

Seismic Vulnerability Assessment of Reinforced Concrete Buildings

By

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The development of seismic vulnerability evaluation standards for reinforced concrete buildings in Japan is briefly reviewed, with emphasis on the assessment for low- to mid-rise reinforced concrete buildings. 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, Japanese screening procedures are briefly introduced which recognize the different resistance and ductility levels of structural members and the sequence of failure from brittle to ductile members. Seismic vulnerability is assessed by comparing the earthquake resistance capacity of a building and the seismic input. The earthquake characteristics are represented by the response spectrum at engineering bedrock, amplification of ground motion by soil deposits and the seismicity of the region. A procedure consistent with the present design provisions is briefly introduced.

1. 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. The following countermeasures are needed to mitigate earthquake disasters: i.e.,

- (a) Effective earthquake resistant building codes,
- (b) Earthquake vulnerability assessment methods for existing buildings,
- (c) Seismic strengthening technology for vulnerable buildings,
- (d) Seismic damage evaluation methods of buildings after an earthquake,
- (e) Technology to repair damage for immediate occupancy, and
- (f) Technology to strengthen damaged buildings for permanent use.

The development in Japan is briefly reviewed in the following section.

The 1968 Tokachi-oki earthquake caused significant damage, for the first time in Japan, to reinforced concrete buildings, which had been believed by engineers, researchers and the general public to be earthquake resistant since the 1923 Kanto earthquake. Reinforced concrete columns failed in shear in school buildings in the earthquake. The concern was expressed by many organizations about the earthquake safety of existing reinforced concrete buildings; e.g., the Ministry of Education

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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⁾. The standard was revised in December 1990. The reliability of the method was tested after several earthquakes including the 1978 Miyagi-ken Oki earthquake and the 1995 Hyogo-ken Nanbu earthquake (the Kobe earthquake disaster) for school buildings.

The Ministry of Construction sponsored a coordinated technical research project (1981-1985) for the development of post earthquake measures and the development of repairing and strengthening technique²⁾. The Guidelines³⁾ for post earthquake inspection and assessment of earthquake damages were published in February 1991. The guidelines were used after the 1995 Hyogo-ken Nanbu earthquake to evaluate the damage level of affected buildings and the need for strengthening.

After the 1995 Hyogo-ken Nanbu earthquake, Japanese Diet, recognizing the urgent importance of improving seismic resistance of existing buildings, proclaimed a law⁴⁾ to promote the seismic strengthening of existing buildings. The law, enforced on December 25 1995, requires that the owner of a "specially designated building" should make efforts to assess seismic vulnerability of the structure. The law also requires that the owner should make efforts to strengthen the structure if needed. The specially designated building is defined as an existing building that does not satisfy earthquake resistant building requirements of the current Building Standard Law, and that is used by a number of people such as schools, gymnasiums, hospitals and medical offices, theaters, assembly halls, exhibition halls, department stores, banks, offices, markets and stores, hotels, restaurants, apartment buildings, dormitories, nursery homes, public baths, factories, transportation stations, automobile garages, and government offices.

The Ministry of Construction initiated a coordinated technical research project (1995-1998) for the development of new technology for earthquake disaster reduction in large cities. As a part of the project, the Building Research Institute, Ministry of Construction, organized a research program to develop earthquake disaster mitigation methodologies through the application of new technology from April 1996 to March 1999. The research has been carried out in three subject areas; i.e., (a) the development of methods to strengthen structural members, (b) the development of methods to control structural response, and (c) the development of methods to strengthen foundation structures. A strong emphasis was placed on the development of methodology for the retrofitting work without interrupting the use of a building or without significantly disturbing the resident by noise or vibration⁵⁾.

Severe earthquake disasters occurred in Turkey and Taiwan in 1999. Old buildings collapsed in these events, which re-emphasized the importance of seismic vulnerability assessment of existing buildings in seismic regions. This paper summarizes the methods of seismic vulnerability assessment used in Japan.

2. 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 after the 1985 Mexico earthquake⁶⁾, the 1990 Luzon, Philippines, earthquake⁷⁾, the 1992 Erzincan, Turkey, earthquake⁸⁾, and the 1995 Hyogo-ken Nanbu earthquake⁹⁾. 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 was classified to (a) none or light damage, (b) minor damage, (c) medium (moderate) damage, (d) major damage, and (e) collapse.

The 1985 Mexico Earthquake

An M8.1 earthquake occurred on the Mexican West Coast on September 19, 1985, followed by an M7.5 after-shock on September 21. The two successive events caused a significant damage in Mexico City approximately 400 km away from the epicenter. The structural damage was concentrated in a region of old lakebed while the damage was light in an area of stiff soil. The damage statistic⁶⁾ of RC and masonry buildings in Fig. 1 was obtained one to two months after the earthquake in the old lakebed zone. The investigated area included heavily damaged residential as well as commercial districts, covering slightly more than 20 percent of the central city. Ninety-four (94.3) percent of the buildings suffered light to minor damage in the surveyed areas. More than 94 percent of these low-rise buildings (six stories or lower) suffered very light damage. The percentage of severely damaged buildings increased with the number of stories. The damage level was high especially in buildings of 7 stories and higher. More than half of nine-story and taller buildings suffered heavy damage or collapsed.

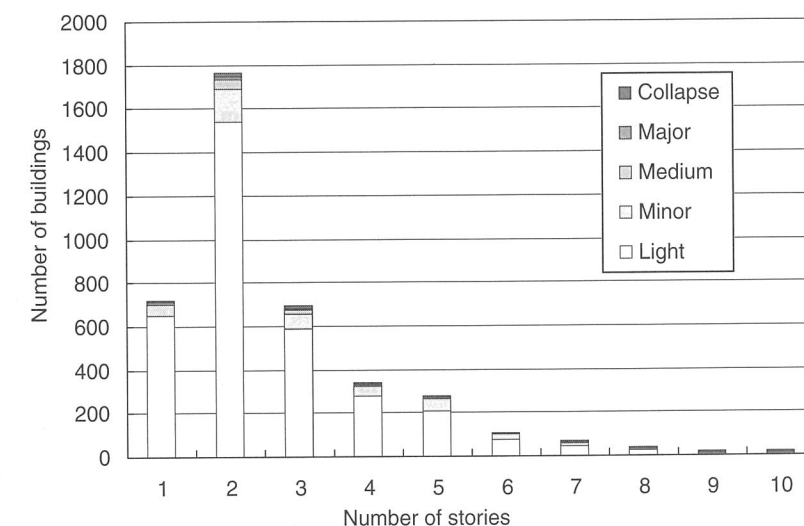


Fig. 1: Damage statistics of reinforced concrete and masonry buildings in Mexico City, 1985

