New Seismic Design Provisions in Japan

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Synopsis: The seismic design requirements in the Building Standard Law of Japan were revised in June 2000 toward a performance-based design framework. The performance objectives are (a) life safety and (b) damage control of a building at two corresponding levels of earthquake motions. The design earthquake motion is defined in terms of acceleration response spectrum at engineering bedrock. The amplification of ground motion by surface geology and the soil-structure interaction must be taken into consideration. The response is examined by so-called “capacity spectrum method” by comparing the linearly elastic demand spectrum of design earthquake motions and the capacity curve of an equivalent single-degree-of-freedom (ESDF) system. The structure as designed is reduced to an ESDF system using a nonlinear static analysis under monotonically increasing horizontal forces. Equivalent damping is used to modify the demand spectrum taking into account the energy dissipation capacity of a structure at the prescribed limit states.
INTRODUCTION

The first Japanese building law, Urban Building Law, was promulgated in 1919, to regulate building construction in six major cities at the time. Building Law Enforcement Regulations specified structural requirements. The regulations introduced a design seismic coefficient of 0.10 in 1924 after the 1923 Kanto (Tokyo) Earthquake.

Building Standard Law, applicable to all buildings throughout the country, was proclaimed in May 1950. The objectives were to safeguard the life, health, and properties of people by providing minimum standards concerning the site, structure, equipment, and use of buildings. The law outlines the basic requirements, and the technical details are specified in the Building Standard Law Enforcement Order (Cabinet Order) and in a series of Notifications by Minister of Construction.

The law requires that design documents and drawings should be submitted to a municipal government for the confirmation of the design documents to satisfy the legal provisions. This requirement made the code prescriptive because building officials must be able to judge the conformity. The structural design was based on the allowable stress design framework, using different allowable stresses for...
long-term (gravity) loading and short-term loading (seismic and wind forces). The structural requirements were revised from time to time after devastating earthquake disasters.

The seismic design provisions of the Building Standard Law Enforcement Order were significantly revised in 1981; major revisions in seismic design were listed below:

1. Structural calculation is required for (a) story drift under design earthquake forces, (b) lateral stiffness distribution along the height, (c) eccentricity of mass and stiffness in plan, and (d) story shear resisting capacity at the formation of a collapse mechanism,
2. Earthquake resistance is specified (a) in terms of story shear rather than horizontal floor forces, (b) as a function of fundamental period of a building and soil type, and (c) separately for the allowable stress design and the examination of story shear resisting capacity, and
3. Required story shear resisting capacity is varied for construction materials and with the deformation capacity of hinging members under earthquake forces.

The Building Standard Law was substantially revised in 1998 to introduce a performance-based design procedure to the existing prescriptive framework. New technical specifications in the form of the Law Enforcement Order and a series of Notifications of Ministry of Construction were issued in June 2000, including the definition of performance objectives for design limit states and the specifications for verification methods.

This paper introduces briefly the concept and framework of new provisions with emphasis on earthquake resistant building design.

**PERFORMANCE-BASED REQUIREMENTS**

The performance-based requirements in building codes are expected to expand the scope of structural design, especially for the application of new materials, construction and structural systems. It is further expected to remove international trade barriers in the construction markets and to encourage the engineer to develop new construction technology and engineering.

The new procedure, introduced in the Building Standard Law, deals with the evaluation and verification of performance (response) at a given set of limit states under gravity loads, snow loads, wind and earthquake forces. The structural specifications include the method of structural calculation, the quality control of construction and materials, durability of buildings, and the performance of nonstructural elements. For continuity, the design loads and forces were maintained at the same levels as the existing provisions. However, a new format of seismic design forces was introduced; i.e., the response acceleration spectrum of
the earthquake motion is specified at engineering bedrock, having shear wave velocity in the range of several hundred meters per second. The amplification of ground motion by surface geology above the engineering bedrock must be duly taken into account in defining the design ground motion at the free surface.

Two limit states are considered as the minimum standards for building structures to safeguard the life and property of the inhabitants; i.e., (a) life safety and (b) damage initiation. Two sets of design loadings are considered, each having a different probability of occurrence. The structural damage should be prevented in events that may occur more than once in the lifetime of the building for the protection of properties; i.e., the damage must be prevented in structural frames, members, interior and exterior finishing materials. A return period for such events may be 30 to 50 years. For the protection of human life, no story of the building should collapse under extraordinary loading conditions. The maximum possible earthquake motion level is determined on the basis of historical earthquake data, recorded strong ground motions, seismic and geologic tectonic structures and identified activities of active faults. A return period of several hundred years is assumed in defining the design earthquake motions.

