DEVELOPMENT OF SEISMIC PERFORMANCE EVALUATION PROCEDURES IN BUILDING CODE OF JAPAN

By

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ABSTRACT

The building code of Japan will be changed from current prescriptive into performance-based type by 2000.
This paper presents the evaluation procedure of structural seismic performance against major earthquake motions in the performance-based building code of Japan under development at the Building Research Institute (BRI). The basic concept of BRI proposal for seismic design spectra for major earthquake motions is 1) basic design spectra defined at the engineering bedrock, and 2) evaluation of site response from geotechnical data of surface soil layers. The principle of evaluation procedures is that the predicted response values should not exceed the estimated limit values. In case of major earthquakes, the maximum response values of strength and displacement of a structure should be smaller than the ultimate capacity for strength and displacement. The proposed evaluation procedure applies the equivalent single-degree-of-freedom (ESDOF) system and the response spectrum method, while the current procedures are based on the estimation of the ultimate capacity for lateral loads.
The proposed evaluation procedure makes it realistic and simple to predict maximum structural response in case of major earthquakes as well as to confirm whether the predicted response values are smaller than the limit ones.

KEYWORDS: *building code of Japan, evaluation procedure, seismic performance, major earthquake motions, equivalent linearization, equivalent single-degree-of-freedom system, response spectrum method, site response, soil-structure interaction*

1. INTRODUCTION

On February in 1996 it was officially announced by the Ministry of Construction authorities that the Building Standard Law of Japan should be revised on the basis of performance principles. In the Building Council report[1] of March 24, 1997, entitled "For a New Building Administration Framework, which could cope with the economic & social changes and their prospects in the 21st century", it was clearly stated that in order to compile a highly flexible New Building Standard Law, the current provisions must be revised into those based on performance. That served as a basis for the compilation of "Guidelines for the Performance-based Building Code".

Unlike the in-current-use conventional building code, the performance-based building code prescribes clearly the type and the level of the required performance for a given building structure. In other words, for the precisely

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determined response of the structure subjected to assumed loads and forces, it prescribes the evaluation procedures to be used for the estimation of structure's conformity with the required structural safety.

However, even in the case of the performance-based provisions, the prescription of a minimum required level is necessary, being the same as in the current building code provisions. In this sense, the performance-based code is different form the performance-based design, which has been a current topic among structural engineers. The latter deals with a design procedure, which is based on a clearly defined target performance for the structure considered, normally being prescribed higher than the minimum required performance level. Basically, the performance-based design is the structural design on the basis of consultation between the structural engineer and the owner of the building.

This paper presents the evaluation procedure of structural seismic performance against major earthquake motions in the performance-based building code of Japan under development at the Building Research Institute (BRI)[2), 3), 4)].

The basic concept of BRI proposal for seismic design spectra for major earthquake motions is 1) basic design spectra defined at the engineering bedrock, and 2) evaluation of site response from geotechnical data of surface soil layers.

The principle of evaluation procedures is that the predicted response values should not exceed the estimated limit values. In case of major earthquakes, the maximum response values of strength and displacement of a structure should be smaller than the ultimate capacity for strength and displacement. The proposed evaluation procedure applies the equivalent single-degree-of-freedom (ESDOF) system and the response spectrum method, while the current procedures are based on the estimation of the ultimate capacity for lateral loads.

It should be noticed here that presented in this paper is just the state-of-the-art and up-to-now level of development of the performance-based structural provisions at BRI in Japan. It should not be considered as a fully completed version, ready to be presented for the final official approval.

2. CONCEPTUAL FRAMEWORK OF PERFORMANCE-BASED STRUCTURAL CODE

The conceptual framework of performance-based structural code proposed by BRI is shown in Fig. 1. Following the principles of structural safety, the evaluation procedures to be used for the estimation of structure's conformity with the required performance level are roughly classified as:

a. Proposed route
b. Conventional route
c. Small building route, and
d. Others

The proposed route represents a new evaluation procedure to be used instead of the current one, which is based on the calculation of allowable stress and estimation of ultimate capacity for lateral load. It considers the effects of major earthquakes as well as other forces and loads. The other effects, which are not considered in the structural calculations, such as construction quality, durability, quality of construction materials, and nonstructural elements, are covered by structural specifications. In essence, by using this procedure it is possible to evaluate and verify the structural performance possessed by a designed structure, regardless of the design method used. It is just an evaluation procedure that verifies whether or not the prescribed performance objectives are met.

The second route represents the conventional evaluation procedure now in use, adopted as the standard structural calculation method. It can be supplemented with additional provisions in addition to those of the first route described above. However, if the principles of performance-based provisions are to be followed, it should be noticed that the obviously unnecessary parts to be considered by structural calculations are eliminated. To this extent, this route can be considered as a kind of deemed-to-satisfy evaluation procedure.

The third route applies to small buildings. This route does not require structural calculations and is considered to be deemed-to-satisfy provisions. It prescribes only conventional-based structural specifications.

In the fourth route are included all other alternative evaluation procedures and deemed-to-satisfy provisions, such as those developed and certified by private institutions as well as those requiring expert judgments.

The types of loads and forces considered in the newly proposed evaluation procedure remain almost the same with those currently in use. However, for the case of seismic effects only, new earthquake motion provisions are prepared to replace the current earthquake force provisions.

In a definite proposal, the earthquake motion response spectra at the engineering bedrock, assumed to be the stratum having shear wave velocity in the range of several hundreds m/s, is considered as the basic design spectra. On the basis of this conception for the earthquake input motion, it is possible that earthquake effects be not only accounted rationally through the incorporation of influence of local soil conditions on ground motion characteristics at the free surface but also conveniently incorporated in the newly developed design procedures of seismically isolated and response controlled structures. Furthermore, it is anticipated that the future proposals expected for the evaluation and design procedures are suitably implemented.

