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# ESTIMATION METHOD OF SEISMIC CAPACITY INDEX OF EXISTING HIGH-RISE RC BUILDINGS IN JAPAN

T. Akita<sup>1</sup> and N. Izumi<sup>2</sup>

## ABSTRACT

In this paper, equations are proposed in order to estimate the seismic capacity index of the ultimate limit state for existing high-rise RC buildings in Japan. The estimation equations are constructed from the designed base shear coefficient and the first natural period. The designed base shear coefficient and the first natural period are obtained easily because those data are released in the performance evaluation sheet. Three estimation equations are proposed according to the designed years. The estimation equations can be express as follows.

First period (designed from 1971 to 1989) :  $HIS = 4.60(C_B \times T_1) - 0.132C_B + 0.061T_1 + 0.364$

Second period (designed from 1990 to 1999):  $HIS = 5.78(C_B \times T_1) - 0.003C_B + 0.054T_1 + 0.170$

Third period (designed from 2000 to 2009) :  $HIS = 5.77(C_B \times T_1) - 1.966C_B - 0.004T_1 + 0.591$

Where,  $HIS$  indicates the seismic capacity index,  $C_B$  indicates the designed base shear coefficient and  $T_1$  indicates the first natural period, respectively. The seismic capacity indexes of 373 existing high-rise RC buildings in Japan are calculated by the estimation equations and the frequency distributions of the seismic capacity index of the buildings are obtained. As a results, following conclusion are obtained.

- (1) The frequency distributions of the seismic capacity index of ultimate state for existing high-rise RC buildings in Japan exhibit a mountain shape which has single peak. The peak shifts slightly toward the small side of the seismic capacity index.
- (2) The average and the median of the frequency distribution of the third period are relatively large compared with those of the other period.
- (3) Considering the variation of the estimation equations, the percentages of the building which has the seismic capacity index value under 1.0 are from 0% to 20%.

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In this paper, equations are proposed in order to estimate the seismic capacity index of the ultimate limit state for existing high-rise RC buildings in Japan. The estimation equations are constructed from the designed base shear coefficient and the first natural period. The designed base shear coefficient and the first natural period are obtained easily because those data are released in the performance evaluation sheet. Three estimation equations are proposed according to the designed years. The seismic capacity indexes of 373 existing high-rise RC buildings in Japan are calculated by the estimation equations and the frequency distributions of the seismic capacity index of the buildings are obtained. Then, the distribution shapes of the frequency distributions and the variation of the estimation equations are examined.

### Introduction

So far, more than 500 high-rise RC buildings were built in Japan. However, the structural characteristics of these existing high-rise RC buildings differ depending on the designed years. In addition, seismic capacity of the existing high-rise RC buildings is not grasped. It is necessary to grasp the seismic capacity of the existing high-rise RC buildings not only to understand the present situation but also to enhance the earthquake resistance capacity of high-rise RC buildings. The authors have been studying a method of evaluating the seismic capacity index (HIs value) of high-rise RC buildings. First, a database of existing high-rise RC buildings was created and the distribution of the structural characteristics (designed base shear coefficient  $C_B$ , first natural period  $T_1$ , etc.) of existing high-rise RC buildings was analyzed [1]. Based on the analysis results, frame models that simulating existing high-rise RC buildings were created [2]. And then, the method of calculating the seismic capacity index of high-rise RC buildings were proposed and the seismic capacity index of the frame models were calculated by the proposed method [3]. As a result, it became possible to express the seismic capacity of high-rise RC buildings by HIs value. However, in the proposed calculation method, it is necessary to conduct the pushover analysis

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and the time history response analysis when calculating the  $H_I$ s value. Therefore, it is practically impossible to calculate  $H_I$ s value of the existing high-rise RC buildings.