**DEFINITION OF DESIGN EARTHQUAKE MOTIONS**

The design seismic forces were previously specified in terms of story shear forces as a function of building period and soil conditions. In other words, the design seismic forces were specified as the response quantities of a structure without defining the ground motion and the response amplification by a specific structure.

In the revised provisions, the acceleration response spectrum $S_A(T)$ of free surface ground motion at a 5% damping factor is represented as follows:

$$S_A(T) = Z \cdot G_s(T) \cdot S_0(T) \quad (1)$$

where $Z$ is the seismic zone factor, $G_s(T)$ is the amplification factor by surface geology, $S_0(T)$ is the response spectral acceleration ordinate of ground motion at exposed engineering bedrock, and $T$ is the period of a building in sec at the damaged state.

**Earthquake Motion at Engineering Bedrock**

The ground motion is defined by an acceleration response spectrum at exposed engineering bedrock. The engineering bedrock is defined as a thick soil stratum whose shear wave velocity is on the order of 400 m/s or higher. The exposed
The engineering bedrock is used in the definition to eliminate the effect of the surface geology on the ground motion.

The acceleration response spectrum at the engineering bedrock consists of a uniform acceleration portion in a short period range and a uniform velocity portion in a long period range. For the sake of continuity in seismic design provisions, the intensity of ground motion at the engineering bedrock was established to yield design seismic forces that are comparable to those for intermediate soil condition before the revision of the Building Standard Law. Therefore, the constant acceleration and velocity response spectral ordinates for the life-safety events are specified to be 8.0 m/sec² and 815 mm/sec, respectively, at a 5% damping ratio on exposed engineering bedrock.

The design spectrum $S_o(T)$ at exposed engineering bedrock is shown in Fig. 1 or given by Eq. (2) for the life-safety limit state:

\[
S_o(T) = (3.2 + 30T) \quad \text{for} \quad T < 0.16 \\
S_o(T) = 8.0 \quad \text{for} \quad 0.16 \leq T < 0.64 \\
S_o(T) = \frac{5.12}{T} \quad \text{for} \quad 0.64 \leq T
\]

where $S_o(T)$ is the spectrum ordinate (m/sec²), $T$ is the period (sec) of the building at the life-safety limit state.

The design spectrum for the damage-initiation limit state is to be reduced to one-fifth of the spectrum for the life-safety limit state.

Amplification Factor for Surface Geology

The amplification of ground motion by surface geology is evaluated using the geological data at the site. The nonlinear amplification of ground motion by surface soil deposits is estimated using the equivalent linearization technique. A lumped-mass shear-spring model (Fig. 2) was used to represent a layer of soil deposits; the stiffness of soil layers was represented by secant shear modulus at maximum response shear strain under the first mode oscillation. The shear modulus reduction factors and damping factors are specified for cohesive and sandy soils at various shear strain levels in Notification 1457 of the Ministry of Construction.

The equivalent shear wave velocity and impedance of an equivalent uniform soil layer were estimated for the equivalent linear shear model. The amplification of ground motion by a uniform soil layer above the engineering bedrock is obtained by considering one-dimensional wave propagation in the frequency domain. The
dynamic amplification function by the surface geology is modified by connecting the two peak points of the first and second modes by a straight line. The basis and reliability of this procedure was examined for different soil deposits and the results were reported in Ref. (1).

The following expressions are given for the amplification function $G_s(T)$ by surface geology in Notification 1457 of the Ministry of Construction;

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

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

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

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

where $T_1$ and $T_2 = (T_1/3)$ are the dominant periods of surface soil deposits, $G_{s1}$ and $G_{s2}$ are the amplification factors of the soil deposits in the first and second modes. The first-mode period is estimated on the basis of the depths, shear moduli at strain amplitude and mass density of the soil layers. Empirical formulae are provided to determine the first- and second-mode amplification factors $G_{s1}$ and $G_{s2}$ considering the hysteretic energy dissipation and impedance ratios.

If the detailed analysis is not used, the following simple expression can be used;

1) For soil type I (soil layer consisting of rock, stiff sand gravel, and pre-Tertiary deposits);

$$G_s = 1.5 \quad \text{for} \quad T < 0.576$$

$$G_s = \frac{0.864}{T} \quad \text{for} \quad 0.576 \leq T < 0.64$$

$$G_s = 1.35 \quad \text{for} \quad 0.64 \leq T$$

where $T$ is the period of a structure (sec).

