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Seismic capacity of reinforced concrete buildings which suffered 1987 Chibaken-toho-oki earthquake

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ABSTRACT: The seismic capacity of damaged and undamaged reinforced concrete buildings which suffered the 1987 Chibaken-toho-oki earthquake (M=6.7) in Japan was evaluated using the Japanese Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings. A border of the seismic capacity between damaged and undamaged buildings was discussed through the seismic capacity of two adjacent buildings in two sites and though the results of nonlinear response spectra to the accelerograms observed under the earthquake, the maximum acceleration of which was 400 gal. The frequency distribution of the seismic capacity of 463 reinforced concrete buildings in Chiba Prefecture and 2078 buildings in Japan was shown. It implied that the quite number of old reinforced concrete buildings might suffer from medium damage, even in such not so severe earthquake conditions as the earthquake.

1 INTRODUCTION

Seismic design method has prevailed in Japan since the provision of seismic coefficient was adopted in the Urban Building Law in 1924, a year after the 1923 Kanto earthquake which was one of the most destructive earthquake ever experienced. It was succeeded by the Building Standard Law (BSL)in 1950. It has been applied to practically all low and medium buildings (less than 45m in height) and therefore the seismic capacity of buildings in Japan is one of the highest levels in the world. However the 1968 Tokachi-oki earthquake caused damage to reinforced concrete buildings designed according to the BSL for the first time. Several damaging earthquakes have continued.

The lessons for seismic design have been learned from these earthquake, especially from the 1968 Tokachi-oki earthquake. At the same time, however, the seismic safety of existing buildings was doubted.

All the actions, such as the amendment of BSL and the Enforcement Ordinance of BSL (1970,1981,1986), the revision of the "Reinforced Concrete Structure Calculation Standard" of Architectural Institute of Japan (AIJ,1971,1982,1988) and the proposal of several practical methods to evaluated the seismic performance of existing buildings are attributable to the lessons, one of which is to consider the dynamic behavior of buildings under earthquakes.

The Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings (Ref.1) was complied in 1977 under the sponsorship of the Ministry of Construction, Japanese Government and was devised in 1990, on the basis of the experience over ten years. The seismic performance of several thousand of reinforced concrete buildings in Japan has been evaluated by the standard and some of them have been strengthened owing to shortage of the seismic capacity.

In this paper the seismic capacity evaluated by the standard for reinforced concrete buildings which suffered the 1987 Chibaken-toho-oki earthquake are discussed, compared with the degree of damage and the results of response analyses to observed accelerograms.

2 EARTHQUAKE

The basic data of the earthquake are made public as follows;

1.Occurrence Time	11:08(local time),17 December,1987
2.Epicenter	latitude 35 21' longitude 140 29'
3.Focal depth	58km
4.Magnitude	6.7

The earthquake hit Chiba Prefecture adjacent to Tokyo Metropolis. In Chiba Prefecture, 2 persons died and 144 persons were injured. The roofing tiles of about 70,000 wooded houses were damaged.

Accelerograms were observed on the free

surface of 9 sites in Chiba Prefecture. The location of each site and the maximum acceleration of the two-component accelerograms are shown in Fig. 1, together with the epicenter. The digitalized data of the accelerograms are offered by the National Research Institute for Earth science and Disaster Prevention, Science and Technology Agency. Two typical response acceleration spectra are shown in Figs. 2 and 3. The former is the spectrum corresponding to relatively hard-soil condition and the latter to relatively soft-soil condition.

3 INVESTIGATED BUILDINGS

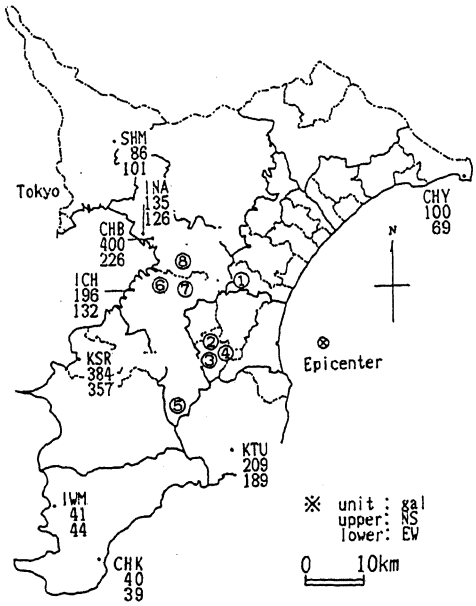
Ten reinforced concrete buildings were investigated after the earthquake. Their locations are shown in Fig. 1, together with the seismic capacity and the degree of damage. In observing the damage caused by the earthquake, the shear failure of columns, especially of short columns at the first story was the most prevalent type of damage, and some of short columns had severe damage.