In this paper, equations are proposed in order to estimate the seismic capacity index of the ultimate limit state for existing high-rise RC buildings in Japan. The estimation equations are constructed from the designed base shear coefficient and the first natural period. The designed base shear coefficient and the first natural period are obtained easily because those data are released in the performance evaluation sheet. The seismic capacity indexes of 373 existing high-rise RC buildings in Japan are calculated by the estimation equations and the frequency distributions of the seismic capacity index of the buildings are obtained. Then, the distribution shapes of the frequency distributions and the variation of the estimation equations are examined.

### Calculation Method of Seismic Capacity Index

This chapter describes the calculation method of seismic capacity index ( $H_I$ s value) for existing high-rise RC buildings. For details of the calculation method of  $H_I$ s value, refer to the existing research [3]. This calculation method is based on the method of calculating seismic capacity index for RC buildings of 60m or less proposed by Architectural Institute of Japan in 2004 [4]. Figure 1 presents the evaluation diagram of the seismic capacity index. The basic procedure of the calculation method is as follows. (1) Calculate the limit story drift angle based on the ductility factor of the beam obtained by the pushover analysis. (2) Conducted the time history response analysis to determine the limit earthquake motion. (3) Calculate the  $H_I$ s value by the ratio of the intensity of the limit earthquake motion to the standard earthquake motion. The calculation method is applied to the beam collapse type building, so that the target member of the evaluation is beam.

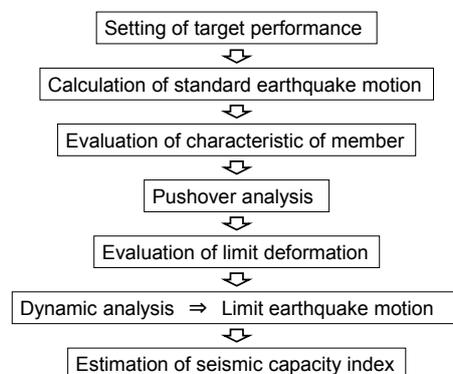


Figure 1. Evaluation diagram

The essentials of this calculation method are shown below. (1) Defining the limit state by the ductility factor of the beam. (2) The limit story drift angle is obtained by the pushover analysis. (3) When the maximum response story drift angle obtained by the time history response analysis reaches the limit story drift angle, the intensity of the earthquake motion is taken as the limit earthquake motion intensity. Figure 2 shows the restoring force characteristics of member and the member deformation corresponding to the limit states (Serviceability limit state, Reparability limit state I, Reparability limit state II and Ultimate limit state). A tri-linear model which has a cracking point and a yielding point is adopted to the restoring force characteristics of member. A ductility factor ( $DF$ ) which calculated by yielding deformation ( $R_y$ ) defines the limit states of

member. It is assumed that a member reaches the serviceability limit state when DF is equal to 1. Similarly, the ductility factors corresponding with the reparability limit state I, reparability limit state II and ultimate limit state are 2, 3 and 4, respectively. In this paper, the  $\mu$  value is calculated by using BCJ-L2 earthquake motion (maximum speed:57cm/sec, maximum acceleration:356cm/sec<sup>2</sup>, duration time:120sec) as the standard earthquake motion [5].

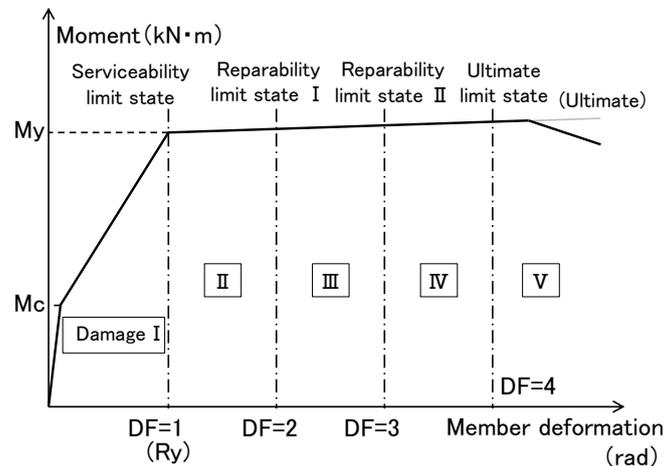


Figure 2. Restoring force characteristics and limit states

### Relationship between $\mu$ Value and Structural Characteristics of Frame Model

In this Chapter, Outline of frame model of existing high-rise RC building is described, and the relationship between  $\mu$  value and structural characteristics of frame model is indicated.