2) For soil type II (soil layer other than types I and III) and type III (alluvium layer mainly consisting of humus and mud whose depth is more than 30 m, or filled land of more than 3 m deep and worked within 30 years);
\begin{align}
G_s &= 1.5 \quad \text{for} \quad T < 0.64 \\
G_s &= 1.5 \left( \frac{T}{0.64} \right) \quad \text{for} \quad 0.64 \leq T < 0.64 \left( \frac{g_v}{1.5} \right) \\
G_s &= g_v \quad \text{for} \quad 0.64 \left( \frac{g_v}{1.5} \right) \leq T
\end{align}

where \( g_v = 2.03 \) for type II soil and 2.7 for type III soil.

Seismic Zoning Factor

The advancement in the simulation methodology and the collection of strong motion records in near-source regions in this decade made it feasible to estimate realistic intensity and characteristics of earthquake motions at engineering bedrock. The seismic zone factor evaluates (a) relative difference in expected ground motion parameters, such as peak ground acceleration or peak ground velocity for strong and intermediate intensity earthquake motions, and (b) frequency content for acceleration and velocity waveforms. Two levels of ground motion are defined; i.e.,

1) Large earthquake: largest annual maximum in 500 years, and
2) Intermediate earthquake: 10th largest annual maximum in 500 years.

The historical earthquake data over the last 500 years in Japan and fault parameters identified for major earthquakes were used in the study.

Two empirical attenuation formulae for near-source and far-field events estimated peak ground acceleration and velocity amplitudes for various subdivided regions of the country as a function of earthquake magnitudes, distance to the fault plane and average shear wave velocity of the upper 30-m surface soil deposit. Regional seismic maps were drawn for peak ground acceleration and velocity amplitudes, separately, expected in 50 years and several hundred years.

The expected intensity levels estimated for the 500-year return period are comparable with or slightly larger than the level of seismic force currently in use for type-II soil. Therefore, the seismic zone factors, varying from 0.7 to 1.0, in the previous Building Standard Law Enforcement Order are maintained in the revised design requirements.

VERIFICATION OF STRUCTURAL PERFORMANCE

The performance of a building is examined at the two limit states under the two levels of design earthquake motions; i.e., (a) damage-initiation limit state and (b)
life-safety limit state.

The damage-initiation limit state is attained when the allowable stress of materials has been reached in any member or when the story drift reaches 0.5 percent of the story height at any story. The initial elastic period is used for a structure.

The life-safety limit state is attained when the structure cannot sustain the design gravity loads in any story under additional horizontal deformation; i.e., a structural member has reached its ultimate deformation capacity. The ultimate deformation of a member must be calculated as the sum of flexure and shear deformations of the member and deformation resulting from the deformation in the connection to adjacent members. The ultimate flexural deformation $\theta_{fu}$ may be estimated as

$$
\theta_{fu} = \frac{\phi_y}{3} a + (\phi_u - \phi_y) \ell_p \left(1 - \frac{\ell_p}{2a}\right)
$$

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

**Equivalent SDF Modeling**

A multi-story building structure is reduced to an equivalent single-degree-of-freedom (ESDF) system (Fig. 3) using the results of a nonlinear static analysis under constant-amplitude gravity loads and monotonically increasing horizontal forces (often called a “pushover analysis”). The distribution of story shear coefficients (design story shear divided by the weight supported by the story) is defined by the following expression, consistent with the previous Building Standard Law:

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

$$
\alpha_i = \frac{\sum_{j=1}^{i-1} W_j}{\sum_{j=1}^{n} W_j}
$$

where $W_i$ is the sum of dead and live loads at $i$-th floor, and $T$ is the fundamental period of the structure. Horizontal force acting at a floor level is calculated as the difference of story shears immediately above and below.
The deflected shape of the pushover analysis is assumed to represent the first-mode shape of oscillation. The deflected shape does not change appreciably with the distribution of horizontal forces along the structural height; therefore, the constant force distribution is used during the pushover analysis.

The modal participation factor $\Gamma_i$ is necessary to relate the SDF response and modal response of a structure under horizontal ground motion; i.e.,

$$\Gamma_i = \frac{\{\phi_i\}^T [m] \{1\}}{\{\phi_i\}^T [m] \{\phi_i\}}$$

(9)

where $\{\phi_i\}$ is the first-mode shape vector (normalized to the roof-level displacement), $[m]$ is the lumped floor mass matrix (diagonal matrix), and $\{1\}$ is a vector whose elements are unity.