Building #3 had severe damage on some short columns due to landslide accompanied with liquefaction. Building #6 had severe damage on the non-structural elements, but small damage on the beams and columns (medium damage on the whole). The seismic capacity of these two building was not evaluated.

4 METHODS

4.1 Evaluation of seismic capacity

The overall method of the standard consists



NO.	Building	Story	Is-Indices at 1st floor	Degree of Damage
1-a	School	4	0.36 (0.30)	Small
1-b	School	4	0.38 (0.24)	Medium
2	Office	3	0.56	Medium
3	Office	2		Medium
4	School	3	0.31 (0.22)	Medium
5	School	2	0.32 (0.43)	Medium
6	Office	2		Medium
7-a	School	4	0.46 (0.39)	Small
7-b	School	4	0.40 (0.27)	Medium
8	School	4	0.39	Small

Fig.1 Location of building and station and maxium acceleration

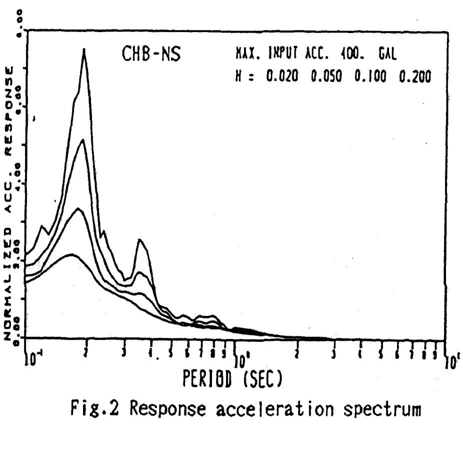


Fig.2 Response acceleration spectrum

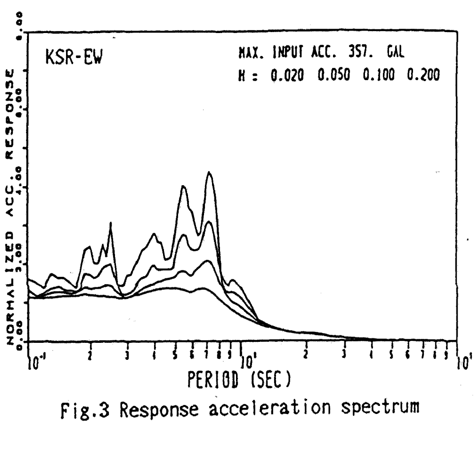


Fig.3 Response acceleration spectrum

of three sequential levels which are classified by the grade of simplification and reliance. The standard evaluates the seismic capacity at each story and in each direction of the building by the following index;

$$Is = Eo * SD * T \quad \ldots\ldots\ldots\ldots\ldots (1)$$

where, Eo = basic structural index calculated by ultimate strength, ductility, number of stories and story level considered

SD = structural design index to modify Eo-index due to the building shape and distribution of stiffness

T = time index to modify Eo-index due to grade of deterioration of strength and ductility

The standard values of the SD- and T-indices are 1.0. The Eo index for the simple structural system can be expressed by the product of the ultimate horizontal strength index in term of story shear coefficient (C), ductility index(F) and story index. The story index at the first floor level is 1.0. Therefore, the Eo index at the first floor level of the simple structure can be defined as;

$$Eo = C * F \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

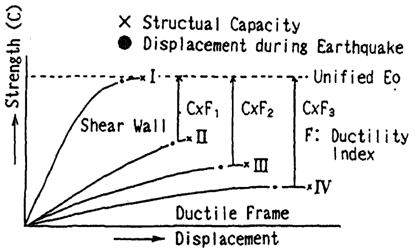


Fig.4 Displacement and strength relationship of idealized model

The ductility index(F) is thought as followings; if peak response of various type of simple structural systems under an earthquake would halt in the same manner just before the structural capacity as shown in Fig. 4, the seismic capacity of these systems must be all the same.

There are various type of structural members in a building. The Special Hazardous Column (SHC) plays a important role on the determination of the Eo-index, and means that the shear failure of the column causes a fatal damage such as falling of the floor, because gravity loads can not transfer from the column to neighboring vertical load-carrying members such as walls and columns.

The strength-displacement relationship for an entire building is shown in Fig. 5. Three Eo-indices are calculated, but the final Eo-index is always the value at the collapse point of short columns in the case where the short columns are SHC, and the value at the collapse point of shear columns in the case where it is larger than that at the collapse point of short columns and the short columns are not SHC by reason that the collapse of short columns is acceptable. The Eo-index at the collapse point of ductile columns is the final one in the case where it is the largest value and the short and shear columns are not SHC.

