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PERFORMANCE-BASED BUILDING CODE OF JAPAN -FRAMEWORK OF SEISMIC AND STRUCTURAL PROVISIONS

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SUMMARY

The Japanese Building Code related to the structural engineering will be drastically revised from the prescriptive type of the provisions to the performance-based one. There are two kinds of performance requirements for the seismic provisions in the new code : human life safety and damage control of a building corresponding two earthquake motion levels. The return periods of this two earthquake motion levels are approximately 500 years and approximately 50 years, respectively. The earthquake motion is defined as the design acceleration response spectrum in the code which is specified at the engineering bedrock in order to take the soil condition and soil-structure interaction effect into consideration as properly as possible. Here, the engineering bedrock is defined based on shear wave velocity of the soil at a site. The ordinate of the design response spectrum is determined by considering historical seismicity data, seismo-tectonic zones and active faults of the region.

The response values of structures are estimated based on this design response spectrum. They should be less than the limit values such as deformation capacity of structural members.

INTRODUCTION

Japanese Building Standard Law was revised in 1998. The highlight related to the structural engineering is the drastic revision from the prescriptive type of the provisions to the performance-based one. The detailed specification is now under development and new code related to structural engineering will be enforced by June 2000. In the proposal developed under the revised Law, the precise definitions for performance requirements and verification method based on accurate response and limit values are specified so that the code should be applicable to any kind of materials and any type of structures such as seismic isolation systems as long as the material property is clear and the structural behavior of a building is appropriately estimated.

This paper presents the framework and concepts of the proposal developed by Building Research Institute for the new performance-based structural provisions, focusing on earthquake engineering.

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CONCEPTUAL FRAMEWORK OF PERFORMANCE-BASED STRUCTURAL CODE

The conceptual framework of performance-based structural code proposed by BRI is shown in Figure 1. Following the principles of structural requirement, the evaluation procedures to be used for the estimation of structure's conformity with the required performance level are roughly classified as: Proposed route, Conventional route, Small building route and Others.

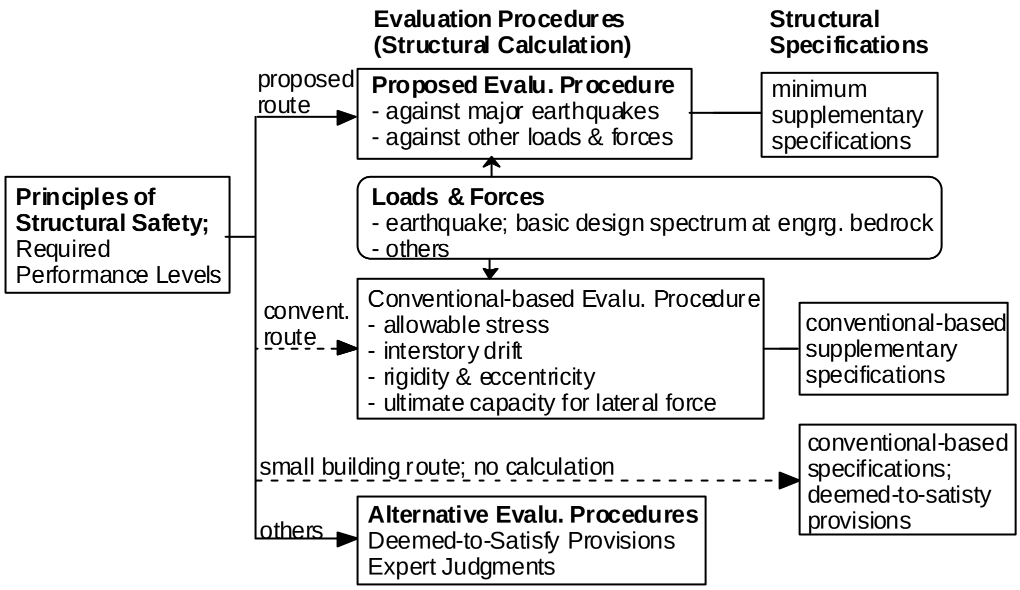


Figure 1: Conceptual framework of proposed performance-based structural provisions

The proposed route represents a new evaluation procedure to be used as well as the current one. The current one is based on the calculation of allowable stress and estimation of ultimate capacity for lateral load. It considers the 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 is applied 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, 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 is 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.