For a spectral response acceleration $S_A(T)$ and displacement $S_D(T)$ at the first-mode period and damping, the first-mode inertia force vector $\{f\}_i$ and displacement vector $\{d\}_i$ are defined as follows;

$$\{f\}_i = [m] \{\phi\}_i \Gamma_i S_A(T)$$
$$\{d\}_i = \{\phi\}_i \Gamma_i S_D(T)$$

(10)

For the mode shape vector normalized to the roof-level displacement, the roof displacement $D_{Ri}$ is equal to $\Gamma_i S_D(T)$. The first-mode base shear $V_{B_i}$ is the sum of inertia forces at each floor level. For the lumped floor masses, the base shear is calculated as follows;

$$V_{B_i} = \{1\}^T \{f\}_i$$
$$= \{1\}^T [m] \{\phi\}_i \Gamma_i S_A(T)$$
$$= M_i^* S_A(T)$$

(11)

where $M_i^*$ is the effective modal mass as given below;

$$M_i^* = \Gamma_i \{\phi\}_i^T [m] \{1\}$$

(12)

The effective mass must be not less than 0.75 times the total mass of the structure.
Capacity Curve

In general, the roof-level displacement $D_R$ and the base shear $V_B$ are governed by the first-mode response. Therefore, the base shear $V_B$ divided by the effective modal mass $M_1^*$ and the roof displacement $D_R$ divided by the participation factor $\Gamma_1$ represent the response spectral acceleration $S_A(T)$ and displacement $S_D(T)$:

$$S_A(T) = \frac{V_B}{M_1^*}$$
$$S_D(T) = \frac{D_R}{\Gamma_1}$$

The relation between $S_D(T)$ and $S_A(T)$ may be plotted for a structure under monotonically increasing horizontal forces. The relation is called the “capacity curve” of the structure (Fig. 4).

The effective first-mode period $T_e$ of a structure at a loading stage is approximated by the following relation (Fig. 4):

$$T_e = 2\pi \sqrt{\frac{S_D(T)}{S_A(T)}}$$

It should be noted that the effective period changes with the amplitude of horizontal forces and displacements. The effective period may be modified by the following factor $r$ taking into account the effect of soil-structure interaction:

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

where $T_{sw}$ is the period of sway oscillation, and $T_{ro}$ is the period of rocking oscillation. The sway and rocking periods must be evaluated for the stiffness of soil corresponding to the excitation level of the super-structure.

Equivalent Damping Ratio

Equivalent damping ratio for the first mode is prescribed to be 0.05 for the damage-initiation limit state because the state of a structure remains elastic at this stage.
Equivalent viscous damping ratio $h_{eq}$ at the life-safety limit state is defined by equating the energy dissipated by hysteresis of a nonlinear system and the energy dissipated by a viscous damper under resonant steady-state vibration:

$$h_{eq} = \frac{1}{4\pi} \frac{\Delta W}{W}$$  \hspace{1cm} (16)

where $\Delta W$ is the hysteresis energy dissipated by a nonlinear system during one cycle of oscillation, and $W$ is the elastic strain energy stored by a linearly elastic system at the maximum deformation (Fig. 5).

Naturally, such equivalence does not hold in the case of response under random earthquake excitation. The equivalent damping ratio must be effectively reduced to correlate the maximum response of an equivalent linear system and a nonlinear system under a random earthquake excitation.

A series of nonlinear SDF systems having different hysteretic characteristics (bilinear, degrading bilinear, slip bilinear and Takeda models) and equivalent linear SDF systems were analyzed under natural and artificial earthquake motions in Ref. (2). Analytical parameters include elastic periods and yielding resistance. The maximum response of equivalent linear SDF systems was reported to be comparable to that of nonlinear systems when the equivalent damping ratio is reduced to approximately 70 percent of that calculated by Eq. (16). The reduction was observed to increase with ductility demand of nonlinear systems.

Therefore, the equivalent damping ratio $m h_{eq}$ of a structural member $i$ is estimated by the following expression;

$$m h_{eq} = \frac{1}{4} \left(1 - \frac{1}{\sqrt{\mu}}\right)$$  \hspace{1cm} (17)

where $\mu$ is the ductility factor of the member attained at the life-safety limit state of the structure. If the hysteresis shape of a member exhibits a slip-type characteristic, Eq. (18) must be used;

$$m h_{eq} = \frac{1}{5} \left(1 - \frac{1}{\sqrt{\mu}}\right)$$  \hspace{1cm} (18)

The equivalent damping ratio of an SDF system is estimated as the weighted average with respect to strain energy;
\[ h_{eq} = \frac{\sum_m h_{eq, m} W_m}{\sum_m W_m} + 0.05 \]  

(19)

where \( W_m \) is the strain energy stored in member \( i \) at the life-safety limit state.

