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Seismic Vulnerability Assessment Methods for Buildings in Japan

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ABSTRACT

The development of seismic vulnerability evaluation standards for reinforced concrete buildings in Japan is briefly reviewed. Damage statistics are shown to indicate that severe damage was observed in a relatively small percentage of existing buildings even after damaging earthquakes in the world. Therefore, a simple screening procedure is necessary to identify such vulnerable buildings out of the existing building stock. After discussing the principles of seismic vulnerability assessment using a simple single-degree-of-freedom system, applications to multi-degree-of-freedom systems and to structures of irregular configuration are discussed. A general procedure consistent with the present design provisions in Japan is introduced.

INTRODUCTION

Most building codes in the world explicitly or implicitly accept structural damage to occur in a building during strong earthquakes as long as the hazard to life is prevented. Indeed, many earthquakes caused such damage in the past. Seismic design codes were improved after each earthquake disasters, but old constructions were left unprotected by new technology.

The 1968 Tokachi-oki earthquake caused significant damage, for the first time in Japan, to reinforced concrete buildings; i.e., reinforced concrete columns failed in shear in school buildings. The concern was expressed by many organizations about the earthquake safety of existing reinforced concrete buildings; e.g., the Ministry of Education about school buildings, the Ministry of Construction about government buildings, and construction companies about their clients' buildings. Various methods were developed for the seismic vulnerability assessment of existing buildings against future earthquakes.

The Ministry of Construction organized a committee in 1976 to develop an integrated method to evaluate the seismic vulnerability of existing low- to mid-rise reinforced concrete buildings.

The committee published “Standard for Seismic Vulnerability Assessment of Existing Reinforced Concrete Buildings” [1,2] in 1977.

After the 1995 Hyogo-ken Nanbu earthquake, Japanese Diet (Congress), recognizing the urgent importance of improving seismic resistance of existing buildings, proclaimed a law to promote the seismic strengthening of existing buildings in October 1995. The law, enforced on December 25, 1995, requires that the owner of a building for use by a number of un-identified people must make efforts to perform the seismic vulnerability assessment (examination of safety under a severe earthquake motion) of the structure and that the owner must make efforts to strengthen the structure if needed. A seismic vulnerability assessment procedure was outlined in the Ministry of Construction Notification No. 2089 issued on December 25, 1995. The procedure examines if a structure possesses the seismic resistance of a level specified in the Building Standard Law.

This paper introduces the seismic vulnerability assessment method outlined in the Ministry of Construction Notification No. 2089.

DAMAGE STATISTICS FROM MAJOR EARTHQUAKES

The Architectural Institute of Japan (AIJ) investigated the damage after major earthquakes in Japan as well as in the world. The damage statistics were collected in Mexico City and Lazaro Cardenas after the 1985 Mexico earthquake [3], Baguio City after the 1990 Luzon, Philippines, earthquake [4], Erzincan City after the 1992 Erzincan, Turkey, earthquake [5], and Kobe after the 1995 Hyogo-ken Nanbu earthquake [6]. A heavily damaged area was first identified, and the damage level of all buildings in the area was assessed by structural engineers and researchers. The damage level is classified, in this paper, to (a) operational damage, (b) heavy damage, and (c) collapse. There was a significant code change in 1981 in Japan; therefore the damage statistics are shown for buildings before and after the code change.

The damage statistics show that 75 to 95 percent of buildings in severely damaged areas remained operational after the strong earthquakes in Mexico City, Baguio City, Erzincan City, and Kobe City. It is important to identify the small number of those buildings possibly vulnerable to future earthquakes. A