The 1990 Luzon, Philippines Earthquake

An earthquake (M7.7) occurred approximately 100 km to the north of Manila, Philippines, on July 16, 1990, and killed approximately 2,000 people. Many high-rise hotel buildings collapsed in the resort city of Baguio. The damage statistics of RC buildings⁷⁾ were collected in a major commercial district of the city of Baguio as shown in Fig. 2. Seventy-six (76.2) percent of RC buildings suffered light to minor damage, 18.8 percent suffered medium to major damage, and 5.0 percent collapsed out of 181 buildings surveyed in the area. Severe damage was observed in taller buildings although the number of these high-rise buildings was small. The percentage of heavy damage and collapsed buildings was higher in the low-rise buildings (six-story or lower).

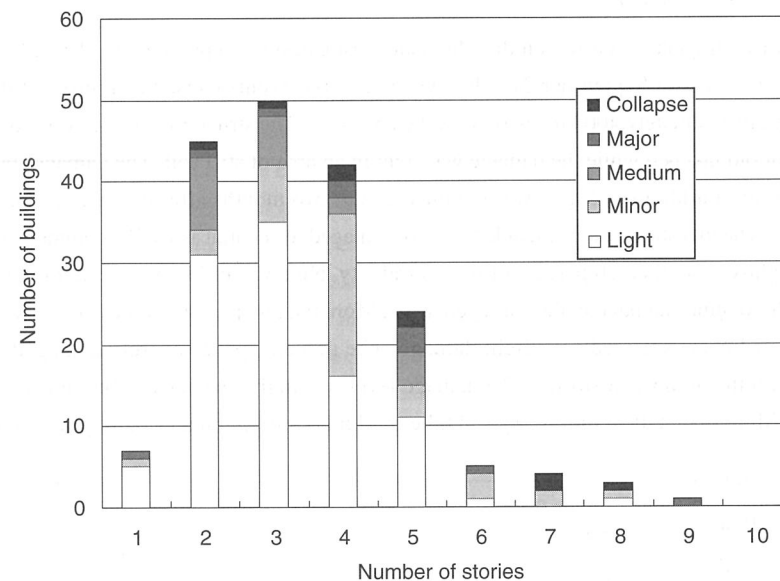


Fig. 2: Damage statistics of reinforced concrete and masonry buildings in Baguio, 1990

The 1992 Erzincan, Turkey, Earthquake

An Ms 6.9 earthquake occurred on March 13, 1992, with an epicenter (focal depth of 28 km) near the City of Erzincan, in the eastern part of the Anatolia Plateau, Turkey. In the City of Erzincan, two heavily damaged residential areas were selected for the inventory survey of RC buildings by joint AIJ-JSCE (Japan Society of Civil Engineers)-Bogazici University team⁸⁾. The damage of one- and two-story buildings was very light as shown in Fig. 3. No three-story buildings collapsed, but 23 percent of the buildings suffered medium to major damage in the surveyed areas. Seventeen (17.3) percent of four-story buildings collapsed and 30.9 percent suffered medium to major damage.

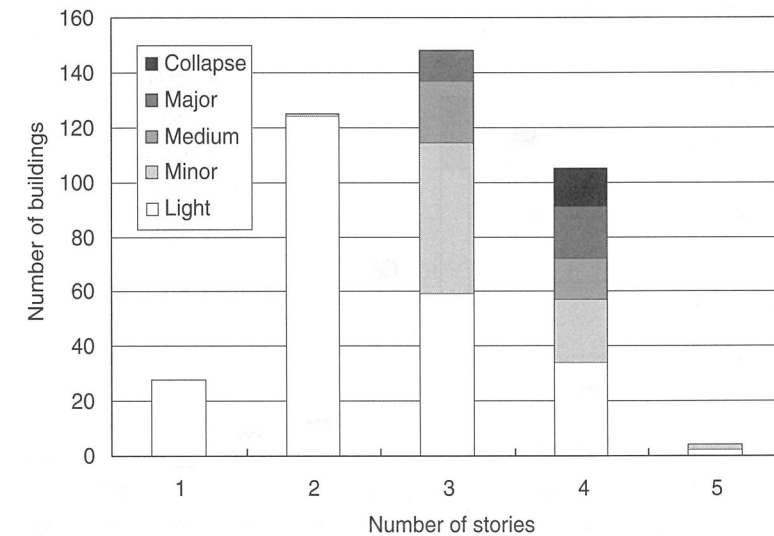


Fig. 3: Damage statistics of reinforced concrete and masonry buildings in Erzincan, 1992

The 1995 Hyogo-ken Nanbu Earthquake

The 1995 Hyogo-ken Nanbu Earthquake killed 6,432 people (including direct and indirect causes by the earthquake), injured more than 40 thousands people. The Kinki Branch, Architectural Institute of Japan (AIJ), investigated the damage of all existing reinforced concrete (RC) buildings (3,911 buildings in total) in the region of seismic intensity VII (highest seismic intensity defined by the Japan Meteorological Agency) in Nada and Higashi-Nada wards, Kobe City⁹⁾.

Eighty-nine (88.5) percent of the buildings surveyed survived with light to minor damage, 5.9 percent suffered medium to major damage and 5.7 percent collapsed (Fig. 4). Out of those 2,035 buildings constructed before the current (1981) Building Standard Law, 7.4 percent suffered heavy damage and 8.3 percent collapsed. Out of those 1,859 buildings constructed using the current Building Standard Law, 3.9 percent suffered heavy damage and 2.6 percent collapsed. Even after the revision of the law, 20 percent of buildings taller than seven stories suffered heavy damage or collapsed. The ratio of severer damage increases with the number of stories, especially for buildings taller than 5 stories.