REQUIRED SEISMIC PERFORMANCE LEVEL'S 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, the requirements for building structures are classified in two categories, which are described below.

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.

Life Safety

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

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 for life safety should be preserved even after the earthquake considered. 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.

Maximum Earthquake Motion Level

This level of earthquake motions corresponds to the category of requirements for life safety 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.

Once-in-a-Lifetime Event Level

This level of earthquake motions corresponds to the category of requirements for damage limitation 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.

Design Earthquake Motion

The design seismic force currently in use specifies the story shear force without apparent prescription of the ground motion. Therefore, this method is so easy to pursue the design procedure. However, a contradiction exists in this method that the derived earthquake ground motions are not equal even within the class, since the design force is derived from the response values of the building itself and the design force is prescribed uniformly with a class of buildings. Considering these situations, it is concluded that the seismic design should start with the defining the input earthquake ground motion. This also coincides with the idea of performance-based structural design aiming at more flexible design.

Design Response Spectrum at Engineering Bedrock

The ground motion is represented with its acceleration response spectrum in the new provisions. The basic ground motion is firstly defined at the engineering bedrock corresponding to the seismicity of the area. The engineering bedrock is defined herein as follows. The engineering bedrock is underlain in the underground within the area. The geotechnical data is mostly obtained in the investigations conducted within the area. And also the considerable number of strong motion data is obtained and the characteristics are evaluated with the recordings at the depths. This upper face of the soil layer is defined as engineering bedrock. This definition of engineering bedrock gives the idea of layers with shear velocity larger than approx. 400m/s. The amplification characteristics of the surface soil layers are to be evaluated with the geological data of the site.

The design earthquake ground motion is represented in the following equation.

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

Where,

$S_A(T)$ = design earthquake ground motion,
Z = regional seismicity coefficient,
$G_s(T)$ =amplification of surface soil,
$S_0(T)$ = basic spectrum at exposed engineering bedrock, and
T = period in second.

The basic spectrum is set up to be very basic. It consists of two parts, i.e., a uniform acceleration portion in shorter periods, and a uniform velocity portion in longer periods. The two intensities of uniform levels are determined with expected peak values and response factors for acceleration and velocity. The intensity level of the design motion is based on the design force for the intermediate soil class specified in the current Building Standard Law of Japan. In addition, the relationship between the story shear force and the input motion is also taken into consideration.

The resultant design ground motion for capacity design is defined as 0.8G for the 5% damping acceleration response spectrum and as 80cm/s for the 5% damping velocity response spectrum at the exposed engineering