3. REQUIRED SEISMIC PERFORMANCE LEVEL FOR BUILDING STRUCTURES

An outline of requirements for building structures and earthquake motion levels is shown in Table 1. In the vertical column on the left hand side of the table are shown the requirements for building structures, while in the rest of the table are shown the earthquake motions to be considered and their corresponding levels for each of the requirements assigned for building structures.

As it is shown in Table 1, requirements for building structures are classified in two categories, which are explained below.

3.1 Life Safety

The essential purpose of this requirement is the safety of life. It should be expected that under the action of earthquake motions taken into consideration, the building should not experience any story collapse.

3.2 Damage Limitation

The aim of this requirement is damage limitation. Under this provision, it is required first that after the action of earthquake motions taken into consideration, no structural damage which could threaten the structural safety of the building will take place. In other words, the structural safety performance required by Section 3.1 should be preserved. Furthermore, it is required that no other kind of damage causing in the building structure a situation which does not comply with other requirements of the Building Standard Law concerning fire safety should be experienced.

3.3 Maximum Earthquake Motion Level

This level of earthquake motions corresponds to the category of requirements in Section 3.1 for building structures and is assumed to produce the maximum possible effects on the structural safety of a building to be constructed at a given site. The maximum possible earthquake motion level is determined on the basis of historical earthquake data, recorded strong ground motions in the past, seismic and geologic tectonic structures, active faults, and others. This earthquake motion level corresponds nearly to that of highest earthquake forces used in the current seismic design practice, representing the horizontal earthquake forces induced in the building structures in case of major seismic events.

3.4 Once-in-a-Lifetime Event Level

This level of earthquake motions corresponds to the category of requirements in Section 3.2 for building structures and is assumed to be experienced more than once during the lifetime of the building. A return period interval of 30-50 years is supposed to cover these events. This level of earthquake motion corresponds nearly to the middle level earthquake forces used in the current seismic design practice, representing the horizontal earthquake forces induced in the building structures in case of moderate earthquakes.

4. EVALUATION PROCEDURES FOR A REQUIRED PERFORMANCE LEVEL

Various response and limit values are considered for use in proposed evaluation procedures, in accordance with each of the requirements prescribed for building structures. A representative example of this arrangement is shown in Table 2. The principle of evaluation procedures is that the predicted response values due to the action of earthquake motions on building structures should not exceed the estimated limit values. Fundamentals of proposed evaluation procedures corresponding to each level of earthquake motions are described below.

4.1 Evaluation Procedures Corresponding to Maximum Earthquake Motion Level

The maximum response values of the building structure subjected to earthquake motions should be smaller than the limit values. In defining strength and displacement limits for earthquake motions, it may be necessary to consider the effects of repeating cycles in the plastic region of the response as well.

4.2 Evaluation Procedures Corresponding to Once-in-a-lifetime Event Level

For this level of earthquake motions, it is required to be confirmed whether the internal forces and displacements taking place at each structural element satisfy the condition of being smaller than the limit strengths and displacements. The limit strengths and displacements mentioned here imply that the whole structure behaves generally within the elastic range.

5. PROPOSED EVALUATION PROCEDURE FOR THE CASE OF MAJOR EARTHQUAKES

Hereafter the focus is put on the proposed evaluation procedure for the case of major earthquakes.

The variety of linearization techniques has already been studied (for example, Ref. 5). Several applications have also been presented in the publicetions[6)-9)].

In the analytical methods to be used for predicting the structural response in the newly proposed evaluation procedure for the case of earthquake excitations, it is expected to apply the ESDOF system and the response spectrum method using the linearization technique.

5.1 General Flow of Proposed Evaluation Procedure

A flow chart of this procedure is illustrated in Fig. 2. There are indeed various analytical methods for predicting the response of structures subjected to earthquake excitations. The one that is shown here is based on the ESDOF system and the response spectrum method.

According to this procedure the steps to be followed are:

I. Confirm the scope of application of the evaluation procedure and the mechanical characteristics of materials and/or members to be used.

II. Determine the response spectra to be used in the evaluation procedure.

i) For a given basic design spectrum at the engineering bedrock level, draw up the free-field site-dependent acceleration (Sa) and displacement response spectra (Sd), for different damping levels.

ii) In the estimation of free-field site-dependent acceleration and displacement response (step i) above), consider the strain-dependent soil deposit characteristics.

iii) In case of need, present graphically the relation of Sa-Sd, for different damping levels.

III. Determine the hysteretic characteristic, equivalent stiffness and equivalent damping ratio of the structure.

i) Model the structure as a simplified ESDOF system and establish its force-displacement relationship (see Fig. 2a).

ii) Determine the limit strength and displacement of the structure corresponding to the ESDOF system mentioned above.

iii) The soil-structure interaction effects should basically be considered.

iv) In case of need, determine the equivalent stiffness in accordance with the limit values.

v) Determine the equivalent damping ratio on the basis of viscous damping ratio, hysteretic dissipation energy and elastic strain energy of the

structure (see Fig. 2b).
vi) In case that the torsional vibration effects are predominant in the structure, these effects should be considered when establishing the force-displacement relationship of the ESDOF system.
IV. Examine the safety of the structure.

In this final step, it is verified whether the response values predicted on the basis of the response spectra determined according to the step II satisfy the condition of being smaller than the limit values estimated on the basis of step III (see Fig. 2c).

In order to determine the limit strength and displacement of the structure, a specific displaced mode is necessary to be assumed in advance for its inelastic response (see Fig. 2a). Basically, any predominant or possible to be experienced displaced mode of the structure subjected to earthquake motions can be applied. The predominant or possible to be experienced displaced mode implies any of the failure modes observed during the major earthquakes such as beam failure mode, story failure mode or any other definite failure mode.