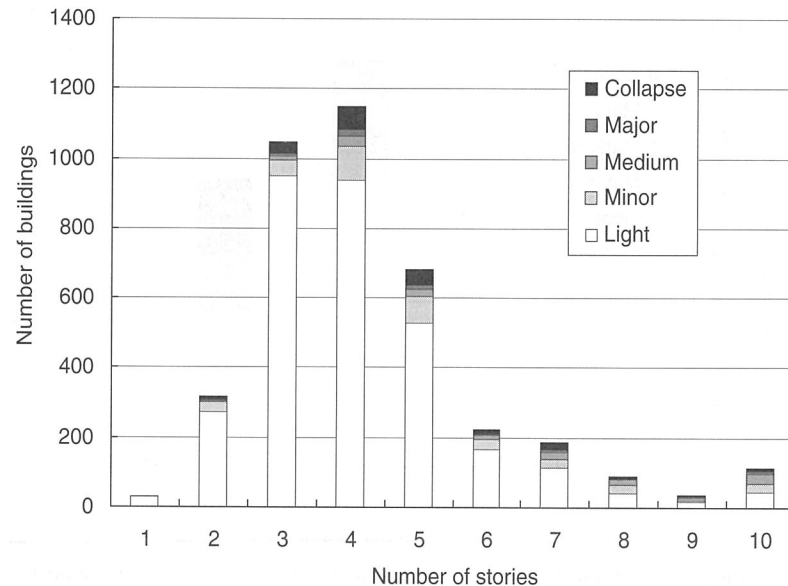


Fig. 4: Damage statistics of reinforced concrete buildings in Kobe, 1995

Discussion

The damage statistics⁶⁻⁹⁾ by the AIJ investigation teams showed 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 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. A definite trend is observed in the damage statistics that the percentage of heavy damage increased with the number of stories. A more careful examination is needed for mid-rise buildings.

3. PRINCIPLES OF SEISMIC RESISTANCE ASSESSMENT

The lateral load strength is not a single index to represent the safety of a building. Deformation capability of constituent members, structural configuration, foundation, site conditions, soil-structure interaction, quality of workmanship, importance of buildings, structure's age and hazard history need to be taken into account in seismic vulnerability assessment.

Basic Concept

A structure can resist strong earthquake motion without collapse if the structure is provided with either (a) sufficient lateral load resistance or (b) limited lateral load resistance but with sufficient ductility. A high lateral-force resisting structure and a low lateral resistance structure but with large

deformation capacity in Fig. 5 can both survive an earthquake motion as long as the maximum response does not exceed the limiting capacity.

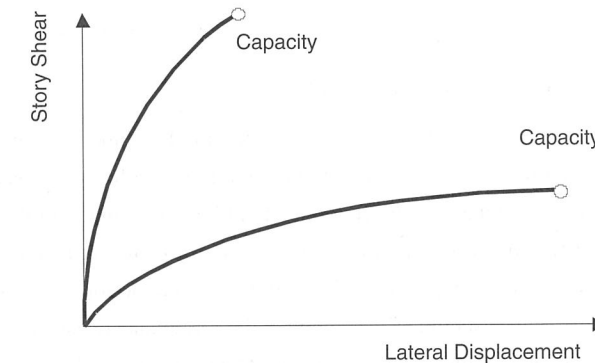


Fig. 5: Story shear-story drift relation

Recall Newmark's well-known design criteria¹⁰⁾, which determined 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 .

$$C_y = \frac{C_e}{\sqrt{2\mu - 1}} \quad \text{for short period systems} \quad (\text{Eq. 1})$$

$$C_y = \frac{C_e}{\mu} \quad \text{for long period systems} \quad (\text{Eq. 2})$$

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

$$C_e = C_y \sqrt{2\mu - 1} \quad \text{for short period systems} \quad (\text{Eq. 1'})$$

$$C_e = C_y \cdot \mu \quad \text{for long period systems} \quad (\text{Eq. 2'})$$

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. It should be noted that the Newmark's design criteria are not exact in scientific terms, but that the criteria should be considered as a crude design guide.

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

$$E_o = C \cdot F \quad (\text{Eq. 3})$$

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).

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 structural members, exhibiting the lateral load deformation relationships shown in Fig. 6. The failure of stiff and less ductile members may significantly reduce the resistance of the structure, but the ductile members may 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_o is evaluated at the failure of the less ductile members (Eq. 4) and at the failure of the ductile members (Eq. 5).

$$E_o = (C_1 + \alpha_2 C_2) \cdot F_1 \quad (\text{Eq. 4})$$

$$E_o = \sqrt{E_1^2 + E_2^2} \quad (\text{Eq. 5})$$

$$E_i = C_i \cdot F_i \quad \text{for } i = 1, 2 \quad (\text{Eq. 6})$$

in which $(C_1 + \alpha_2 C_2)$: total strength index at failure of the less ductile system, $E_i (i = 1, 2)$: structural index for the less ductile and the ductile members, $F_i, C_i (i = 1, 2)$: ductility and strength indices, respectively, of the less ductile and the ductile members. The larger value of the two equations can be taken as structural index E_o . The structural index estimates in this manner is smaller than the sum of the two indices ($E_1 + E_2$). The same concept may be used for a structure composed of more than two representative member groups.

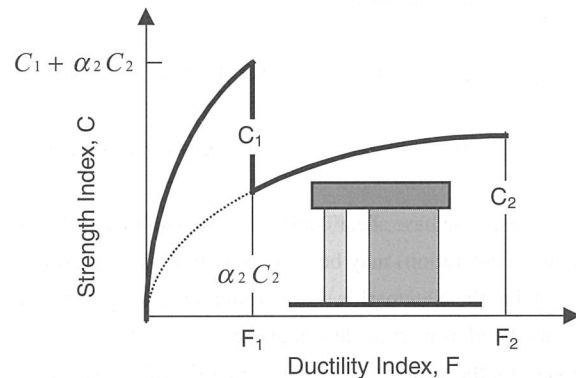


Fig. 6: Force-deformation relation of two-member system

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. 4 should be used to calculate structural index E_o .

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. If a structural wall is attached to such a susceptible column, and if the span lengths to adjacent columns are less than 7.0 m, the column is not considered as a critical load carrying member.