4.2 Judgment of seismic capacity

Judgment of the seismic capacity of a building according to the standard is carried out by the following equation;

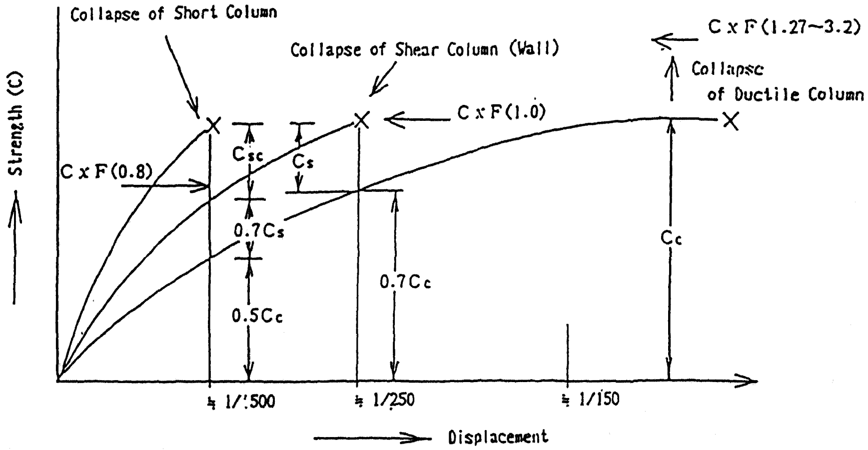


Fig.5 Displacement and strength relationship of entire building

$Is \geq Iso$ and $C * SD \geq 0.3$

$$Iso = Es * Z * G * U \quad ....(3)$$

where, Es: seismic basic-index equal to 0.8 in first level and 0.6 in second and third levels
- Z : zone index to modify Es-index due to seismic activity
- G : Ground index to modify Es-index due to influence of geological and topographical conditions and due to effects of soil-building interaction
- U : Usage index to modify Es-index due to impact by earthquake damage

The standard values of the Z-, G- and U-indices are 1.0.

4.3 Judgment of seismic capacity to observed accelerograms

Nonlinear response analyses to observed accelerograms were carried out and required strength ratio spectra were obtained in the same way as the Reference 2. Two examples of required strength ratio spectra are shown in Figs. 6 and 7. The former is the spectrum corresponding to Degrading Tri-linear Model and the latter to Origin-Oriented Model (Ref. 2). The carves of the spectra become smooth and their peaks sift to the smaller period as their ductility factors increase.

Required strength ratio is a critical strength ratio required to restrict the response displacement of a building within the structural capacity in terms of ductility factor as followings;

$$C = \beta * \ddot{V}g / G = \beta * \alpha \quad ....(4)$$

where
- $\ddot{V}g$: Maximum acceleration of input motion
- G : Acceleration of gravity
- α : seismic coefficient of input motion

The F-index can be calculated by the ductility factor (μ) according to the standard as followings;

$$Es = C * F = F * \beta * \alpha \quad ....(5)$$

where, the F-index is 1.0 for shear failure (1.9 to μ=10 of Origin-Oriented Model), 1.27 for μ=1 of flexural failure (μ=1 Degrading Tri-linear Model), 2.1 for μ=2 and 2.9 for μ=4 (Ref. 3). Since the Z-, G- and U-indices are not necessary in such case, the Iso-index is equal to the Es-index.

5 RESULTS AND DISCUSSION

There are two adjacent buildings in two sites, namely #1-a and #1-b buildings, and #7-a and #7-b buildings. One had severe damage at the first floor and medium damage at the second floor on the short columns (medium damage on the whole), and the other had small damage in both sites. Since their transverse(NS) directions with a large amount of walls have high seismic capacity, their Is-indices of every floor in the longitudinal (EW) directions are shown in the Table 1, which are evaluated by the second level to explain the damage on the columns. The value of the parentheses in the Table 1 is the Is-index at the collapse point of short columns. #1-b and #7-b buildings have the higher Is-index at the collapse point of shot columns than #1-a and #7-a building do, respectively, because the former of both buildings has the larger number of short columns than the latter does.

