIC PERFORMANCE

The new procedures deal with the evaluation and verification of structural performance at a set of limit states under dead, live and snow loads, and wind and earthquake forces. Two limit states need to be considered for building structures to protect the life and property of the occupants against earthquake motions; life safety and damage limitation.

Life safety: the essential purpose of this requirement is the safety of human life. It should be expected that not only the entire building but also no story of the building should collapse under the action of earthquake motions considered.

Damage limitation: the purpose of this requirement is to prevent and limit the damage of a building. It is required that the structural damage which causes the conditions of the building not satisfying the required structural performance for life safety should not take place after the action of earthquake motions considered.

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

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 to be constructed at a site. 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 to 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. 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 provisions were specified in terms of story shear forces as a function of building's period and soil conditions without the apparent definition of earthquake ground motions. Consequently, the previous design seismic forces were easily applied to the seismic design. However, they become inconsistent. The estimated earthquake ground motions without the effects of surface soil layers were not equal among the different soil conditions, since the previous design seismic forces were specified as the response values of a building and the response amplification by a specific building. It is further difficult to apply the design seismic forces to new structural systems and construction 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 coincides with the framework of the performance-based structural engineering aiming at the flexible design. Therefore, the new seismic design procedures including the design earthquake response spectrum (Hiraishi 1999, 2000; Midorikawa 2000; Miura 2000) have been introduced to BSLJ.

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 that is expected to occur once in approximately 500 years. A soil layer whose shear wave velocity is not less than about 400 m/s is assumed to be the engineering bedrock. 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).

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

where, S_0: basic design acceleration response spectrum at the exposed (outcrop) engineering bedrock (m/s^2), and, T: natural period (s). ‘Major EQ’ and ‘Moderate EQ’ in Fig. 1 correspond to the maximum earthquake motions to be considered and the once-in-a-lifetime earthquake motions, respectively.

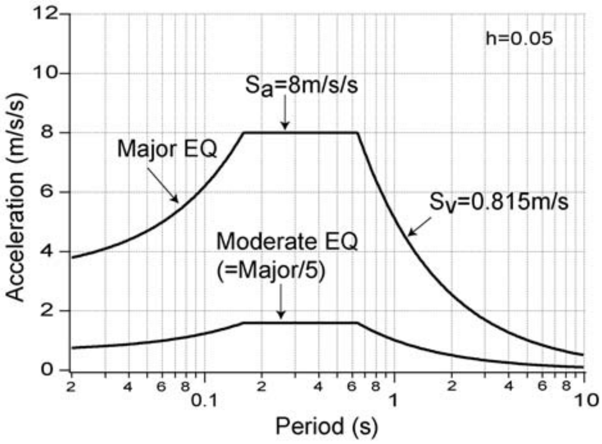


Fig. 1 Basic design earthquake acceleration response spectra at exposed engineering bedrock

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 such as 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).

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

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

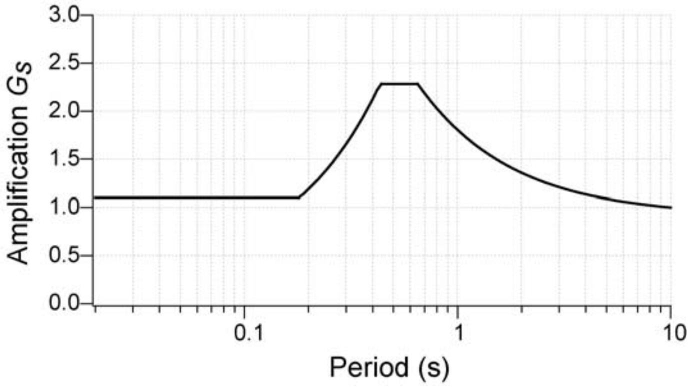


Fig. 2 Amplification factor of surface soil layers

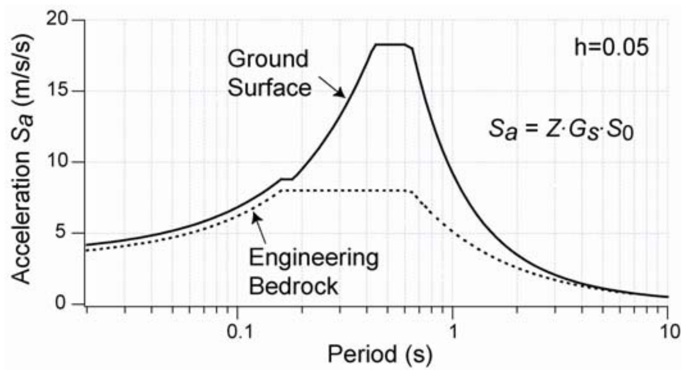


Fig. 3 Design earthquake acceleration response spectrum at ground surface

The calculation procedures of the amplification factor, G_s, are given by the accurate or simplified procedures (Miura 2000; MLIT 2000). 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 (Schnabel 1972; Ohsaki 1978; MLIT 1980). G_s is given by Eq. (3):