Multi-story Structures

For a multi-story (MDF) structure, structural index E_o 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 above the story concerned. Structural index E_{oi} of story i must be interpreted into that 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 $\{\phi\}_1$, participation factor γ_1 , acceleration spectral value and mass matrix;

$$\{f\}_1 = [m] \{\phi\}_1 \gamma_1 S_a \quad (\text{Eq. 7})$$

where the participation is defined as

$$\gamma_1 = \frac{\{\phi\}_1^T [m] \{1\}}{\{\phi\}_1^T [m] \{\phi\}_1} \quad (\text{Eq. 8})$$

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

$$V_i = \sum_{j=i}^n f_{1j} = \sum_{j=i}^n m_j \phi_{1j} \gamma_1 S_a \quad (\text{Eq. 9})$$

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 given by dividing story shear V_i by the total weight above the story;

$$C_i = \frac{V_i}{\sum_{j=i}^n m_j g} = \frac{\gamma_1 S_a}{g} \frac{\sum_{j=i}^n m_j \phi_{1j}}{\sum_{j=i}^n m_j} \quad (\text{Eq. 10})$$

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

$$C = \frac{S_a}{g} = \phi_i \cdot C_i = \frac{1}{\gamma_1} \frac{\sum_{j=i}^n m_j}{\sum_{j=i}^n m_j \phi_{1j}} C_i \quad (\text{Eq. 11})$$

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 ;

$$\phi_i = \frac{1}{\gamma_1} \frac{\sum_{j=i}^n m_j}{\sum_{j=i}^n m_j \phi_{1j}} \quad (\text{Eq. 12})$$

If a linear deflected shape is assumed for the fundamental mode of a structure with uniform story height and mass distribution, story index ϕ_i is expressed as

$$\phi_i = \frac{2}{3} \frac{2n+1}{n+i} \quad (\text{Eq. 13})$$

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 standard¹⁾ to consider crudely higher mode contribution at upper stories;

$$\phi_i = \frac{n+1}{n+i} \quad (\text{Eq. 14})$$

Equation 14 represents the ratio of the base shear coefficient to a story shear coefficient for a uniform building (uniform story height and mass distribution) under the linear deflected shape. This is commonly used in Japanese screening procedures.

Structural index E_{oi} of the i -th story in terms of an equivalent SDF system is expressed:

$$E_{oi} = \phi_i \cdot C_i \cdot F_i \quad (\text{Eq. 15})$$

Structural Irregularity and Structural Decay with Age

The seismic resistance of a system should be modified by taking into account the irregularity of structural configuration and the deterioration of structural properties with age. Therefore, structural seismic capacity index I_s of a structure is expressed as

$$I_s = E_o \cdot S_D \cdot G \cdot T \quad (\text{Eq. 16})$$

where S_D : configuration index, T : age index. Configuration index S_D takes into consideration ill effect on the response by (a) distribution of lateral stiffness and resistance in plan shape, (b) distribution of stiffness along height, (c) distribution of story heights, and (d) existence of basement. Age index T recognizes the deterioration in earthquake resistance by (a) existing cracks, (b) observed deflection, (c) uneven settlement caused by foundation deformation, (d) neutralization of concrete, and (e) rust on reinforcement, which must be determined on the basis of the investigation of building conditions at the building site.

Definition of Earthquake Ground Motions

The safety of a building under an earthquake motion cannot be discussed from the structural point of view only, but the characteristics of the earthquake motion must be taken into consideration. Buildings must be protected against ground motions caused by a large magnitude earthquake at far distance as well as more frequent medium magnitude earthquakes at close distance. An earthquake motion may be represented by dominant frequency contents, the intensity level and duration of ground motion. These characteristics are influenced by the site characteristics, the magnitude, source mechanism and epicentral distance of an earthquake. A transfer function of ground motion from an engineering bedrock to the ground surface, and the motion at the engineering bedrock by considering the epicentral distance, earthquake source mechanism and magnitude are necessary. The state of the art in seismic engineering cannot define the time history of critical earthquake motions for a building, and the response spectrum may be assumed to define the approximate characteristics of critical earthquake motions.

Thus, the characteristics of a ground motion are expressed by the required seismic capacity index I_{so} which is defined as follows;

$$I_{so} = E_s \cdot Z \cdot G \cdot U \quad (\text{Eq. 17})$$

where, E_s : seismic index, Z : seismicity index, G : soil index, U : importance index. Seismic index E_s defines the shape of an acceleration response spectrum at the engineering bedrock as a function of period. Seismicity index Z defines the seismic activity at the construction site. The seismicity index may be taken the same as the seismic zone factor in the Building Standard Law. Soil index G defines the amplification of ground motion by the surface soil deposits. Soil index G , for example, is 1.0 for normal cases, 1.25 for the construction site adjacent to steep slopes or irregular soil conditions or for local hill site. If a transfer function of motion from the bedrock to the ground surface can be defined, such function can be used for soil index G . In other words, the product of the seismic index, seismicity

index and soil index represents the acceleration response spectra of critical ground motion on the free surface. Importance index U defines the level of safety required for the use of the structure and may be selected by the owner of the building.

The structure may be judged safe if the structural seismic capacity index I_s is greater than the required seismic capacity index I_{so} . The smaller is the value of index I_s compared with that of index I_{so} , the higher is the probability of severe damage. However, the approximate nature of the assessment method and uncertainties in ground motions will not permit decisive assessment about the damageability of a building.

4. SCREENING PROCEDURES USED IN JAPAN

The Standard for Seismic Vulnerability Assessment of Existing Reinforced Concrete Buildings¹⁾, intended for low-rise buildings less than seven-stories, uses three-level screening procedures. The application of the standard is limited to low-rise buildings because the standard assumes constant acceleration response amplitude with periods of buildings (flat response acceleration spectrum with periods). The value of seismic index E_s for use in determining required seismic capacity index I_{so} has been selected to yield the safety level similar to the building designed and constructed in conformance with the present Building Standard Law.

A simple screening procedure is intended to identify majority of earthquake resistant buildings by examining story shear strength provided by columns and structural walls. Those buildings, identified as questionable by the simple procedure, must be analyzed by more sophisticated second-level procedure considering deformation capacity of vertical members. The third-level procedure is a general and detail procedure and requires the nonlinear static or dynamic analysis of the entire structure.

First-Level Screening Procedure

The first-level screening procedure is intended to find out those buildings provided with lateral resistance sufficient to resist earthquakes. It should be noted that Japanese buildings has been designed in accordance with the Building Standard Law since 1950 using allowable stress procedure and a constant seismic coefficient of 0.2 (lateral floor force divided by the weight of the floor). For bending resistance, the allowable stress of concrete is two-thirds of the specified compressive strength and that of reinforcement is the nominal yield stress. In shear resistance evaluation, the contribution of lateral reinforcement was not recognized before 1971 (the spacing of ties in columns was made less than 100 mm in the end regions by the 1971 revision of the Building Standard Law), and the entire design shear must be resisted by the concrete at an allowable shear stress $(0.75 + 0.015 F_c)$ (MPa), where F_c : concrete compressive strength). In order to meet the high seismic design forces and low allowable shear stress requirements, the Japanese building is provided with large columns and walls. At the same time, the importance of structural walls was emphasized in design and, hence, structural walls were placed in a building wherever possible. Therefore, a Japanese building is believed to possess lateral strength larger than that required by the code. The first-level screening procedure is to identify these strong buildings, especially with many structural walls, by a simple calculation.