5.2 Basic Response Spectrum at Engineering Bedrock

In the BRI proposal, the evaluation earthquake load is specified with earthquake ground motion not with seismic force. The earthquake ground motion is basically given at the exposed (outcrop) engineering bedrock. The evaluation earthquake motion is represented with the acceleration response spectrum in the following formula.

$$S_a = ZGS_0 \quad (1)$$

where,

S_a: acceleration response spectrum for evaluation,

Z: seismic zoning factor,

G: soil amplification factor, and

S_0: basic acceleration response spectrum at engineering bedrock.

The engineering bedrock is defined as a layer with more than 400 m/s in shear wave velocity.

The basic concept in estimating the evaluation earthquake motions is expressed in the followings.

The basic acceleration response spectra to be given at the engineering bedrock shall be consistent with the design seismic shear force given in the current Building Standard Law of Japan.

In this case, the consistency is maintained so that the newly defined evaluation spectrum preserves the equivalent seismic effects in the seismic forces specified for buildings constructed on the soil condition with soil profile of Type-2, which contains the majority of the ordinary buildings in Japan.

The basic response spectrum shown in Fig. 3 is to be given within specific ranges of period. The shorter period range is determined with uniform acceleration amplitude and the longer period range is determined with the uniform velocity amplitude. Each level of spectra is understood simply related with the peak ground acceleration (PGA), and the peak ground velocity (PGV) at the engineering bedrock expected at the site.

The relationship between the base shear coefficient and the acceleration response spectrum is given with the earthquake response of the uniform shear beam model. It is reported that in this model the base shear force shall be multiplied by 1.23 for uniform acceleration spectrum, and shall be multiplied by 1.1 for uniform velocity spectrum[10]. In addition to this simplified relationship, we assumed the soil amplification factors 1.5 for acceleration dominated period range, and 2.0 for velocity dominated period range. These factors of 1.5 and 2.0 are based on the nonlinear response computations using simplified models of surface soil layers. With these assumptions, the basic acceleration response spectrum is derived as shown in Fig. 3.

The seismic zoning factor represents the relative difference in the expected earthquake intensity values for specified periods. The earthquake intensity values are represented with peak ground acceleration (PGA) and peak ground velocity (PGV). Based on the earthquake source catalogue during the recent 500 years larger than the magnitude of 5.0, and the empirical attenuation equations which are consistent with the near source records during recent major earthquakes such as the 1995 Hyogoken-nambu earthquake, we computed the expected amplitudes in acceleration and velocity for 500 and 50 years for every 0.05 degree in longitude and latitude point. Here, we considered the source area for larger earthquakes whose fault model is available. And we also included the influence of active faults. The major active faults in Japan are being investigated for its possible return period and probabilities for causing earthquakes in near future. Based on these computation results, we drew the hazard maps for the representative values as shown in Fig. 4. Our proposal on the seismic zoning factors will be based on these results.

5.3 Acceleration Response Spectrum at Ground Surface

To evaluate the acceleration response spectrum at ground surface, the amplification of surface soil deposits on the engineering bedrock is estimated. An evaluation method by using the equivalent linearization technique considering soil properties of nonlinearity is explained below.

(1) Evaluation procedure

The acceleration response spectrum at the ground surface is obtained as follows.

a) Transformation of response spectrum defined at outcropped engineering bedrock

The earthquake motion defined at the outcropped engineering bedrock is given as the acceleration response spectrum with 5% damping ratio; $S_{ao}(T, \zeta=0.05)$. The acceleration response spectrum with arbitrary damping ratios; $S_{ao}(T, \zeta)$ is calculated based on $S_{ao}(T, \zeta=0.05)$. $S_{ao}(T, \zeta=0)$, a velocity response spectrum; S_v, and a Fourier spectrum of acceleration; $F_{ao}(T)$ have the approximate relation as follows.

$$F_{ao}(T) \fallingdotseq S_v(T, \zeta=0) \fallingdotseq (T/2\pi) S_{ao}(T, \zeta=0) \qquad (2)$$

b) Eigen value analysis of soil profile

With subdividing the soil profile, a shear model of n degrees of freedom is formed, as shown in Fig. 5. The shear springs; K_i, damping coefficients; c_i and masses; m_i at discrete points, and a spring at the bottom of the surface soil layers are defined as follows.

$$K_i=G_i/d_i \qquad c_i=h_i\, G_i\, T_i/(\pi d_i) \qquad (3)$$

$$m_i=0.5(\rho_i\, d_i+\rho_{i-1}\, d_{i-1}) \qquad (4)$$

$$K_b=8\, G_b\, B/(2-\nu_b) \qquad (5)$$

Where G_i, ρ_i, d_i and h_i are shear modulus, mass density, layer height, and damping ratio at the i-th layer from the surface. G_b and υ_b are shear modulus and Poisson's ratio at the engineering bedrock, and B = 0.564m. T_1 is the fundamental natural period of the surface ground. Through the eigen value analysis, the natural period, the vibration mode; U_i (normalized by the value at the surface) and the modal damping ratio; ζ_1 are obtained.

c) Equivalent shear wave velocity and impedance

The surface soil layers is replaced to an uniform stratum with an equivalent shear wave velocity; V_{se} and an equivalent mass density; ρ_e and an equivalent damping ratio; ζ_1, which are calculated from properties in each layer.

$$V_{se}=\frac{1}{H}\sum_{i=1}^{n-1} V_{si} d_i \qquad (6)$$

$$\rho_e=\frac{1}{H}\sum_{i=1}^{n-1} \rho_i d_i \qquad (7)$$

where, $V_{si}=\sqrt{(G_i/\rho_i)}$ and H is the total thickness of the surface soil layers. The impedance of a wave motion; α between the equivalent uniform surface ground and the engineering bedrock is expressed as follows.

$$\alpha=(\rho_e\, V_{se})/(\rho_b\, V_{sb}) \qquad (8)$$

where, V_{sb}: shear wave velocity, and

ρ_b: mass density.

d) Amplification of surface ground

The amplification, in frequency domain, of the uniform surface ground to the outcropped engineering bedrock is obtained by using the one-dimensional wave propagation. The transfer function of the surface ground and the engineering bedrock to the outcropped one are expressed as follows.