$$\begin{aligned}
G_S &= G_{S2}\frac{T}{0.8T_2} && \text{for} \quad T \le 0.8T_2 \\
G_S &= G_{S2} + \frac{G_{S1} - G_{S2}}{0.8(T_1 - T_2)}(T - 0.8T_2) && \text{for} \quad 0.8T_2 < T \le 1.2T_1 \\
G_S &= G_{S1} && \text{for} \quad 0.8T_1 < T \le 1.2T_1 \\
G_S &= G_{S1} + \frac{G_{S1} - 1.0}{\frac{1}{1.2T_1} - 0.1}\left(\frac{1}{T} - \frac{1}{1.2T_1}\right) && \text{for} \quad 1.2T_1 < T
\end{aligned} \tag{3}$$

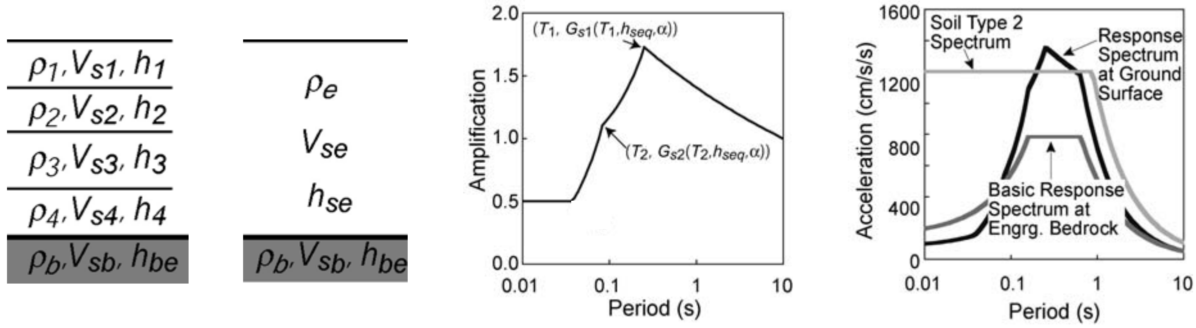
where, G_s: surface-soil-layer amplification factor, G_{s1}: G_s value at the period of T_1, G_{s2}: G_s value at the

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 $T \leq 1.2T_1$ and 1.35 for $1.2T_1 < T$ at the damage-limitation limit state, and 1.2 for $T \leq 1.2T_1$ and 1.0 for $1.2T_1 < T$ at the life-safety limit state.

The factors of 0.8 and 1.2 in the period classification such as $0.8T_1$, $0.8T_2$ and $1.2T_1$ 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 (MLIT 2000) are shown in Fig. 4. The iteration is needed in the calculation procedures because of nonlinear behavior of soils.



(a) (b) (c) (d)

Fig. 4 Amplification factor of surface soil layers; (a) Properties of soil layers, (b) Equivalent uniform surface soil layer, (c) Amplification factor of equivalent surface soil layer, (d) Design acceleration response spectrum at ground surface

The amplification of ground motion by surface soil layers is evaluated 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 estimated by applying the theory of one-dimensional wave propagation.

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 given by Eqs. (4) to (7):

$$V_{se} = \frac{\sum V_{si} d_i}{H} \tag{4}$$

$$\rho_e = \frac{\sum \rho_i d_i}{H} \tag{5}$$

$$h_{se} = \frac{\sum h_i W_{si}}{\sum W_{si}} \tag{6}$$

$$H = \sum d_i \tag{7}$$

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

Eq. (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 (MLIT 2000). Finally, the viscous damping ratio, h_{seq}, of the equivalent surface soil layer is evaluated by Eq. (8) at the final step of iteration in the calculation, considering the scatter of geotechnical data for estimating damping ratios.

$$h_{seq} = 0.8 \frac{\sum h_i W_{si}}{\sum W_{si}} \tag{8}$$

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):

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

$$G_{s1} = \frac{1}{1.57 h_{seq} + \alpha} \tag{10}$$

$$G_{s2} = \frac{1}{4.71 h_{seq} + \alpha} \tag{11}$$

$$\alpha = \frac{\rho_e V_{se}}{\rho_b V_{sb}} \tag{12}$$

where, α: wave impedance ratio, ρ_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.

Eqs. (10) and (11) are obtained from the previous studies (Roesset 1969; Ohsaki 1982).