The lateral strength of a story is crudely evaluated by examining the shear strength of columns and walls by their cross sectional areas. The strength of girders is not examined at this stage because (a) the column is believed to be more vulnerable to earthquake forces, (b) the failure of columns will lead to the collapse of a building, and (c) the girder is believed to be more ductile.

The vertical members are grouped into

- (a) short columns (clear height to depth ratio less than 2),
- (b) walls and
- (c) columns.

A crude and conservative shear strength per unit sectional area of short columns and columns was estimated to be 1.5, 1.0 MPa, respectively, on the basis of dimensions, materials, reinforcement ratios of columns, commonly used in reinforced concrete buildings in Japan. Unit shear strength of walls is estimated to be 3.0 MPa for walls with boundary columns on both sides, 2.0 MPa for walls with boundary columns only on one side, and 1.0 MPa for walls without boundary columns. The cross sectional area of boundary columns is not considered to resist shear for a wall.

Accordingly, story shear coefficients C_j carried by the three group members ($j = 1$ for short columns, $j = 2$ for walls, and $j = 3$ for columns), are estimated separately by multiplying cross sectional areas and corresponding unit shear strength of the group members.

Short columns are likely to fail in a brittle shear mode, and a small ductility index ($F = 0.8$) is assigned. The wall and column are assumed to develop 70% and 50% of their strength, respectively, when the short column fails in shear. Structural index E_{oi} of story i is evaluated by Eq. 18 at the failure of short columns;

$$E_{oi} = \frac{n+1}{n+i} (C_1 + 0.7 \cdot C_2 + 0.5 \cdot C_3) \times 0.8 \quad (\text{Eq. 18})$$

If short columns do not exist in a story or if the failure of short columns will not lead to the collapse of the story, structural index E_{oi} of story i is estimated by the following expression at the failure of the wall;

$$E_{oi} = \frac{n+1}{n+i} (C_2 + 0.7 \cdot C_3) \times 1.0 \quad (\text{Eq. 19})$$

where, ductility index F_2 of the wall is selected to be 1.0, and 70 percent of the column strength is assumed to develop at the failure of the wall (Fig. 7).

If there is no structural wall in a story ($C_2=0.0$), then the structural index is estimated by Eq. 20;

$$E_{oi} = \frac{n+1}{n+i} C_3 \times 1.0 \quad (\text{Eq. 20})$$

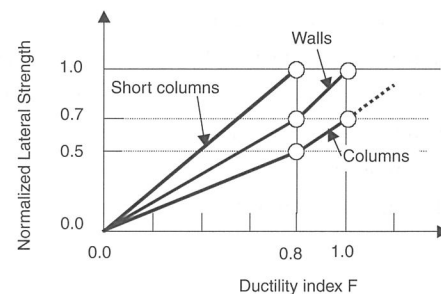


Fig. 7: Strength and deformation relation in first-level screening procedure

Structural seismic capacity index I_s is expressed by Eq. 16. The irregularity of structural configuration and decay of structural properties with age were crudely considered in the procedure. Configuration index S_D considers irregularity in plan, longitudinal-to-transverse plan length ratio, expansion joints, existence of basement, and abrupt discontinuity of stiffness along structural height, especially soft first story. A simple grading chart is provided to determine the configuration index which varies from 0.43 to 1.2. The first-level screening procedure should not be used if a large eccentricity exists in a floor. The observed deformation of a building caused by uneven settlement of foundation, cracks in columns and walls, past and present use of chemicals, past fire experiences, finishing conditions and building age, are considered in evaluating age index T which varies from 0.7 to 1.0.

Required seismic capacity index I_{so} is defined in Eq. 17. Seismic index E_s is conservatively chosen to be 0.8 for low-rise buildings (less than 7 stories) on the basis of past earthquake experiences.

If structural seismic capacity index I_s is more than required seismic capacity index I_{so} , the structure is judged "safe" against earthquake motions observed in the 1968 Tokachi-oki earthquake, the 1978 Miyagi-ken Oki earthquake or the 1995 Hyogo-ken Nanbu earthquake. If index I_s is less than index I_{so} but more than 0.65 I_{so} , the structure is thought to possess reasonable seismic resistance, but the vulnerability assessment by the second-level screening procedure is recommended.

Second-Level Screening Procedure

The second-level screening procedure is to be applied if any building is found vulnerable or questionable by the first-level screening procedure. In this procedure again, the girder is assumed to be infinitely stiff and strong. The ductility capacity of columns and walls is estimated crudely for their failure modes (shear or flexure) and on the basis of shear-to-flexural strength ratios. The combination of different ductility levels and shear resistances of vertical members were considered in estimating the earthquake resistance of a structure.

The shear resistance of vertical members (columns and walls) must be calculated at the formation of flexural yielding at the member ends and at shear failure on the basis of the member geometry, the amount of longitudinal and lateral reinforcement, and concrete strength. Failure mode, either flexure or shear, is determined by comparing the shear strength and the flexural strength, where the flexural strength is defined as shear force acting at the flexural yielding at member ends. In general, the flexural

strength of a structural wall becomes unrealistically high if the lateral resistance is evaluated for flexural yielding at the top and the bottom of a story. Therefore, a uniform horizontal force distribution is assumed to act on a structural wall. The flexural strength of a wall at a story is calculated as a shear force acting at the flexural yielding at the base of the story. The flexural strength of a column and a wall may be calculated for the axial force acting under the gravity loading condition.

Vertical members are classified into one of the five groups depending on the failure mode; i.e.,

- Flexural column (the flexural strength lower than the shear strength),
- Flexural wall (the flexural strength lower than the shear strength),
- Shear column (the shear strength lower than the flexural strength),
- Shear wall (the shear strength lower than the flexural strength), and
- Short column (shear column with clear height to depth ratio less than 2.0).