1) surface/outcropped engineering bedrock;

$G_s(T, \zeta_1, \alpha)$

2) engineering bedrock /outcropped;

$G_b(T, \zeta_1, \alpha)$

e) Response acceleration and displacement at ground surface at fundamental natural period T_1

The response acceleration of the ground surface; $A_s(T_1)$ and of the engineering bedrock; $A_b(T_1)$, and their response displacements; $D_s(T_1)$ and $D_b(T_1)$ are obtained as the product of the Fourier amplitude of the outcropped engineering bedrock; $F_{ao}(T_1)$ and the amplification factor of the surface ground.

$$A_s(T_1)=(1/T_1)\, G_s(T_1, \zeta_1, \alpha)\, F_{ao}(T_1) \qquad (9)$$

$$A_b(T_1)=(1/T_1)\, G_b(T_1, \zeta_1, \alpha)\, F_{ao}(T_1) \qquad (10)$$

$$D_s(T_1)=(T_1/2\pi)^2 A_s(T_1) \quad (11)$$
$$D_b(T_1)=(T_1/2\pi)^2 A_b(T_1) \quad (12)$$

f) Nonlinearity of surface ground

The relative displacement; u_i of the i-th mass points to that of the engineering bedrock is estimated by the following equation.

$$u_i=\{D_s(T_1)-D_b(T_1)\} \quad (13)$$

An effective strain; γ_{ei} is

$$\gamma_{ei}=0.65(u_i-u_{i+1}) / d_i \quad (14)$$

An equivalent shear modulus; G_{ei} and an equivalent damping ratio; h_{ei} are calculated through the G-γ, h-γ relationships of soil properties.

g) Convergence judgement

Setting new values of soil properties (G_{ei}, h_{ei}), the evaluation is repeated from item b). The evaluation will be repeated until the natural period of the surface ground is converged.

h) Acceleration response spectrum at ground surface and engineering bedrock.

The acceleration response spectra at the ground surface and the engineering bedrock are evaluated as follows.

$$S_{as}(T, \zeta=0) \fallingdotseq F_{ao}(T)\, G_s(T, \zeta_1, \alpha)/(T/2\pi) \quad (15)$$
$$S_{ab}(T, \zeta=0) \fallingdotseq F_{ao}(T)\, G_b(T, \zeta_1, \alpha)/(T/2\pi) \quad (16)$$

i) Modification of acceleration response spectrum at ground surface

To estimate the acceleration response spectrum conservatively at the ground surface, the spectrum is modified as follows.

1) Connect the two peak points, by a straight line, of the acceleration response spectrum corresponding to the first and second modes of the surface ground, in order to avoid an excessive dip between those peaks.

2) Equalize the acceleration of the spectrum at the ground surface with that at the outcropped engineering bedrock in the very short period range.

(2) Examples of evaluation

a) Soil properties of surface soil layers

Figure 6 shows shear wave velocities (V_s) in several soil deposits, which are evaluated. These velocities are measured by the PS logging method. The soil layer with more than 400m/s of Vs is selected as the engineering bedrock. The model proposed by Ohsaki et al.[11] is used as the nonlinear characteristics of the surface ground. The mass density of the soil is around 1.6 to 2.0.

b) Acceleration response spectrum at ground surface

The amplification factors (transfer function) of the ground surface to the outcropped engineering bedrock are shown in Fig. 7. The figures include the results from three analytical methods in the following.

1) Transfer functions in case of multi-layers with V_s by the linear analysis (indicated with "LINEAR").

2) Ratios of acceleration response spectrum at the ground surface to that at the outcropped engineering bedrock, which are calculated through the program Shake (indicated with "SHAKE").

3) Transfer functions obtained through the proposed method (indicated with "METHOD")

The predominant periods of the surface ground subjected to severe earthquake motions are longer by 1.3 to 2.0 times than those to moderate earthquake motions because of nonlinear behavior of soils. The proposed method gives the shorter periods than those by Shake in Sites 3 and 4. The amplification factors in Sites 3 and 4 by the proposed one are a little less than those by Shake.

Figure 8 shows the acceleration response spectra at the surface, which are obtained by the proposed method and the Shake program. A seismic base shear force of buildings for medium soil deposits in the Building Standard Law, which is modified to the value for one degree of freedom is also included. The response spectra by the proposed method have a good agreement with those by the Shake program.

5.4 Hysteretic Characteristics and Equivalent Damping Ratio of Structure

A multi-story building structure is reduced to an ESDOF system as shown in Fig. 9. The reduction to ESDOF system is based on the result of a push-over static analysis by applying horizontal forces at each floor level. The force-displacement relationship of SDOF system is assumed that its force corresponds to the base shear (Q_B), and its displacement (${}_1\Delta$) corresponds to the displacement at the height (h_e) where the natural modal participation function is equal to 1.0 ($\beta_1\{u\}_1=1.0$).

The equivalent damping ratio is defined by the viscous damping ratio, hysteretic dissipation energy, elastic strain energy of structure, and radiation effects of ground. Here, the effects of soil-structure interaction are considered. As for the structure, the equivalent damping factor of SDOF system, h_{eq}, is given by the following equation (see Fig. 10).

$$h_{eq} = \frac{1}{4\pi}\left(\frac{\Delta W}{W}\right) \tag{17}$$

where,

ΔW: dissipation energy of SDOF system, and

W: potential energy in SDOF system (= $Q_{B}\cdot{}_1\Delta/2$)

Here, the dissipation energy of stationary hysteretic loop at the assumed maximum response of the structure can be calculated by the cyclic loop of the structure using the application of push-over static analysis, or based on the total of damping ratio of all members and joints to be considered.