Table 1 Damage statistics of buildings from major earthquakes

Earthquake, year	Operational damage	Heavy damage	Collapse	Total
Mexico City, 1985	4,251 (93.8%)	194 (4.3%)	87 (1.9%)	4,532
Lazaro Cardenas, Mexico, 1985	137 (83.5%)	25 (15.2%)	2 (1.2%)	164
Baguio City, Philippines, 1990	138 (76.2%)	34 (18.8%)	9 (5.0%)	181
Erzincan City, Turkey, 1992	328 (77.4%)	68 (16.8%)	28 (6.6%)	424
Kobe (pre-1981 construction), 1995	1,186 (79.4%)	149 (10.0%)	158 (10.6%)	1,493
Kobe (post-1982 construction), 1995	1,733 (94.0%)	73 (4.0%)	38 (2.1%)	1,844

simple procedure is desirable to "screen out" the majority of safe buildings. A more detailed and sophisticated procedure may be utilized only when some problems are detected in the building. The damage rate was small in Mexico City because the majority of buildings were low-rise less than four stories high. The standard [1] introduced an example of such screening procedures. The procedure introduced in this paper is not suitable for this purpose.

A definite trend is observed in the damage statistics that (a) the percentage of heavy damage increased with the number of stories, and (b) the damage rate decreased with the development of new technology.

PRINCIPLES OF SEISMIC RESISTANCE ASSESSMENT

The lateral load strength is not a single index to represent the safety of a building. Strength and deformation capability of constituent members, material properties on site, structural configuration, foundation, site conditions, soil-structure interaction, quality of workmanship, importance of buildings, structure's age, the installation of building facilities, the safety of non-structural elements and hazard history need to be taken into account in seismic vulnerability assessment. The structural deterioration in earthquake resistance caused by (a) existing cracks, (b) observed deflection under gravity conditions, (c) uneven settlement caused by foundation deformation, (d) neutralization of concrete, and (e) rust on reinforcement, should be carefully examined through the investigation at the building site. This paper assumes that the strength and deformation capacities of structural members have been estimated on the basis of actual dimensions and material properties investigated on site.

The Newmark's design criteria [7] determine a minimum base shear coefficient C_y required for an elastic-plastic single-degree-of-freedom (SDF) system having a ductility μ (ultimate deformation divided by the yield deformation) to resist a ground motion which intensity produces an elastic response base shear coefficient C_e.

$$C_y = \frac{C_e}{\sqrt{2\mu - 1}} \quad \text{for short period systems} \tag{1a}$$

$$C_y = \frac{C_e}{\mu} \quad \text{for long period systems} \tag{2a}$$

The relation can be rewritten in the following forms to represent the intensity of ground motion, in terms of elastic response base shear coefficient C_e, for an elasto-plastic SDF system having the lateral load resistance C_y and a ductility capacity μ to survive.

$$C_e = C_y \sqrt{2\mu - 1} \quad \text{for short period systems} \tag{1b}$$

$$C_e = C_y \cdot \mu \quad \text{for long period systems} \tag{2b}$$

The maximum elastic response base shear coefficient (maximum acceleration response expressed as a fraction of the gravity acceleration) may be used as an index to represent the intensity of ground motion.

Thus, for an SDF system, the structural index E_0 of earthquake resistance is expressed as

$$E_0 = C \cdot F \tag{3}$$

in which C: strength index (lateral strength expressed in terms of base shear coefficient; i.e., lateral force capacity divided by the total weight), and F: ductility index (index of deformation capacity).

The intensity of earthquake response by a target seismic event varies with the seismicity of region and surface geology of a construction site. Design acceleration spectrum is expressed as the product of seismic zone factor Z and vibration characteristic factor $R_t(T)$ in the Building Standard Law. An index I_s (structural seismic capacity index) may be introduced to represent the level of seismic safety margin of a structure with respect to the code specified design earthquake forces. The elastic base shear coefficient C_e corresponding to a building seismic performance may be represented as

$$C_e = I_s \cdot Z \cdot R_t(T) = E_0 \tag{4}$$

Thus, the structural seismic capacity index I_s is expressed as

$$I_s = \frac{C_e}{Z\ R_t(T)} = \frac{E_0}{Z\ R_t(T)} \tag{5}$$

The vibration characteristic factor $R_t(T)$ (Fig. 1) represents the shape of design earthquake spectrum for three types of soil;