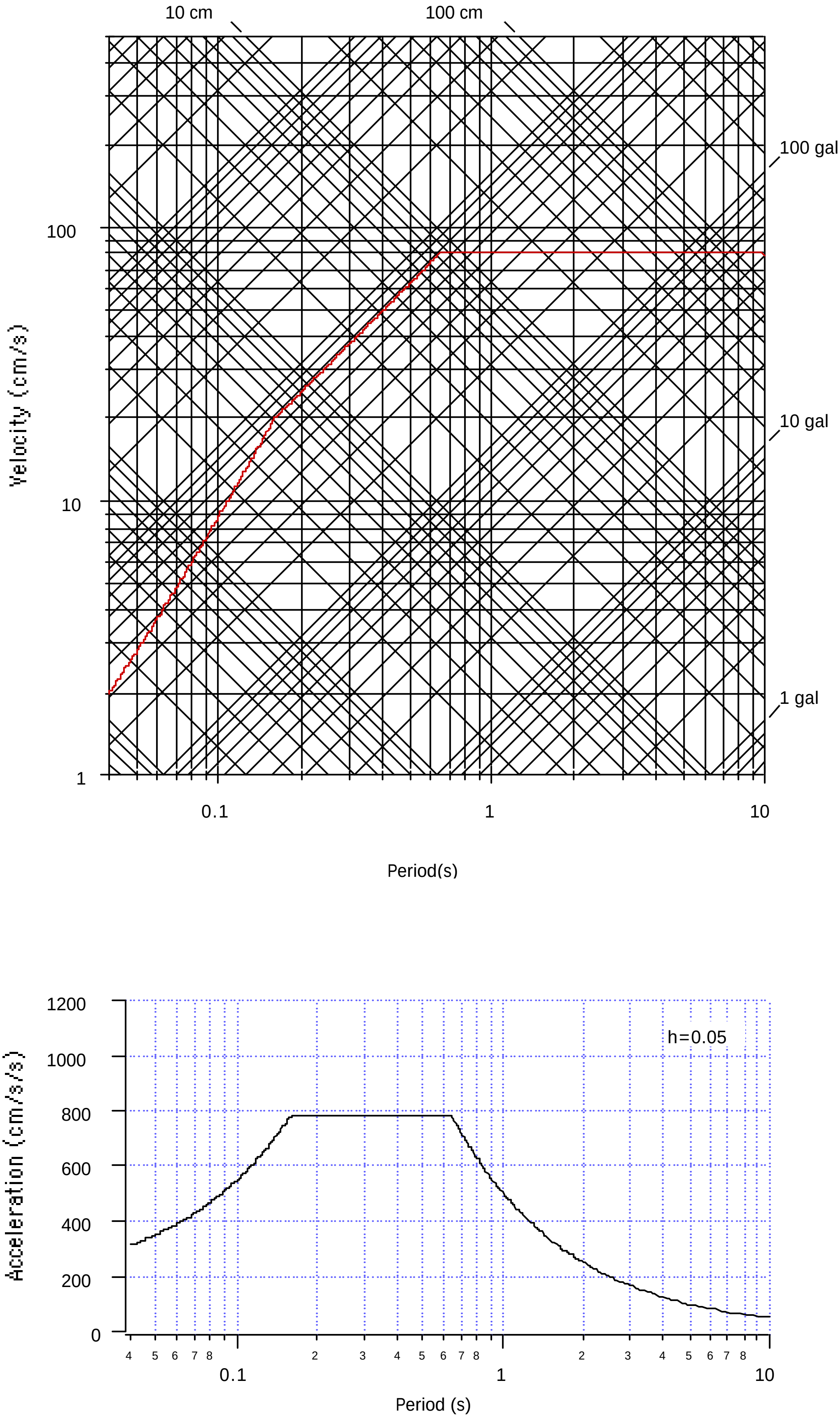


Figure 2: Design response spectrum of major earthquake motion at the exposed engineering bedrock

bedrock. For the periods shorter than 1/4 of the intersection of the uniform acceleration and the velocity, the spectral values are reduced so that the acceleration amplitude at zero period equals to the peak acceleration of the input motion. The peak acceleration is set to 1/2.5 of the uniform acceleration level. The design spectrum thus defined is shown in Figure 2.

The Regional Seismicity Coefficient

The research on the seismic hazard map on which the coefficient is based has been continued from the pioneering work by Dr. Kawasumi[1). The advances in the prediction methodology of strong ground motion with the collection of strong motion records for near source region in this decades have become a thrust to upgrade the site-specific design earthquake motion to date. The regional seismicity coefficient is evaluated as relative factors representing the difference in the expected ground motion parameters such as peak ground acceleration or peak ground velocity with regions for both of large and intermediate earthquake motions. In addition, since the frequency content presumably differs in the expected values, the coefficients are to be defined with separately for acceleration and velocity, respectively. Therefore, as results, the four types of coefficients are proposed reflecting the intensity levels and the frequency contents.

To prepare the regional maps, the following two ground motion levels are defined.

1) Large earthquake motion which is represented by the largest annual maximum in 500 years.
2) Intermediate earthquake motion which is represented with the 10^{th} largest annual maximum in 500 years.

The data used in this study are as follows.
1) Earthquakes occurring during 1496-1984 selected from the Usami's Engineering Catalogue.
2) Earthquake data issued from the Japan Meteorological Agency (JMA) during 1985-1995.
3) Fault parameters which have been proposed for major past earthquakes.