VERIFICATION OF SEISMIC PERFORMANCE

Verification Procedures for Major Earthquake Motions

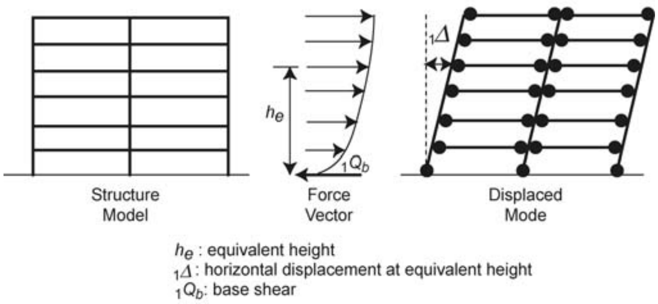
The new verification procedures involve the application of the equivalent linearization technique using an ESDOF system and the response spectrum analysis. A variety of linearization techniques has already been studied (e.g. Shibata 1976). Several applications of linearization techniques have also been published (Freeman 1978; AIJ 1989, 1992;ATC-40 1996).

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 due to the effects of earthquake motions on building structures should not exceed the estimated limit values. In the case of a major earthquake, the maximum response values of force and displacement of a building should be smaller than the ultimate capacity for strength and displacement.

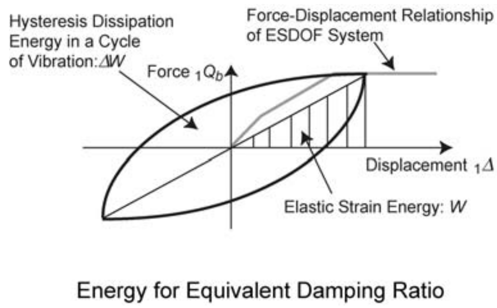
The focus is hereafter put on the verification procedures for major earthquake motions. The analytical method used for predicting the structural response applies the equivalent linearization technique using an ESDOF system and the response spectrum analysis, as illustrated in Fig. 5.

According to the verification procedures, the steps are as follows:
(1) Confirm the scope of application of the procedures and the mechanical properties of materials and/or members 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.

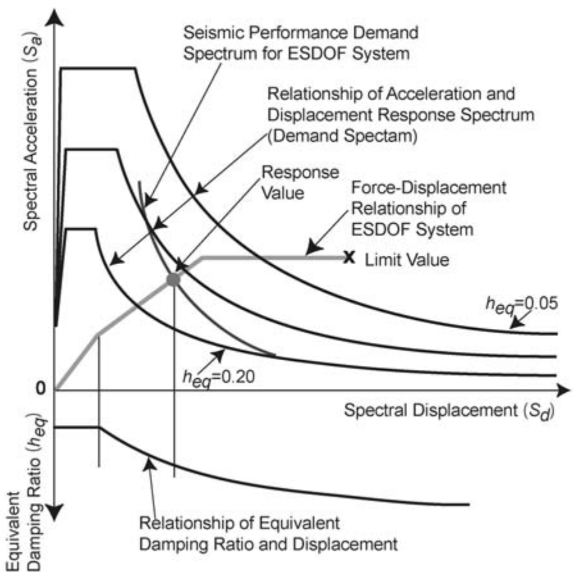
b) In the estimation of the free-field site-dependent acceleration and displacement response, consider the strain-dependent soil deposit characteristics.
c) If needed, construct the relation of S_a-S_d for the different damping levels (see Fig. 5(c)).
(3) Determine the hysteretic characteristics, equivalent 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 effects of soil-structure interaction 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 effects of torsional vibration 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)).



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



(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

It is necessary to assume a specific displaced mode for its inelastic response in order to determine the design limit strength and displacement of the building (see Fig. 5(a)). Any predominant or potential displaced mode should basically be considered.

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 force 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 (MLIT 1980).

The deflected shape resulted 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 multi-degree-of-freedom and ESDOF systems.

The modeling is discussed in detail elsewhere (Kuramoto 2000).

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:

$$ {}_1S_a = {}_1Q_b / {}_1M_e = \frac{{}_1\{\delta\}^T [m]_1\{\delta\}}{\left({}_1\{\delta\}^T [m]\{1\}\right)^2} {}_1Q_b \tag{13} $$

$$ {}_1S_d = {}_1\Delta = {}_1S_a / {}_1\omega_e{}^2 = \frac{{}_1\{\delta\}^T [m]_1\{\delta\}}{{}_1\{\delta\}^T {}_1\{f\}} {}_1S_a \tag{14} $$

where, ${}_1S_a$: spectral response acceleration for the first mode, ${}_1S_d$: spectral response displacement for the first mode, ${}_1Q_b$: base shear corresponding to the first mode, ${}_1M_e$: effective modal mass corresponding to the first mode, ${}_1\Delta$: displacement at the equivalent height corresponding to the first mode, h_e, where the modal participation function, ${}_1\beta_1\{u\}$, ${}_1\omega_e$: effective circular frequency for the first mode, ${}_1\{\delta\}$: displacement vector for the first mode, $[m]$: lumped floor mass matrix, ${}_1\{f\}$: inertia force vector for the first mode, $\{1\}$: unit vector, ${}_1\beta$: modal participation factor for the first mode, and, ${}_1\{u\}$: mode shape vector for the first mode (normalized to the roof displacement, ${}_1\beta_1S_d$).