A value of ductility index F is assigned to each type members (Table 1), considering failure mode, axial load level, shear stress level at member strength, tensile reinforcement ratio, and shear span to depth ratio. Ductility index F of a flexural column is estimated by the following expression:

$$F = \frac{4\sqrt{2\mu - 1}}{3(1 + 0.05\mu)} \quad (\text{Eq. 21})$$

where, μ : ductility factor of the flexural column, but $1.0 \leq \mu \leq 5.0$. The ductility factor of a flexural column is estimated as a function of the shear strength to the flexural strength ratio, spacing of ties, and shear stress to concrete strength ratio at flexural strength. However, the ductility index of a flexural column is 1.0 if (a) axial stress is more than 0.4 times the concrete strength, or (b) shear stress is more than 0.2 times the concrete strength, or (c) tensile reinforcement ratio is greater than 0.01, or (d) clear height-to-depth ratio is less than 2.0. Ductility index F of a flexural wall is 1.0 if the shear strength is less than 1.2 times the flexural strength, and 2.0 if the shear strength is more than 1.3 times the flexural strength. A linear interpolation is used in-between.

Table 1 Ductility Index for the second level screening procedure

Member Type	Ductility index
Flexural columns	1.27 - 3.2
Flexural walls	1.0 - 2.0
Shear columns	1.0
Shear walls	1.0
Extremely short columns	0.8

The vertical members (columns and walls) are classified into three groups; each group is represented by the smallest ductility index of grouped members. Three groups are number from the smallest to the largest ductility indices (F_1 , F_2 , and F_3), and corresponding strength indices C_1 , C_2 , and C_3 of the groups are evaluated.

Structural index E_{oi} at story i is calculated as the larger of the two expressions;

$$E_{oi} = \frac{n+1}{n+i} (C_1 + \alpha_2 C_2 + \alpha_3 C_3) \cdot F_1 \quad (\text{Eq. 22})$$

$$E_{oi} = \frac{n+1}{n+i} \sqrt{E_{1i}^2 + E_{2i}^2 + E_{3i}^2} \quad (\text{Eq. 23})$$

where, α 's are the ratio of story shear of group 2 or 3 members, respectively, resisted at the failure of group 1 members to the shear capacity of corresponding group members. The values of α 's are selected from the story shear coefficient - story drift angle relations in Fig. 8 on the basis of experimental results of typical reinforced concrete members. If extremely short columns exist in a story, Eqs. 22 and 23 must be evaluated neglecting the extremely short columns and Eq. 22 including the extremely short columns. The largest value can be used. If there exist critical load carrying members in a story, Eq. 22 must be used considering the extremely short columns.

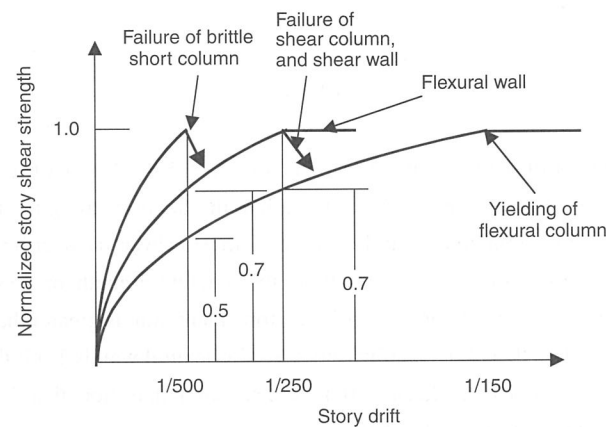


Fig. 8: Story shear coefficient-drift angle relations (Second level screening)

If the eccentricity between the mass and stiffness centers is large, structural index E_o should be taken the smaller of the values calculated neglecting the vertical members causing the large eccentricity and the value calculated by Eq. 22 at the failure of the vertical members causing the large eccentricity.

Structural seismic capacity index I_s is calculated by Eq. 16 as the product of structural index E_o , configuration index S_D and age index T . Configuration index S_D considers eccentricity of mass and stiffness centers and the change in weight-stiffness ratios of each story. An eccentricity ratio is defined as the eccentricity distance divided by the radius of gyration. Weight-stiffness ratio is defined as the story stiffness divided by the weight the story support. The configuration index varies from 0.42 to 1.42. Age index T is common for all stories and is an average of age indices at the number of stories investigated for the deterioration. Grading points are assigned to the level of structural cracks and deformation and for the deterioration with age separately in floor slabs, girders, and columns and walls. The age index varies from 0.49 to 1.0.

Structural seismic capacity indices of 1,615 reinforced concrete buildings, mainly three to four storied school buildings, were evaluated in Shizuoka prefecture where the probable occurrence of a major earthquake was warned in 1978. Majority of the buildings were constructed before the 1971 revision of the Building Standard Law. The distribution of the structural seismic capacity indices¹¹⁾ is shown in Fig. 9. The frequency of moderately or severely damaged buildings in the 1968 Tokachi-oki and the 1978 Miyagi-ken Oki earthquake is normalized in percentage and shown in shaded columns. No building was damaged if the structural seismic capacity index was larger than 0.6 from these earthquakes. If index I_s becomes smaller than 0.6, the probability of severe damage increases with decreasing index value.

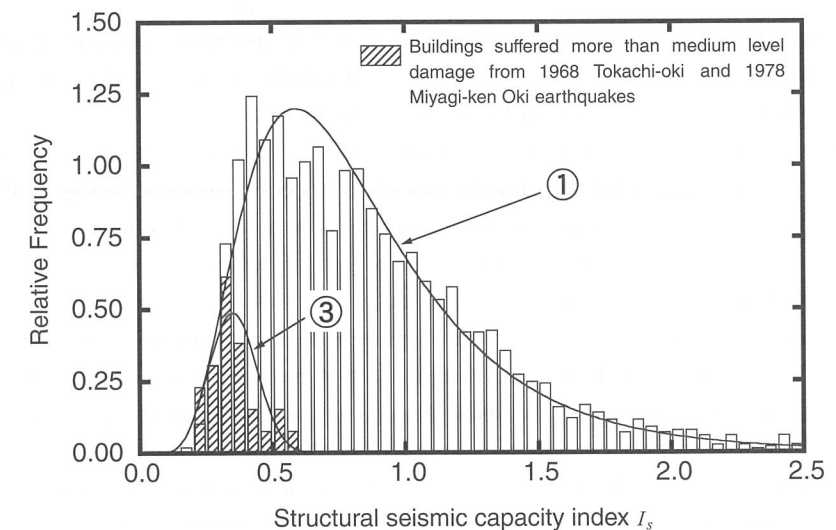


Fig. 9: Distribution of structural seismic capacity indices of existing and damaged buildings¹¹⁾

Some school buildings suffered moderate to severe damage in the 1995 Hyogo-ken Nanbu earthquake when their structural seismic capacity index I_s was 0.4 to 0.6. If index I_s was larger than 0.6, very few school buildings suffered severe damage, and most buildings survived with less than moderate damage.