The equivalent damping ratio may be modified considering the soil-structure interaction effect:

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

(20)

where \( r \) is the period modification factor defined in Eq. (15), \( h_b \) is the damping ratio of the super-structure, \( h_{sw} \) is the damping ratio of sway oscillation of surface soil layers corresponding to shear strain level considered, but the value is limited to 0.30, \( h_{ro} \) is the damping ratio of rocking oscillation or surface soil deposits corresponding to shear strain level considered, but the value is limited to 0.15, \( T_{sw} \) and \( T_{ro} \) are the sway and rocking oscillation periods at the life-safety limit state, and \( T_e \) is the period of a structure at the life-safety limit state as defined by Eq. (14).

**Demand Spectrum**

Response spectral displacement \( S_D(T) \) is estimated from the linearly elastic design acceleration response spectrum \( S_A(T) \) at the free surface by dividing the spectral acceleration by the square of a circular frequency \((= T / 2\pi)\):

\[ S_D(T) = \left( \frac{T}{2\pi} \right)^2 S_A(T) \]  

(21)

The demand spectrum (Ref. 3) is constructed by plotting an SDF response acceleration in the vertical axis and corresponding displacement in the horizontal axis along a straight line with slope equal to the square of a circular frequency. The period (circular frequency) of SDF systems was gradually varied in the plot (Fig. 6).

Demand spectra are prepared for a damping ratio of 0.05 up to the damage-initiation limit state, and for an equivalent damping ratio at life-safety limit state. For the life-safety limit state, the response spectral acceleration and displacement are reduced by the following factor \( F_h \):
\[ F_h = \frac{1.5}{1 + 10h_{eq}} \]  \hspace{1cm} (22)

where \( h_{eq} \) is the equivalent damping ratio defined by Eq. (19).

Performance Criteria

The performance of a structure under a given design earthquake motion is examined by comparing the capacity diagram of the structure and the demand spectrum of design earthquake motions. The intersection (called performance point) of the demand spectrum for an appropriate equivalent damping ratio and the capacity curve represents the maximum response under the design earthquake motion if the damping ratio of the demand spectrum is equal to an equivalent damping ratio of the SDF system evaluated at the deformation at the performance point (Fig. 7). A continuous capacity curve is not necessary in design, but two points on the capacity curve must be evaluated for the two limit states.

The Building Standard Law Enforcement Order requires that spectral acceleration of a structure, defined by Eq. (13), at a limit state should be higher than the corresponding acceleration of the demand spectrum using the equivalent damping ratio, expressed by Eq. (18) or (19) at the same limit state.

The Building Standard Law Enforcement Order further requires that the exterior finishing and curtain walls should not fail under the design loads and displacements at the life-safety limit state. This requirement is intended to limit the story drift of the structure to a reasonable range.

SUMMARY

This paper presented an evaluation procedure of structural seismic performance under major earthquake motions introduced in June 2000 in the revised Building Standard Law and associated regulations. Life safety and damage control of a building are two performance objectives in the seismic provisions.

The earthquake motions are defined by acceleration response spectrum that is specified at engineering bedrock in order to consider the soil condition and soil-structure interaction effect. The return periods of the earthquake motion of approximately 500 years and approximately 50 years are considered for life-safety and damage-initiation limit states, respectively.

The capacity spectrum method is used to examine the damage initiation and
life-safety of a building. A structure as designed is reduced to an equivalent SDF system by the use of a pushover analysis. A linearly elastic capacity spectrum at a 0.05 damping ratio is modified for an equivalent damping corresponding to the displacement of the nonlinear SDF system at the two limit states.

REFERENCES


Fig. 1: Design earthquake acceleration response spectrum for life-safety limit state on exposed engineering bedrock

Fig. 2: Equivalent single-layer soil model (\( \rho \): mass density, \( G \): shear modulus, \( V \): shear wave velocity, \( h \): damping factor, \( d \): layer depth, \( m \): mass, \( K \): stiffness, and \( c \): damping coefficient)
Fig. 3: Reduction of a structure to a single-degree-of-freedom system by pushover analysis and equivalent viscous damping using hysteretic energy dissipation

Fig. 4: Capacity curve of a structure in terms of spectral acceleration $S_A(T)$ and displacement $S_D(T)$
Fig. 5: Equivalent viscous damping ratio for hysteresis energy dissipation

\[ \omega^2 = \left( \frac{2\pi}{T_e} \right)^2 \]

Fig. 6: Formulation of demand spectrum of design earthquake motion
Fig. 7: Performance criteria using demand spectrum of design earthquake motions and capacity curve of an equivalent SDF system.
Keywords:
Seismic design, Performance-based design, Capacity spectrum, Demand spectrum, Capacity curve, SDF system, pushover analysis, Life safety, Damage initiation, Limit states