$$\begin{aligned} R_t(T) &= 1.0 && \text{for} \quad T < T_c \\ R_t(T) &= 1.0 - 2.0\left\{\frac{T}{T_c} - 1\right\}^2 && \text{for} \quad T_c \le T < 2T_c \\ R_t(T) &= 1.6\frac{T_c}{T} && \text{for} \quad 2T_c \le T \end{aligned} \tag{6}$$

where, T_c: dominant period of subsoil (= 0.4sec for stiff sand or gravel soil, = 0.6sec for other soil, and = 0.8sec for alluvium mainly consisting of organic or other soft soil); T: natural period of a building. The building period T (sec) may be estimated by

$$T = 0.02\ H \tag{7}$$

where, H: total height of a reinforced concrete building in m. The seismic zone factor Z varies from 0.7 to 1.0 in Japan.

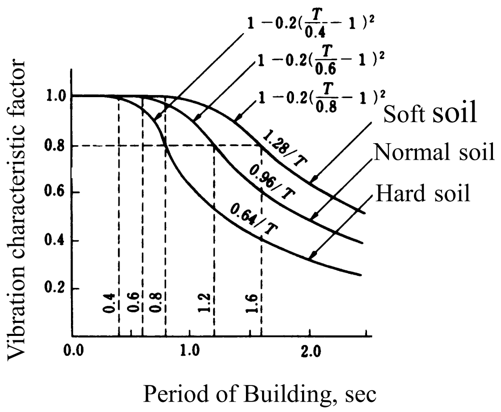


Fig. 1 Vibration characteristic factor $R_t(T)$

STRUCTURES CONSISTING OF DIFFERENT STRUCTURAL MEMBERS

Equation (3) holds for an SDF structure consisting of structural members of identical properties. In a real structure, some members fail earlier than the others. For simplicity, let us consider a system consisting of two types of structural members, exhibiting the lateral load deformation relationships shown in Fig. 2. The failure of stiff and less ductile members may significantly reduce the resistance of the structure, but the ductile members may be able to resist the remaining ground motion. The effect of the delay in reaching the maximum resistance should be accounted in the earthquake resistance assessment. Hence, structural index E_0

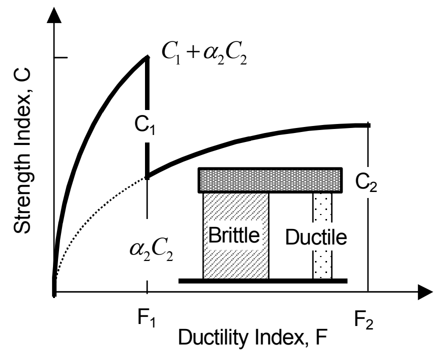


Fig. 2 Force-deformation relation of two-member system

is evaluated at the largest lateral resistance (Eq. (8)) and at the failure of the ductile members (Eq. (9)).

$$E_0 = (C_1 + \alpha_2 C_2) \cdot F_1 \tag{8}$$

$$E_0 = \sqrt{(C_1 \cdot F_1)^2 + (C_2 \cdot F_2)^2} \tag{9}$$

in which ($C_1 + \alpha_2 C_2$): total strength index at failure of the less ductile system, F_i, C_i (i = 1, 2): ductility and strength indices, respectively, of the less ductile and the ductile members. Equation (9) was suggested on the basis of a series of nonlinear earthquake response analyses of two member systems. The larger value of the two equations can be taken as structural index E_0. The same concept may be used for a structure composed of more than two representative member groups.

CRITICAL LOAD CARRYING MEMBERS

Note that the failure of brittle members, accompanied by the loss of the gravity load carrying capacity, may lead to the collapse of a structure. Such essential vertical members are called "*critical load carrying members.*" It becomes necessary to examine if the gravity load could be transferred to adjacent columns upon failure of the less ductile members. If critical load carrying members exist in a structure, Eq. (8) should be used to calculate structural index E_0.

Structural walls are thought to carry vertical load even after failing in shear because the failure mode is often in shear-compression mode. If the shear failure of some columns is critical for earthquake resistance of a story, the transfer of their vertical loads to neighboring columns and walls through shear transfer by above structural walls, adjacent girders and slabs must be carefully examined.