5.5 Prediction of maximum response

The response spectrum method using equivalent linearization technique is applied as typical procedure for predicting the maximum earthquake response. In this method, as shown in Fig. 2(c), the intersection of the force-displacement curve of SDOF system and the required seismic performance spectrum is the maximum response point. In the Sa-Sd spectrum, the acceleration response is divided by the gravity acceleration, g. In general, the maximum response calculated by the equivalent damping h_{eq} obtained from Equation (17) is too small, because Equation (17) is theoretically valid in case of stationary vibration. In case of nonstationary vibration caused by earthquakes, h_{eq} based on the assumed response has to be reduced appropriately for predicting the maximum response. Through the examination under recorded and synthesized earthquake motions, and some hysteresis curves that consist of bi-linear or tri-linear skeleton curves such as normal bi-linear and Takeda models, the factor for reducing h_{eq} is examined. Figure 12 illustrates an example of the results on bi-linear skeleton curve shown in Fig. 11. Hysteresis curves are normal bi-linear, degrading bi-linear and slip bi-linear. The input earthquake motions are four that are a synthesized earthquake motion examined by the Building Center of Japan and three recorded earthquake motions; 1995 Kobe NS, 1968 Hachinohe EW and 1940 El Centro NS. In this analysis, there are some analytical parameters that are the natural period, yield stiffness and yield strength. The damping ratio of 2% except for the hysteretic energy dissipation is included. The equivalent damping ratio estimated is reduced to 70 percent of h_{eq}. As a result of comparison of the responses between the time history analysis and the equivalent linearization method, they have a good agreement on the whole, though the deviation becomes large at the range of large ductility.

6. CONCLUDING REMARKS

In this paper, presented is the state of the art of the performance-based building code in Japan, currently under development at BRI. In essence, performance-based provisions intend to provide as clearly as possible answers to the frequently raised questions: for what purpose, towards what objective, and for what conditions. The greatest advantage of this new approach lies on the fact that it focuses primarily on the achievement of the prescribed objectives, regardless of the methodology used.

The evaluation procedure presented in this paper is in essence a blend of ESDOF modeling of building structures with the site-dependent response spectrum concept, which makes possible the prediction of maximum structural response in case of major earthquakes without using time history analyses.

The proposed evaluation procedure also make it realistic and simple to predict the maximum structural response in case of major earthquakes as well as to confirm whether the predicted response values are smaller than the limit ones.

In the proposed evaluation procedure, the limit value referred to is the maximum value. Nevertheless, besides maximum values any other measures can be used, for example energy. In this case, both the response and limit values should be expressed in terms of energy.

ACKNOWLEDGMENT

This manuscript was stimulated by the authors' participation as members of the "Task Committee to Draft Performance-based Building Provisions" established at BRI, where they have shared their opinions with other members. They

would like to express their gratitude towards all of them.

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Table 1 Requirements for Building Structures and Earthquake Motion Levels

Requirement	Earthquake
(a) Life Safety (to prevent failure of stories in structural frames)	Maximum Earthquake to be considered (earthq. records, seismic and geologic tectonic structures, active faults, etc.)
(b) Damage Limitation (to prevent damage to structural frames, members, interior and exterior finishing materials in order to avoid the conditions not satisfying the requirement (a) and others)	Once-in-a-lifetime Event (return period: 30-50 years)

Note: The deterioration of materials during the lifetime of a structure should be considered.

Table 2 Representative Illustration of Proposed Evaluation Procedures

Requirement		Earthquake
a) Life Safety	Level	Maximum Earthquake
	Response Value	Maximum Internal Force/Displacement
	Limit Value	Limit Strength/Displacement*1
b) Damage Limitation	Level	Once-in-a-lifetime Event (Return period 30 - 50 years)
	Response Value	Internal Force/Displacement taking place at each structural element
	Limit Value	Limit Strength/Displacement*2

*1 - Repeating cycles effect at plastic region of response to be taken into account.
*2 - The whole building structure behaves roughly within elastic range.

Notes :

1) The limit values corresponding to Maximum Event Level are determined based on the condition that equilibrium of forces and displacement compatibility in the structural system are guaranteed.

2) Displacement and acceleration related limit values, determined on the basis of the requirements for architectural, mechanical and electrical elements permanently attached to building structures, are thought to be considered in certain cases.

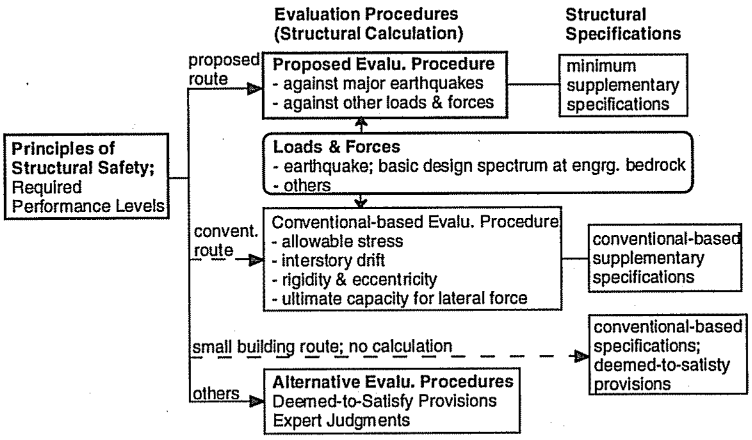
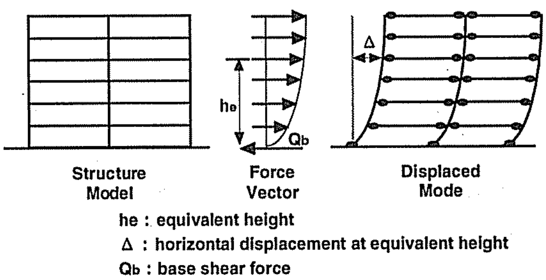
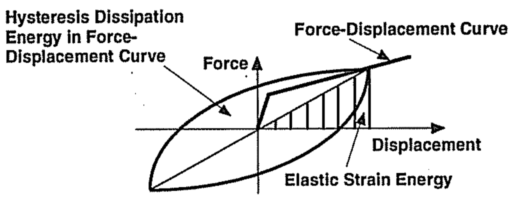