Required seismic capacity index I_{so} is the product of seismic index E_s , seismicity index Z , soil index G and importance index U as shown in Eq. 17. Seismic index E_s for the second-level screening procedure is 0.60, which is smaller than that for the first-level screening procedure.

A structure is generally judged safe if structural capacity index I_s is greater or equal to required seismic capacity index I_{so} . If index I_s is less than index I_{so} but more than 0.65 times index I_{so} , the structure is thought to possess reasonable seismic resistance, but the vulnerability assessment by the third level screening procedure is recommended.

To be consistent with the safety level of buildings designed for ductile performance under the present Building Standard Law, the following minimum resistance must be satisfied to be "safe";

$$1.25 \geq C_T \cdot S_D \geq 0.30 \quad (\text{Eq. 24})$$

where, cumulative strength index C_T is defined by

$$C_T = \frac{n+1}{n+i} (C_1 + \alpha_2 \cdot C_2 + \alpha_3 \cdot C_3) \quad (\text{Eq. 25})$$

where the definition of C 's and α 's are the same as in Eq. 22.

Third-Level Screening Procedure

The third-level screening procedure is to apply if any building is found deficient by the second-level screening procedure. The flexural yielding and shear failures of girders, and the up-lifting of structural walls at their base are considered in the static push-over analysis.

A structure, which can develop flexural yielding at beam ends, dissipates vibration energy through flexural hysteresis, and attains a large ductility. The base of walls may be up lifted due to large earthquake overturning moment before flexural yielding takes place in the wall. Such structural behavior is considered in the third-level screening procedure by recognizing three additional types of members in addition to the five types used in the second-level procedure. They are (f) flexural beam columns (columns connected to flexurally yielding beams), (g) shear beam columns (columns connected to shear failure beams), and (h) up lifting walls. Ductility index of the three types of members are chosen to be 3.0 for flexural-beam columns, 1.5 for shear-beam column, and 3.0 for up-lifting walls (Table 2).

Table 2 Ductility index F of members and failure mode (third-level screening)

Failure mode of columns and walls	Value
Highly ductile columns without fear of shear failure	3.2
Columns connected to girders yielding in flexure	3.0
Ductile columns unlikely to fail in shear	2.2
Columns connected to girders likely to fail in shear	1.5
Less ductile columns, but unlikely to fail in shear	1.3
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
Highly ductile structural walls without fear of shear failure	2.0
Structural walls likely to fail in shear	1.0

Flexural and shear strengths of all members must be calculated to identify their failure modes. A realistic identification of damage (flexural yielding and shear failure) of members and story shear capacity are estimated at the ultimate limit state of the structure, obtained by a limit analysis method or by a nonlinear static analysis (push-over analysis) under monotonically increasing load. A value of

ductility index is selected for each column and wall depending on the failure mode of the own member or connecting girders. For representative member groups, story shear coefficients are evaluated from the shear carried by the vertical members at the formation of lateral collapse mechanism. Structural index E_o is evaluated by Eqs. 22 and 23.

5. ASSESSMENT METHOD CONSISTENT WITH BUILDING CODE

The law⁴⁾ for the promotion of the seismic strengthening of existing buildings refers to the notification of Ministry of Construction on the seismic vulnerability assessment and strengthening. The procedure is based on the second-level screening procedure of the seismic vulnerability assessment standard¹⁾, but was modified to be consistent with the provisions of the Building Standard Law. The method is intended for more general application than the screening procedures.

Structural Seismic Capacity Index

Structural seismic capacity index I_s of a story is calculated by the following expression;

$$I_s = \frac{E_o}{F_{es} Z R_t} \quad (\text{Eq. 26})$$

where, E_o : seismic index of a story, Z : seismic zone factor which varies from 0.7 to 1.0 for four seismic zones in Japan, R_t : vibration characteristic factor, and F_{es} : structural configuration factor, representing the distribution of stiffness and mass in a story. Seismic zone factor Z , vibration characteristic factor R_t , and structural configuration factor F_{es} are the same as those prescribed in the Building Standard Law.

Vibration Characteristic Factor

Vibration characteristic factor R_t (Fig. 10) represents the shape of design earthquake spectrum for three types of soil and is defined by Eq. 27:

$$\begin{aligned}
 R_t &= 1.0 & \text{for } T < T_c \\
 R_t &= 1.0 - 0.2 \left\{ \frac{T}{T_c} - 1 \right\}^2 & \text{for } T_c \leq T < 2T_c \\
 R_t &= 1.6 \frac{T_c}{T} & \text{for } 2T_c \leq T
 \end{aligned} \quad (\text{Eq. 27})$$

where, T_c : dominant period of subsoil (= 0.4 sec for stiff sand or gravel soil, = 0.6 sec for other soil, and = 0.8 sec for alluvium mainly consisting of organic or other soft soil); T : natural period of a building. The natural period T (sec) may be estimated by Eq. 28:

$$T_c = 0.02 H \quad (\text{Eq. 28})$$

where, H : total height of a reinforced concrete building in m.

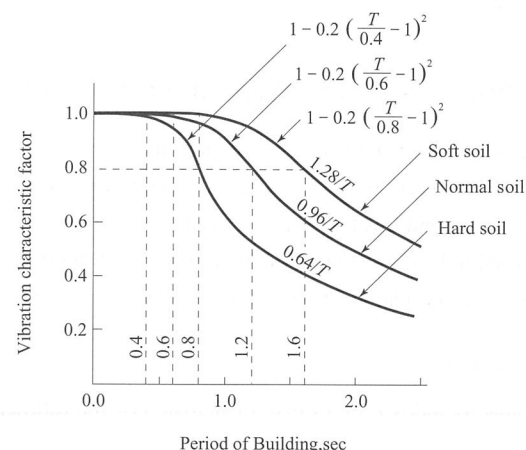


Fig. 10: Vibration characteristic factor R_t

Structural Configuration Factor

Structural configuration factor F_{es} considers the amplification of required story resistance due to an irregular distribution of stiffness along the height of a structure and also due to a large eccentricity of mass center with respect to the center of rigidity in a floor plan. The structural configuration factor 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, as given in Eq. 29:

$$F_{es} = F_s F_e \quad (\text{Eq. 29})$$

The regularity in stiffness distribution along structural height is judged by the value of rigidity ratio, defined by Eq. 30 at each story:

$$R_s = \frac{r_s}{\bar{r}_s} \quad (\text{Eq. 30})$$

in which, r_s : reciprocal of drift angle at a story under design earthquake forces corresponding to $C_o = 0.20$, \bar{r}_s : average value of r_s '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. 11.a).