EXTENSION TO MULTI-STORY STRUCTURES

For a multi-degree-of-freedom (MDF) structure, structural index E_0 must be evaluated in each story. Strength index C_i in story i is defined as the story shear resistance divided by the total weight that the story supports. Structural index E_{0i} of story i must be interpreted to that E_0 of an SDF system.

Suppose an MDF system oscillates in the fundamental mode, maximum inertia force $\{f\}_1$ may be expressed using the fundamental mode shape vector $\{\phi\}_1$, modal participation factor γ_1, acceleration spectral value S_a and mass matrix $[m]$;

$$\{f\}_1 = [m]\ \{\phi\}_1\ \gamma_1\ S_a \tag{10}$$

where the participation factor γ_1 is defined as

$$\gamma_1 = \frac{\{\phi\}_1^T\ [m]\ \{e\}}{\{\phi\}_1^T\ [m]\ \{\phi\}_1} \tag{11}$$

where, $\{e\}$: a vector consisting of all elements equal to 1.0.

Story shear V_i at story i is the sum of all lateral force above the story;

$$V_i = \sum_{j=i}^{n} f_{1j} = \sum_{j=i}^{n} m_j \, \phi_{1j} \, \gamma_1 S_a \tag{12}$$

where, f_{1j} : lateral force in the fundamental mode acting at level j, ϕ_{1j}: element of the fundamental mode vector at level j, m_j: floor mass at level j, and n: total number of stories.

Story shear coefficient C_i at level i is obtained by dividing story shear V_i by the total weight above the story;

$$C_i = \frac{V_i}{\sum_{j=i}^{n} m_j g} = \frac{\gamma_1 S_a}{g} \frac{\sum_{j=i}^{n} m_j \phi_{1j}}{\sum_{j=i}^{n} m_j} \tag{13}$$

where g: acceleration of gravity. Solving for the response acceleration spectral value S_a when story shear coefficient C_i is developed at level i, strength index C of an SDF system may be expressed as

$$C = \frac{S_a}{g} = \varphi_i \cdot C_i = \frac{1}{\gamma_1} \frac{\sum_{j=1}^{n} m_j}{\sum_{j=i}^{n} m_j \phi_{1j}} C_i \tag{14}$$

where φ_i: story index at story i. The story index relates strength index C of an equivalent SDF system to story shear coefficient C_i at story i;

$$\varphi_i = \frac{1}{\gamma_1} \frac{\sum_{j=i}^{n} m_j}{\sum_{j=i}^{n} m_j \phi_{1j}} \tag{15}$$

For a linear mode shape of a structure with uniform story height and mass distribution, story index φ_i is expressed as

$$\varphi_i = \frac{2}{3} \frac{2n+1}{n+i} \tag{16}$$

where, n: the number of stories, i: story number.

A more conservative expression for story index φ_i is suggested in the seismic vulnerability assessment standard [1] to consider crudely higher mode contribution at upper stories;

$$\varphi_i = \frac{n+1}{n+i} \tag{17}$$

Equation (17) represents the ratio of the base shear coefficient C_B to a story shear coefficient C_i for a linear mode shape with uniform story height and mass distribution.

The Building Standard Law suggests the use of factor A_i for the vertical distribution of seismic story shear coefficients normalized to the base shear coefficient;

$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i \right) \frac{2T}{1+3T} \tag{18}$$

where $\alpha_i = W_i / W_1$, and W_i: total dead and live loads story i supports, and W_1: total dead and live loads of the building, T: elastic period of a building. The reciprocal of factor A_i may be conservatively used for story index φ_i.

Structural index E_0 of the i-th story in terms of an equivalent SDF system is expressed:

$$E_0 = \varphi_i E_{0i} \tag{19}$$

where structural index E_{0i} is evaluated as the larger of Eqs. (8) and (9) at story i.