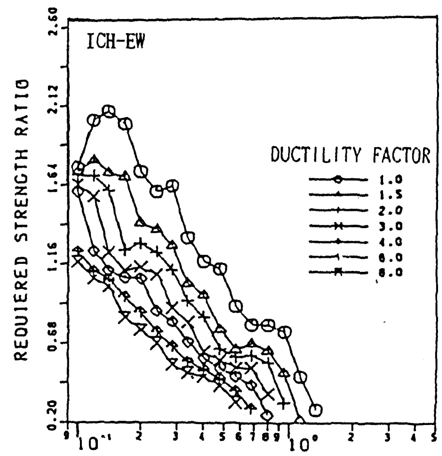


Fig.6 Required strength ratio spectrum (D-Tri)

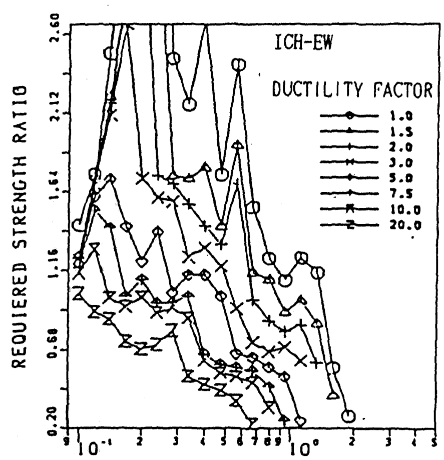


Fig.7 Required strength ratio spectrum (Origin)

Table 1 Is-indices of #1 and #7 buildings

Floor	#1-a	#1-b dmaged	#7-a	#7-b dmaged
4	0.38 (0.48)	0.50 (0.41)	1.26 (0.51)	1.27 (0.43)
3	0.24 (0.34)	0.43 (0.29)	0.75 (0.40)	0.82 (0.22)
2	0.27 (0.33)	0.36 (0.25)	0.55 (0.39)	0.54 (0.25)
1	0.36 (0.30)	0.38 (0.24)	(0.39)	(0.27)

A border between severe damage and small damage on the short columns is judged as the Is-index of 0.25-0.30 which is nearly equal to the values obtained by the required strength ratio spectrum shown in Fig.7 to the ICH accelerograms and the Equation (5).

There were 11 buildings near the CHB station, 13 buildings near the INA station, 5 buildings the KSR station, 11 buildings near the CHY station and no building near other stations, the seismic capacity of which had been evaluated. All buildings had little damage under the earthquake. The Iso-indices of each station were obtained as a function of the natural period of buildings by using such required strength ratio spectra to two components of each accelerogram as Figs. 6 and 7 and the Equation (5). The Is-indices of the buildings near each station were compared with the Iso-indices based on a assumed natural period.

The results showed that the buildings near the INA and CHY stations had no damage and that some buildings near the CHB and KSR stations might have damage. This fact might be explained as followings;

1. The wave forms and amplitude levels of accelerograms may be different form a site to another, even near the site.
2. The response acceleration to the CHB accelerogram in Fig.2 become extremely small with increasing period.
3. The amplitude level of input acceleration at the basement of buildings may become smaller than that of free-surface acceleration and also the response acceleration to stiff buildings on soft soil such as the buildings near KSR station may become small due to soil-building interaction effects.
4. Ultimate shear strength may be estimated low due to the equation of the standard.

The relationship between the identification of damage and the Is-index of reinforced concrete buildings which suffered the 1968 Tokachi-oki, the 1978 Miyagiken-oki, the 1978 Izu-ohshima and the 1987 Chibaken-toho-oki earthquakes in Japan are shown in Fig. 8, where new data are added to the figure in the Reference 4. The lower Is-index connected by a broken line shows the Is-index at the collapse point of short col-

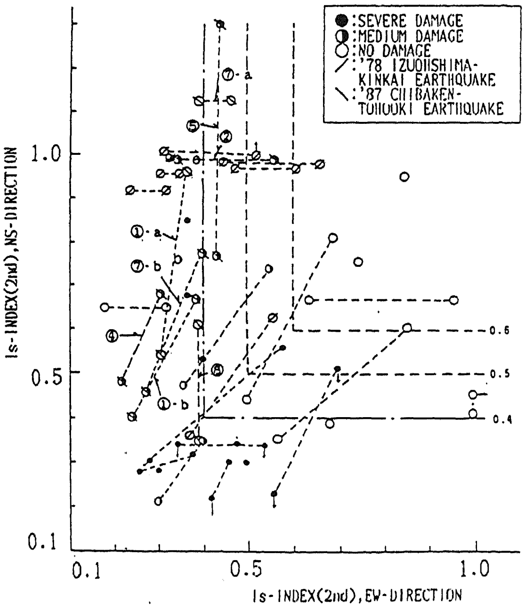


Fig.8 Is-index in second level VS earthquake damage in Japan (added to Ref.4)