Eccentricity ratio R_e is defined by Eq. 31 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;

$$R_e = \frac{e}{r_e} \quad (\text{Eq. 31})$$

Mass center 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 corresponding to $C_o = 0.2$. Elastic radius r_{ex} in the x-direction in plan is defined as the square root of the ratio of the torsional resistance to the sum of lateral resistance (Eq. 32);

$$r_{ex} = \sqrt{\frac{\sum K_x \bar{y}^2 + \sum K_y \bar{x}^2}{\sum K_x}} \quad (\text{Eq. 32})$$

where, K_x and K_y : lateral stiffness of a vertical member at distance x and 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. 11.b).

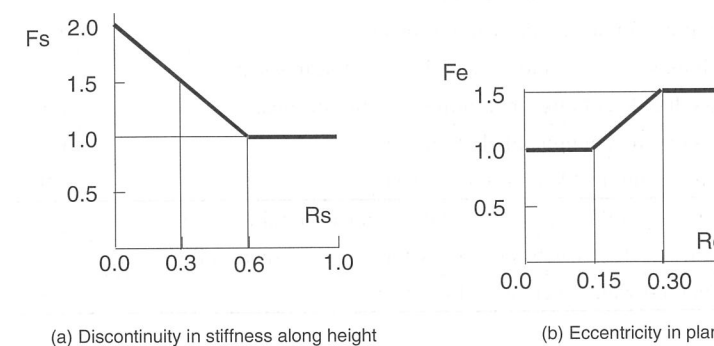


Fig. 11: Amplification of design story shear for irregularity in the 1981 Building Standard Law

Structural Index

Similar to the second- and third-level screening procedures in the Standard for Seismic Vulnerability Assessment of Existing Reinforced Concrete Buildings¹⁾, columns and structural walls of a story must be classified into three groups by their ductility capacity index F corresponding to their failure mode as given in Table 3. Story shear Q_k carried by member group k is calculated. Structural index E_o may be taken as the larger value of Eqs. 33 and 34;

$$E_o = \frac{Q_u F}{W_i A_i} \quad (\text{Eq. 33})$$

$$E_o = \frac{\sqrt{(Q_1 F_1)^2 + (Q_2 F_2)^2 + (Q_3 F_3)^2}}{W_i A_i} \quad (\text{Eq. 34})$$

where, Q_u : maximum story shear carrying capacity, F_j : ductility index (Table 3) of member group j (columns and structural walls) in story i , W_i : total dead and live loads story i supports, A_i : factor representing vertical distribution of a seismic story shear coefficient ($= Q_i/W_i$), given by Eq. 35,

$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i \right) \frac{2 T}{1 + 3 T} \quad (\text{Eq. 35})$$

where Q_i : story shear at story i , $\alpha_i = W_i/W_1$, and W_i : total dead and live loads story i supports, and W_1 : total dead and live loads of the building.

Table 3: Ductility index of members and failure mode (design code procedure)

Failure mode of columns and walls	Value
Highly ductile columns without fear of shear failure	3.5
Columns in highly ductile frame	3.5
Ductile columns unlikely to fail in shear	2.4
Columns connected to girders likely to fail in shear	2.0
Less ductile columns, but unlikely to fail in shear	1.3
Less ductile columns likely to fail in shear	1.3
Brittle columns likely to fail in shear	1.0
Structural walls rotating at the base under lateral loading	3.5
Highly ductile structural walls without fear of shear failure	2.5
Structural walls likely to fail in shear	1.3

If there exists a critical load-carrying member in the story, Equation 33 must be evaluated at the failure of the critical load-carrying member.

Comparison with Screening Procedure

Let us compare the present procedure consistent with Building Standard Law and the second-level screening procedure in the seismic vulnerability assessment standard ¹⁾. Calculation of structural index E_o in the two procedures are almost identical if Eqs. 22 and 23 are compared with Eqs. 33 and 34. The present procedure uses a story shear divided by the weight the story carries while the screening procedure uses a story shear coefficient. The definition of story index ϕ_i is different in the two procedures, but both procedures use the ratio of the base shear coefficient to a story shear coefficient. Ductility index F for member groups is different as noticed by comparing Tables 1 and 3.

Structural seismic capacity index I_s appears different if Eq. 16 for the screening procedure is compared with Eq. 24 of the present procedure. Age index T is not considered in the present procedure. Configuration index S_D in the screening procedure indicated the reduction in resistance due to irregularity

while structural configuration factor F_{es} in the present procedure is a penalty factor in design to require higher resistance due to the eccentricity. Therefore, index S_D and factor F_{es} are in reciprocal relation.

Major difference in the two procedures comes from the fact that the screening procedure is intended for buildings less than seven stories; the acceleration response spectral ordinate is almost constant in this short period range. However, the present procedure is intended for general use, and the acceleration response spectrum is considered in the structural seismic capacity index.

Lateral Force Capacity Index

An index q of structural lateral force resisting capacity is defined by Eq. 36;

$$q = \frac{Q_u}{F_{es} W Z R_t A_i S_t} \quad (\text{Eq. 36})$$

where, S_t : minimum base shear coefficient 0.30 required for reinforced concrete construction in the Building Standard Law.

Seismic Vulnerability Assessment

The seismic vulnerability of a story is assessed by structural seismic capacity index I_s and lateral force capacity index q as shown in Table 4. 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.

Table 4 Seismic vulnerability assessment

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

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.

6. SUMMARY

The state of the art in Japan in evaluating the earthquake resistance of existing reinforced concrete buildings is briefly outlined. The method recognizes the strength and ductility of a building. A simple procedure is essential to identify a majority of earthquake resistant buildings. More sophisticated procedure is required to determine the earthquake resistance of ductile buildings. The earthquake

resisting capacity must be compared with an index to characterize the earthquake damaging power. The reliability of the criteria must be examined with respect to the earthquake damage; i.e., the earthquake resistance index of damaged and undamaged buildings must be compared with the observed level of damage.

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