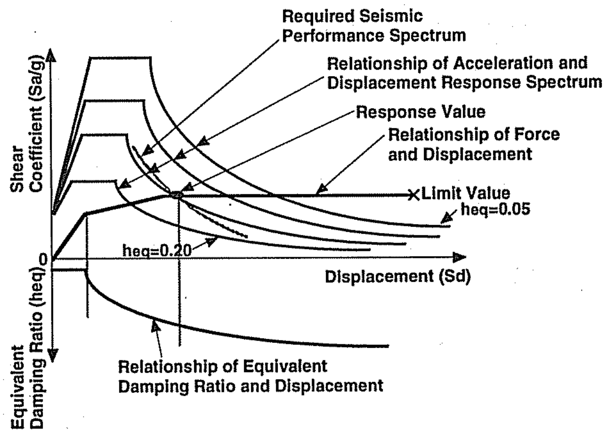
Fig. 1 Conceptual Framework of Proposed Performance-Based Structural Provisions



(a) Structure Model and Inelastic Response



(b) Energy for Equivalent Damping Ratio



(c) Comparison of Expected Response Values and Estimated Limit Values

Fig. 2 Illustration of Proposed Evaluation Procedure for Major Seismic Events

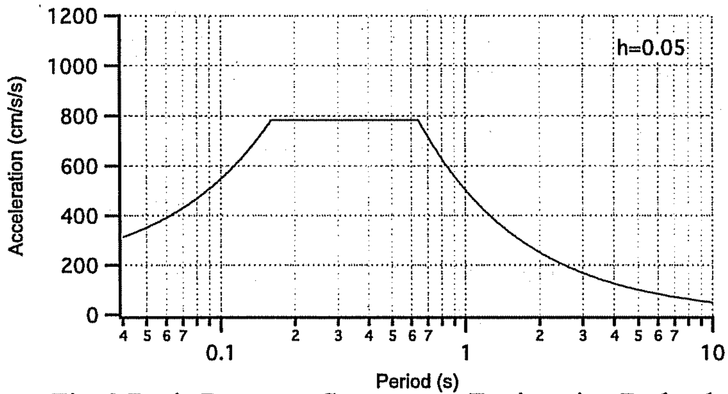


Fig. 3 Basic Response Spectrum at Engineering Bedrock

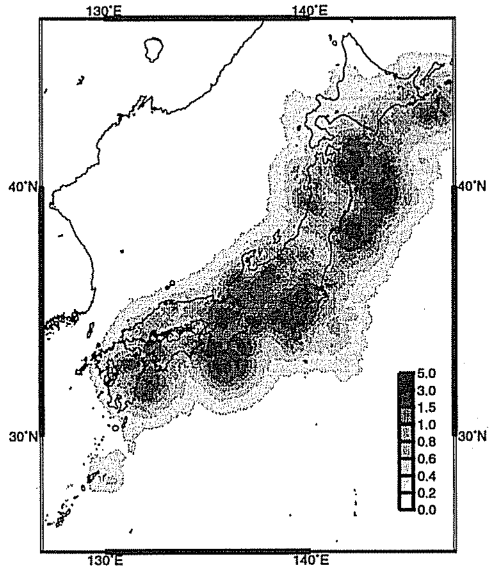


Fig. 4(a) Distribution of PGA at Engineering Bedrock expected in 50 years (normalized with 0.064g)

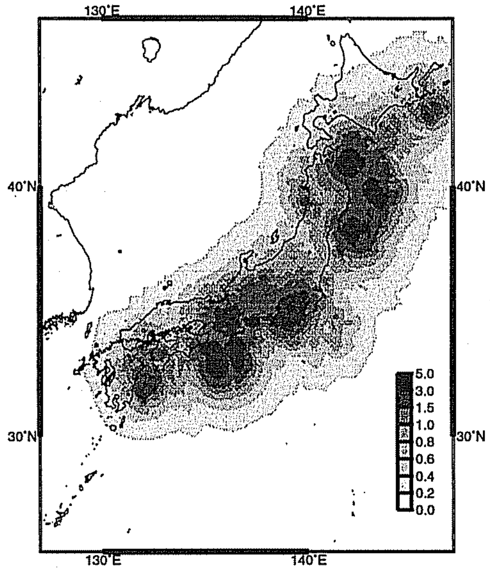


Fig. 4(b) Distribution of PGV at Engineering Bedrock expected in 50 years (normalized with 8 cm/s)

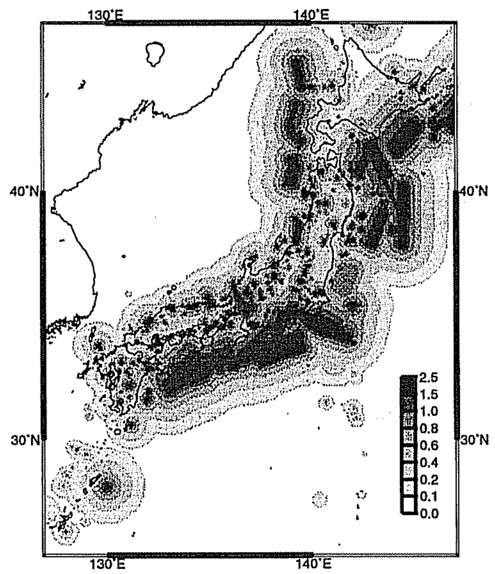


Fig. 4(c) Distribution of PGA at Engineering Bedrock expected in 500 years (normalized with 0.32g)