The following two empirical formulae (attenuation formulae) were used to estimate the peak amplitudes for the earthquakes. These formulae are applicable to the near source area such as Kobe during the 1995 Hyogo-ken Nambu Earthquake.

1) Peak ground acceleration (A) for the diluvial soil class

$$\log A = 0.51M - \log[R + 0.006 \cdot 10^{0.51M}] - 0.0033 + 0.59 \quad (2)$$

[2)]

Where,

A= peak ground acceleration (cm/s^2),
M= earthquake magnitude, and
R= the closest distance to the fault plane in kilometer.

2) Peak ground velocity (V) for the engineering bedrock level

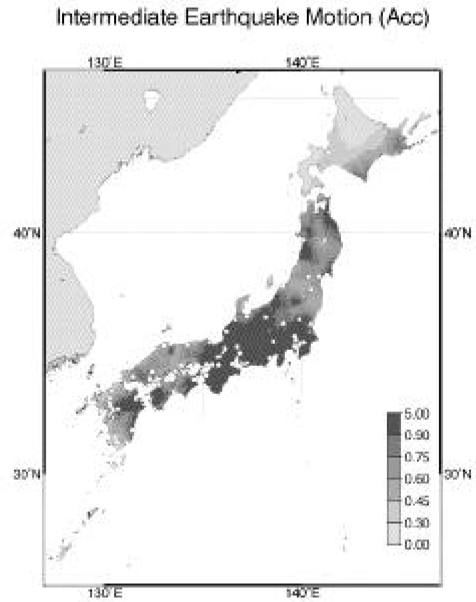
$$\log V = -0.22M^2 + 3.94M - \log[R + 0.01 \cdot 10^{0.43M}] - 0.002R - 11.9 - 0.71 \cdot \log Vs \quad (3)$$

[3)]

Where,

V= peak ground velocity (cm/s),
M_w = moment magnitude,
R = the distance to the fault rupture zone (km), and
Vs = average shear wave velocity for upper 30m surface soil deposit (m/s).

For peak ground velocity, the JMA magnitude is used in place of moment magnitude, since both are approximately equal in values for the scope of our study. The shear wave velocity 600 m/s was used



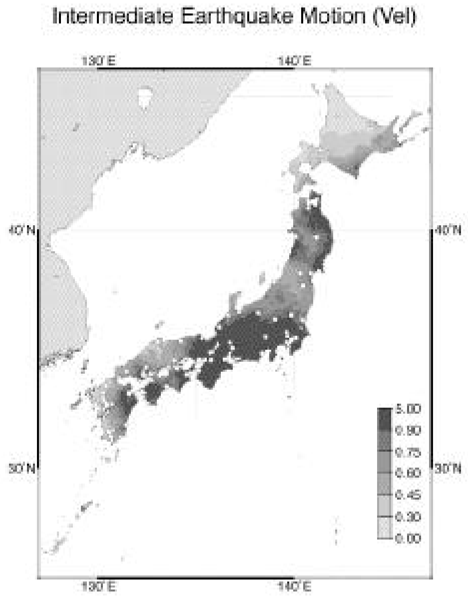
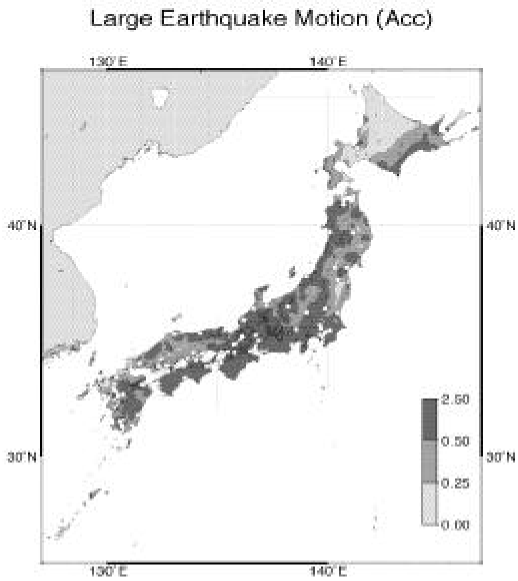