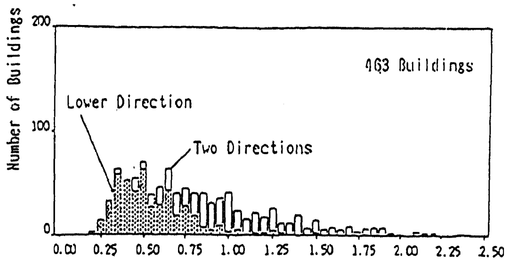


Fig.9 Distribution of Is-index in second level

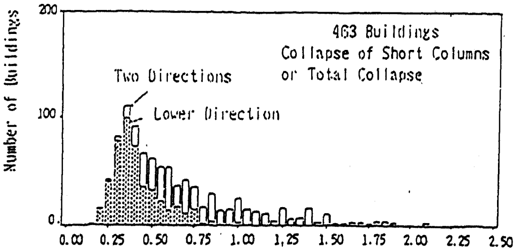


Fig.10 Distribution of Is-index in second level

umns. It is suggested that the Is-index of 0.5-0.6 is a border between damaged and undamaged buildings in such severe earthquake condition as the 1968 Tokachi-oki earthquake and the 1978 Miyagiken-oki earthquake and the value of 0.25-0.3 at the collapse point of the short columns is a border in such a little severe earthquake conditions as the 1978 Izu-ohshima earthquake and the 1987 Chibaken-toho-oki earthquake. The Es-index of 0.6 in the second and third levels of the standard which is shown in the Equation (3) was determined considering above-mentioned results.

The frequency distribution of the Is-index at the first floor in the second level procedure for 463 public buildings in Chiba Prefecture is shown in Figs. 9 and 10. The former is the distribution of the final Is-index and the latter is that at the collapse point of short columns.

The frequency distribution of the Is-index at the first floor in the second level procedure for 2078 buildings constructed until 1971 and from 1972 is shown in Fig. 11 and Fig. 12, respectively (Ref. 5). The former includes the buildings with the lower Is-index than the latter does. One of the reasons is that a large amount of shear reinforcement has been placed in structural members, especially in columns since the revision of the BSL in 1970 and of the AIJ in 1971.

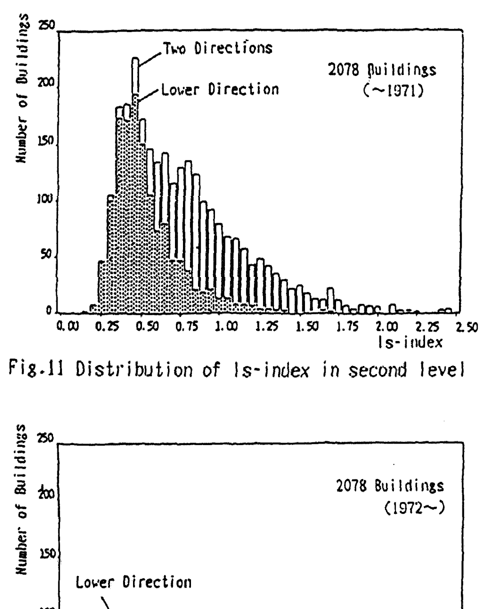


Fig.11 Distribution of Is-index in second level

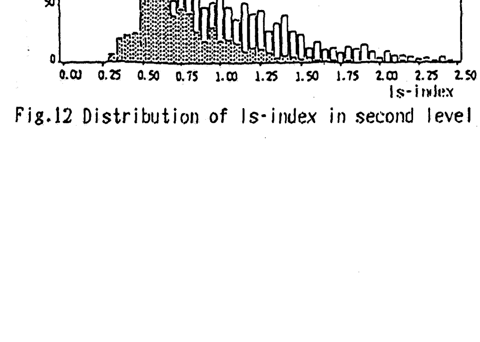


Fig.12 Distribution of Is-index in second level

The distribution shown in Figs. 9, 10, 11 and 12 implies that quite number of old reinforced concrete buildings may have medium damage even such a little severer earthquake conditions as the earthquake and severe damage in severer earthquake conditions than the earthquake.

6 CONCLUSIONS

The Is-index and Iso-index are not deterministic and are probabilistic because there are many uncertainties due to input motion, ultimate strength and ductility, earthquake response and so on, but the following conclusive remarks may be expressed.

The Is-index of 0.25-0.30 is a border between damaged are undamaged buildings under the earthquake, although the value of 0.5-0.6 is a border under past severe earthquakes in Japan.

The frequency distribution of the Is-index implies that quite number of old reinforced concrete buildings may have medium damage even in such not so severe earthquakes as the earthquake.

The data and methods shown herein serve as basic ones for earthquake countermeasures, especially for damage assessment of reinforced concrete buildings as well as the seismic design.

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