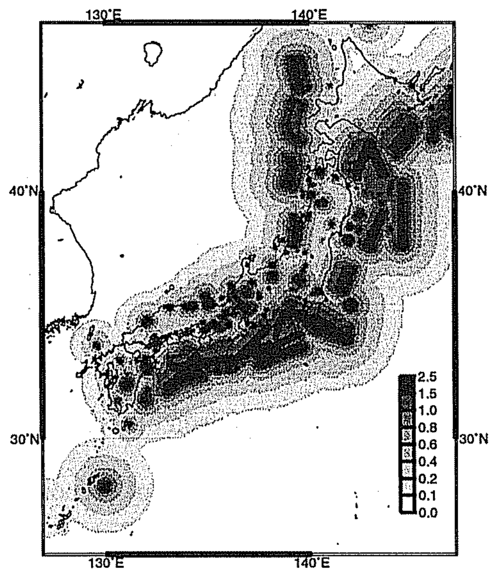


Fig. 4(d) Distribution of PGV at Engineering Bedrock expected in 500 years (normalized with 40 cm/s)

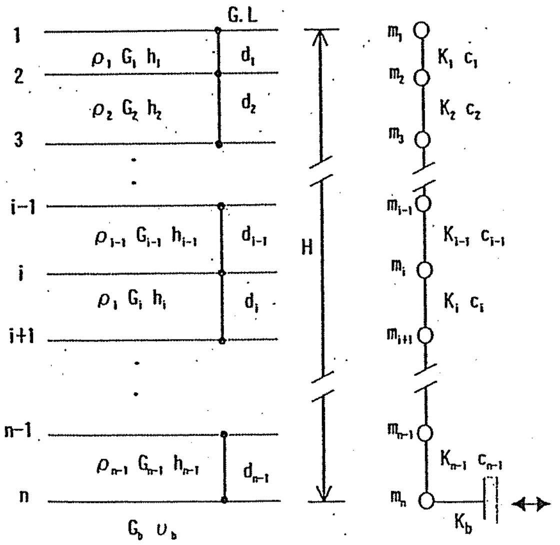


Fig. 5 Analytical Model of Surface Soil Layers

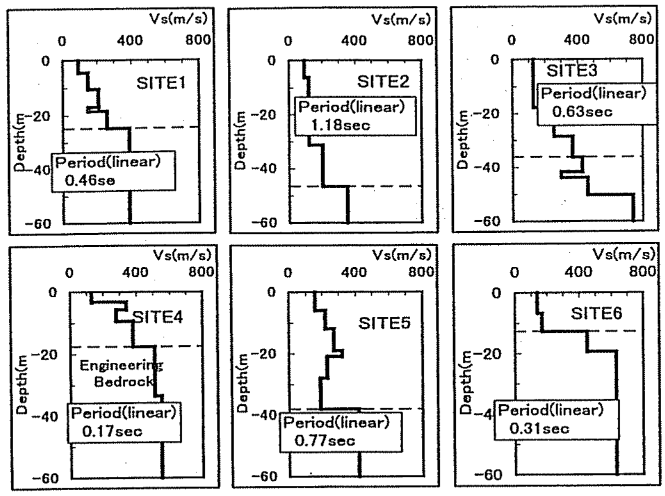


Fig. 6 Shear Wave Velocity Distribution at Several Sites

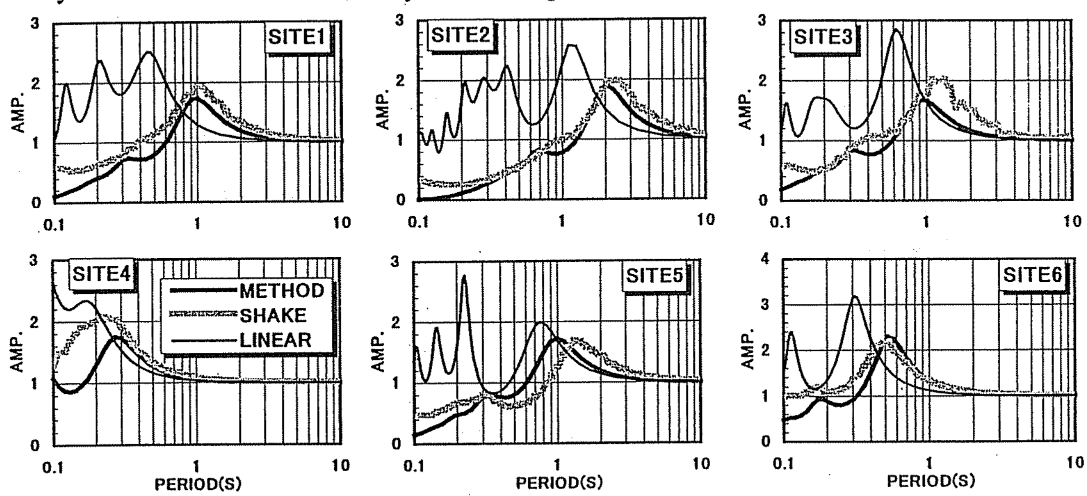


Fig. 7 Transfer Functions of Ground Surface to Engineering Bedrock

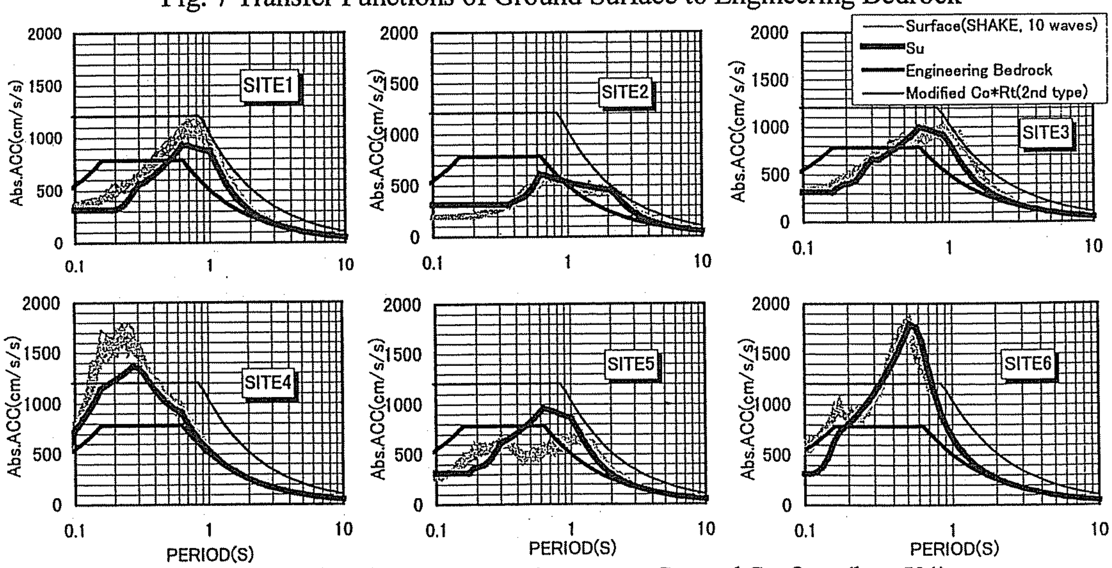