STRUCTURAL IRREGULARITY

Structural configuration may be irregular at a story. The Building Standard Law uses structural configuration factor F_{es} to amplify story resistance required for an irregular distribution of stiffness along the height of a structure and also for a large eccentricity of mass center with respect to the center of rigidity in a floor plan. The structural configuration factor F_{es} at each story is calculated as the product of factors F_s and F_e representing the irregularity in stiffness distribution along height and eccentricity in plan, respectively;

$$F_{es} = F_s\ F_e \tag{20}$$

The regularity in stiffness distribution along structural height is judged by the value of rigidity ratio R_s at each story:

$$R_s = \frac{\gamma_i}{\bar{\gamma}} \tag{21}$$

in which, γ_i: reciprocal of elastic drift angle (inter-story drift divided by inter-story height) calculated at story i under design earthquake forces, $\bar{\gamma}$: average value of γ_i's at all stories. Factor F_s is 1.0 for $R_s \geq 0.6$, 2.0 for $R_s = 0.0$, and is interpolated in the range $0.0 < R_s < 0.6$ (Fig. 3(a)). Factor F_s is extremely important to prevent soft first-story collapse of a building typically observed in reinforced concrete residential buildings in Kobe.

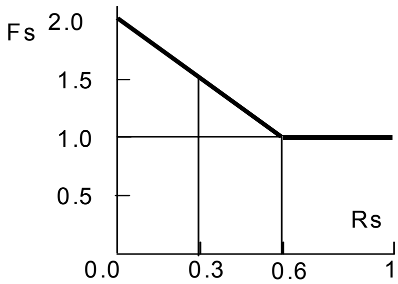
Eccentricity ratio R_e is defined as a ratio of eccentricity e between the center of mass and the center of stiffness to the elastic radius r_e of stiffness in the story;

$$R_e = \frac{e}{r_e} \tag{22}$$

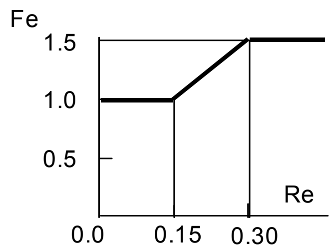
Mass center of a story is determined from the column axial forces under gravity loads. Stiffness center is determined for the lateral stiffness of vertical members; the lateral stiffness of a vertical member is defined as a ratio of the shear to the inter-story drift of the member under design earthquake forces. Elastic radius r_{ex} in the x-direction in plan is defined as the square root of the ratio of the torsional resistance with respect to the stiffness center to the sum of lateral resistance;

$$r_{ex} = \sqrt{\frac{\sum K_x\, \bar{y}^2 + \sum K_y\, \bar{x}^2}{\sum K_x}} \tag{23}$$

where, K_x and K_y: lateral stiffness of a vertical member at distance $\bar{x}$ and $\bar{y}$ in x and y directions from the stiffness center. Factor F_e is 1.0 for $R_e \leq 0.15$, 1.5 for $R_e \geq 0.30$, and interpolated in the range $0.15 < R_e < 0.30$ (Fig. 3(b)).



(a) Discontinuity in stiffness along height



(b) Eccentricity in plan

Fig. 3 Amplification of design story shear for irregularity in the 1981 Building Standard Law

Structural index E_0 of the i-th story in terms of an equivalent SDF system must be further modified for the irregularity;

$$E_0 = \frac{\varphi_i \cdot E_{0i}}{F_{es}} \tag{24}$$

PROCEDURE OF SEISMIC VULNERABILITY ASSESSMENT

Ultimate strengths of structural members are first evaluated for failure modes in shear or flexure. A building is analyzed under lateral loading to failure using, for example, limit analysis of collapse mechanism or nonlinear static analysis under monotonically increasing load. Comparing the member actions at structural failure and member strengths, ductility indices are assigned to vertical members (columns and walls) as given in Table 2 (for reinforced concrete and steel reinforced concrete composite structures). If plastic hinges do not form in a vertical member, the ductility index of the member should be determined looking at plastic hinge formation in connected girders and also overall formation of collapse mechanism of the structure.

Columns and structural walls of a story are classified into three representative groups by their ductility capacity index F; the group is numbered from lowest to highest ductility indices. Shears carried by vertical members in group i are summed to define story shear Q_i of the group. Story shear Q_u is also calculated at the failure of a group that carries largest story shear and the ductility index F of the group is selected.