The effective first-mode circular frequency of the building at each loading step is approximately estimated by Eq. (15).

$$ {}_1\omega_e = \sqrt{\frac{{}_1K_e}{{}_1M_e}} = \sqrt{\frac{{}_1\beta_1\{u\}^T [k]_1\beta_1\{u\}}{\{1\}^T [m]_1\beta_1\{u\}}} = \sqrt{\frac{{}_1\{\delta\}^T {}_1\{f\}}{{}_1\{\delta\}^T [m]_1\{\delta\}}} \tag{15} $$

where, ${}_1K_e$: effective modal stiffness corresponding to the first mode, and, $[k]$: stiffness matrix of the building.

Consequently, using Eqs. (13) and (14), the external force and displacement 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).

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 in 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)).

$$ {}_{st}h_{eq} = \frac{1}{4\pi} \cdot \frac{\Delta W}{W} \tag{16} $$

where, ${}_{st}h_{eq}$: equivalent damping ratio of an ESDOF system under resonant stationary vibration, ΔW: dissipation energy of an ESDOF system, and, W: potential energy of an ESDOF system (${}_{1}Q_{b} \bullet {}_{1}\Delta/2$).

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

Eq. (16) 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 (Hiraishi 1999), the equivalent damping ratio is reduced to approximately 80 percent of that calculated by Eq. (16).

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 (MLIT 2000):

$$ h_{eq} = \frac{\sum {}_{m}h_{eqi}\, {}_{m}W_{i}}{\sum {}_{m}W_{i}} + 0.05 \tag{17} $$

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

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

$$ {}_{m}h_{eqi} = \gamma \left(1 - \frac{1}{\sqrt{\mu}} \right) \tag{18} $$

where, μ: ductility factor of a member reached at the ultimate state of a building.

The factor of γ is the reduction factor considering the damping effect for the transitional seismic response of the building (Shibata 1976). It takes the values of 0.25 for ductile members and 0.2 for non-ductile ones, respectively. When all structural members of a building structure have the same hysteretic characteristics, the equivalent damping ratio of an entire building can be estimated by Eq. (18).

Soil-structure Interaction Effects

The effective period and equivalent damping ratio should be modified by the following equations taking account of 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.

$$r = \sqrt{1 + \left(\frac{T_{sw}}{T_e}\right)^2 + \left(\frac{T_{ro}}{T_e}\right)^2} \tag{19}$$

$$h_{eq} = \frac{1}{r^3}\left\{ h_{sw}\left(\frac{T_{sw}}{T_e}\right)^3 + h_{ro}\left(\frac{T_{ro}}{T_e}\right)^3 + h_b \right\} \tag{20}$$

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. (21). The demand S_a-S_d spectra for different damping ratios are constructed using Eq. (21) as illustrated in Fig. 5(c).

$$S_d(T) = \left(\frac{T}{2\pi}\right)^2 S_a(T) \tag{21}$$

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:

$$F_h = \frac{1.5}{1 + 10h_{eq}} \tag{22}$$

where, F_h: response spectrum reduction factor.

Estimation of Ultimate Deformation of a Structural 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 goes up to the allowable stress of materials in any member or when the story drift reaches 1/200 of the story height at any story. The limit state of life safety is reached when the building can not 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 evaluated by Eq. (23).

$$R_u = R_b + R_s + R_x \tag{23}$$

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 from the actual condition corresponding to the structural type.

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

$$R_b = \frac{\phi_y a}{3} + (\phi_u - \phi_y) l_p \left(1 - \frac{l_p}{2a}\right) \tag{24}$$

where, ϕ_y: curvature of a member when the allowable stress is first reached in the member, ϕ_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.

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 (MLIT 2000), spectral acceleration of a building, defined by Eq. (13), at a limit state should not be lower than the corresponding acceleration of the demand spectrum using the effective period, corresponding to Eq. (15), and equivalent damping ratio, expressed by Eqs. (17) or (18), at the limit state.

CONCLUSIONS

This paper presents the seismic design provisions of BSLJ toward a performance-based structural engineering framework in 2000. The provisions provide 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 to take into consideration the soil conditions and soil-structure interaction effects as properly as possible. The return periods of the design earthquake motions of approximately 500 years and 50 years are expected 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 the structural components.

The features of the verification procedures of seismic performance against the design earthquake motions in the new provisions are the response spectrum analysis using the site-dependent response spectrum concepts, the equivalent linearization using the ESDOF modeling of a building, 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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