Fig. 8 Acceleration Response Spectra at Ground Surface (h = 5%)

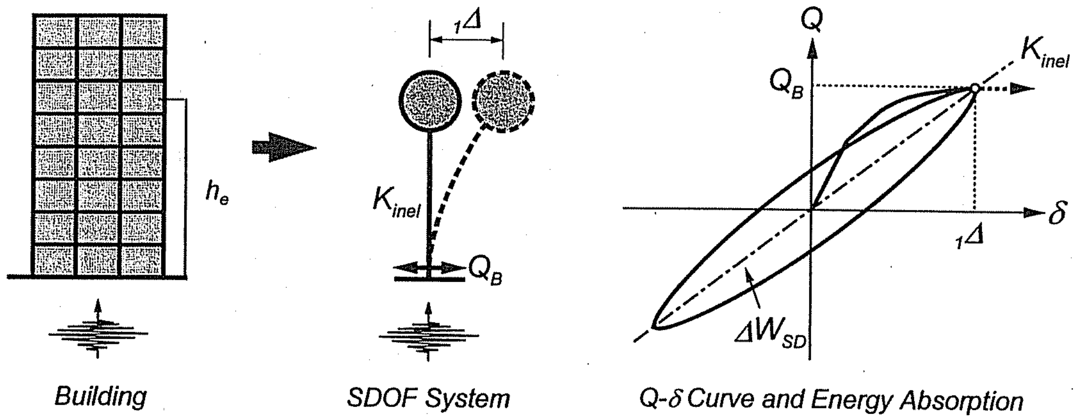


Fig. 9 Reduction to Single-Degree-of-Freedom System

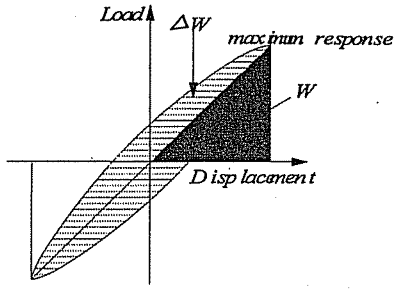


Fig. 10 Hysteretic Dissipation Energy and Potential Energy

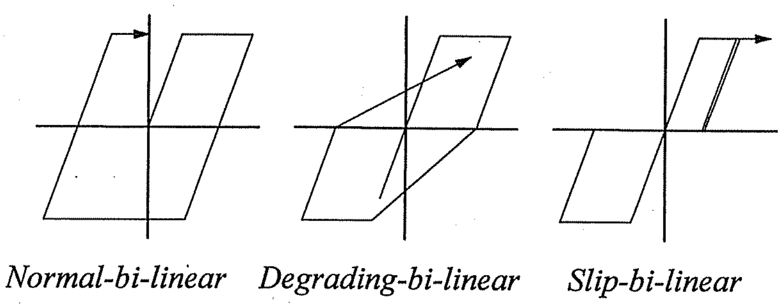


Fig. 11 Analytical Hysteresis Models

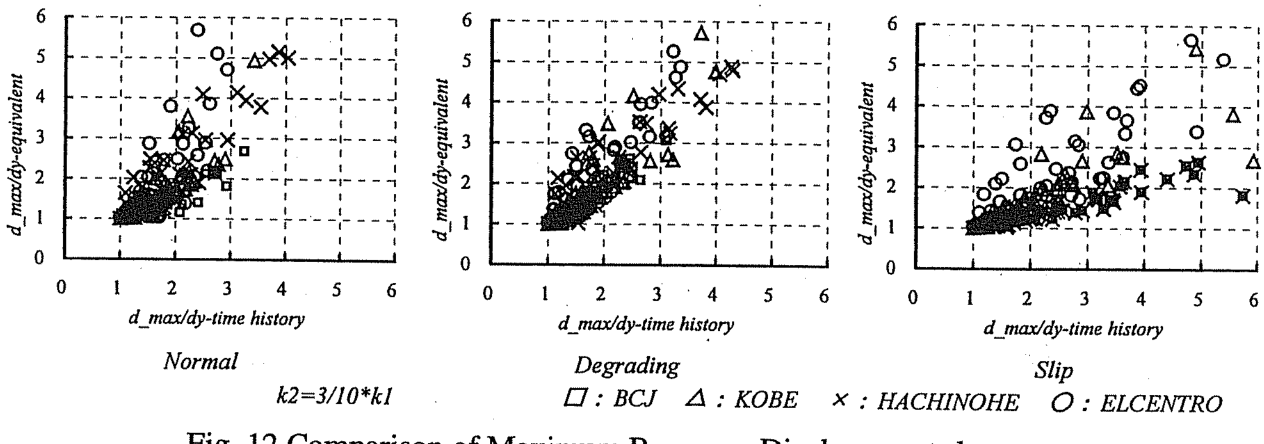


Fig. 12 Comparison of Maximum Response Displacements between Time History Analysis and Equivalent Linearization Method