Figure 3(a): Distribution of PGA at engineering bedrock expected in 50 years (normalized with 0.064G)

Figure 3(b): Distribution of PGV at engineering bedrock expected in 50 years (normalized with 8cm/s)



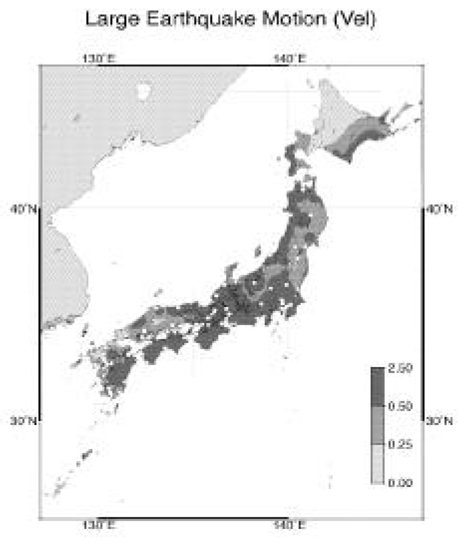


Figure 3(c): Distribution of PGA at engineering bedrock expected in 500 years (normalized with 0.32 G)

Figure 3(d) : Distribution of PGV at engineering bedrock expected in 500 years (normalized with 40 cm/s)

for computation, although this does not affect the result as far as the relative regional difference is concerned.

Using these formulae, the expected peak values for each location were calculated. Regional maps were drawn based on the computed vales normalized by 0.32G and 40cm/s for large earthquake motion, and 0.064G and 8cm/s for intermediate earthquake motion, respectively. The maps are shown in Figure3a-3d. It is seen that larger expected values are found corresponding the epicentral locations of bigger earthquakes. For large earthquake, most locations show the ratio below 2.5 in comparison of the previously mentioned engineering bedrock intensity levels for acceleration, and below 2.5 for velocity levels. The expected intensity levels estimated with the data in 500 years are comparable with or slightly larger than the level of seismic force currently in use for the second class of soil condition.

The maps shown herein are examples of evaluation using the sets of proposed empirical formulae and generally available data. Therefore, a more extensive study including applying other methodologies or referring other proposals will be necessary to establish the regional seismicity coefficient maps to be specified in the building code or provisions.

THE PRINCIPAL FOR THE VARIFICATION

The response values are estimated based on the design response spectrum and the force-displacement relation of structures, which is essentially obtained by push-over non-liner analysis. The limit values of structures such as deformation capacity are calculated from the material property such as stress versus strain relations

CONCLUDING REMARKS

There are two kinds of performance requirements: human life safety and damage control of a building for the seismic provisions. There are corresponding two earthquake motion levels. The earthquake motions are defined as the design acceleration response spectrum which is specified at the engineering bedrock in order to take the soil condition and soil-structure interaction effect into consideration as properly as possible. The ordinate of the design response spectrum is determined by considering historical seismicity data, sieismo-tectonic zones and active faults of the region. Here, the engineering bedrock is defined based on shear wave velocity of the soil at a site. The return periods of the earthquake motion of approximately 500 years and approximately 50 years are used to evaluate life-safety level and damage control level, respectively. The required performance shall be verified by comparing the response values with the limit values.

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REFERENCES

1) Kawasumi, H. (1951), "*Measures of Earthquake Danger and Expectancy of Maximum Intensity Throughout Japan as Inffered from the Seismic Activity in Historical Times*", Bulletin of Earthquake Research Institute, 29, pp.469-481.

2) Fukushima, Y. and Tanaka, T. (1992), "*Revised Attenuation Relation of Peak Horizontal Acceleration By Using A New Data Base*", Progr. Abstr. Seismol. Soc. Jpn., 2, 116 (in Japanese).

3) Midorikawa, S. (1993), "*Preliminary Analysis for Attenuation of Peak Ground Velocity on Stiff Site*", Proceedings of the International Workshop on Strong Motion Data, 2, pp.39-48.