Structural index E_0 may be taken as the larger value of Eqs. (25) and (26);

$$E_0 = \frac{\varphi_i}{F_{es}} \cdot C \cdot F = \frac{1}{A_i \cdot F_{es}} \cdot \left(\frac{Q_u}{W_i}\right) \cdot F \tag{25}$$

$$E_0 = \frac{\varphi_i}{F_{es}} \cdot \sqrt{(C_1 \cdot F_1)^2 + (C_2 \cdot F_2)^2 + (C_3 \cdot F_3)^2}$$
$$= \frac{1}{A_i \cdot F_{es}} \sqrt{\left(\frac{Q_1}{W_i} \cdot F_1\right)^2 + \left(\frac{Q_2}{W_i} \cdot F_2\right)^2 + \left(\frac{Q_3}{W_i} \cdot F_3\right)^2} \tag{26}$$

where, Q_u : maximum story shear carrying capacity, F_j : ductility index (Table 2) of member group j (columns and structural walls) in story i, W_i: total dead and live loads which story i supports, A_i: factor representing vertical distribution of a seismic story shear coefficient given by Eq. (18). Equation (26) should not be used if critical load carrying members exist in a structure.

Table 2 Ductility index F of members and failure mode (RC members)

Failure mode of columns and walls	Value
Highly ductile columns without fear of shear failure	3.2
Columns in highly ductile frame (girder flexural yielding)	3.0
Ductile columns unlikely to fail in shear	2.2
Columns connected to girders likely to fail in shear	1.5
Not ductile columns, but unlikely to fail in shear	1.3
Less ductile columns likely to fail in shear	1.0
Brittle columns likely to fail in shear	0.8
Structural walls rotating at the base under lateral loading	3.0
Ductile structural walls without fear of shear failure	2.0
Structural walls likely to fail in shear	1.0

Structural seismic capacity index I_s is evaluated by Eq. (5). For a structure to resists earthquake motions in the code, the index I_s should be greater than 1.0. However, in the past earthquakes, those reinforced concrete buildings having structural seismic capacity index I_s greater than 0.6 suffered none or small damage.

An index q of structural lateral force resisting capacity is defined by Eq. (27);

$$q = \frac{Q_u}{F_{es}\ W_i\ Z\ R_t(T)\ A_i\ S_t} \tag{27}$$

where, S_t: minimum base shear coefficient 0.30 required for very ductile reinforced concrete construction in the Building Standard Law. A reinforced concrete building designed and constructed in accordance with the current Building Standard Law possesses a story shear resistance defined by the denominator of Eq. (27).

Table 3 Seismic vulnerability assessments

Structural seismic capacity index I_s and lateral force capacity index q	Vulnerability assessment
$I_s < 0.3$ or $q < 0.5$	Likely to collapse
others	Possible to collapse
$I_s \geq 0.6$ and $q \geq 1.0$	Unlikely to collapse

The seismic vulnerability of a story is assessed by structural seismic capacity index I_s and lateral force resisting capacity index q as shown in Table 3. The structure may be considered to be safe when structural seismic capacity indices I_s of every story are greater than 0.6 and lateral force capacity indices q of every story are greater than 1.0. Even if the structural seismic capacity index I_s is greater than 0.6, if the story shear strength is not large enough (e.g. index q less than 1.0), extensive damage may be developed in the story. It should be noted that the damage (ductility demand) is much less in a stronger structure than in a weaker structure if the structural seismic capacity index is the same in the two buildings.

Roofing materials should not fall off by the vibration during an earthquake. Chimneys and water tanks on the roof should have sufficient strength. Water supply and drainage facilities should be provided with sufficient strength for safety.

SUMMARY

The seismic vulnerability assessment method is briefly outlined for existing reinforced concrete buildings in Japan. The method recognizes the strength and ductility of a building, sequence of failure of less ductile to more ductile members. The earthquake resisting capacity must be compared with an index to characterize the earthquake damaging power. The reliability of the procedure needs to be examined with respect to the damage in buildings.

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