#### Outline of Frame Model

Frame models are constructed based on structural planning and structural characteristics of the existing high-rise RC buildings. The structural planning and the structural characteristics were obtained from existing research conducted by the authors [1]. 555 high-rise RC buildings designed from 1971 to 2009 were collected from the performance evaluation sheet [6] and classified into the three design phases by means of the development of structural techniques on high-rise RC buildings. The first period is from 1971 to 1989, the second period is from 1990 to 1999, the third period is from 2000 to 2009. Table 1 summarizes the specifications of the frame models. Three frame models are constructed in each design periods, thus nine frame models are constructed. These nine frame models are called “Standard model”. For details of the standard model, refer to the existing research [2].

In this paper, “Strong model”, “Weak model”, “High-stiffness model” and “Low-stiffness model” are constructed based on the standard model. The strong model and weak model have 1.15 times capacity and 0.85 times capacity compared with the standard model, respectively. The high-stiffness model has 1.2 times stiffness of beams and 0.8 times weight compared with the standard model. The low-stiffness model has 0.8 times stiffness of beams and 1.2 times weight compared with the standard model. Therefore, the frame model has 45 buildings, including 9 standard

models, 18 strong and weak models, 18 high-stiffness and low stiffness models. All of the frame models are beam yield type.

Table 1. Specifications of framed model (Standard model)

Design pahse	1st Phase						2nd Phase						3rd Phase					
	1G20		1G25		1G30		2G20		2G30		2G40		3G20		3G30		3G40	
Model	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y
Direction																		
Height (m)	60.75		75.5		90.25		61.7		91.7		121.7		63.6		94.6		125.6	
Building stories	20		25		30		20		30		40		20		30		40	
Typical story height (m)	2.95		2.95		2.95		3		3		3		3.1		3.1		3.1	
Typical floor area (m <sup>2</sup> )	675		787.5		945		600		900		1050		585		936		1170	
Typical floor area supported by a column (m <sup>2</sup> )	22.5		22.5		22.5		30.0		30.0		30.0		39.0		39.0		39.0	
Span length (m)	4.5	5	4.5	5	4.5	5	5	6	5	6	5	6	6	6.5	6	6.5	6	6.5
Number of spans	6	5	7	5	7	6	5	4	6	5	7	5	5	3	6	4	6	5
Aspect ratio	2.25	2.43	2.40	3.02	2.87	3.01	2.47	2.57	3.06	3.06	3.48	4.06	2.12	3.26	2.63	3.64	3.49	3.86
Design compressive strength of concrete [Fc] (N/mm <sup>2</sup> ) <sup>※1</sup>	36		36		42		36		48		60		42		54		70	
Tensile yield strength of longitudinal bar (N/mm <sup>2</sup> ) <sup>※2</sup>	390		390		390		390		490		490		490		490		490	
Average weight (kN/m <sup>2</sup> ) <sup>※3</sup>	14.5[11.2]		14.3[11.3]		14.8[11.9]		15.5[11.8]		14.9[11.9]		14.4[11.7]		15.4[11.6]		14.3[11.4]		13.4[10.9]	
Natural period [T1] (sec)	1.11	1.12	1.36	1.36	1.65	1.66	1.17	1.17	1.69	1.71	2.27	2.35	1.27	1.28	1.79	1.92	2.34	2.40
Base shear coefficient [C <sub>B</sub> ]	0.163		0.130		0.113		0.145		0.105		0.074		0.134		0.090		0.068	

※1: The maximum value of design compressive strength of used concrete.

※2: The maximum value of tensile yield strength of used longitudinal bars.

※3: The value calculated from typical floor weight divided by typical floor area which excluded balcony. (The value inside [ ] is including balcony.)

### Relationship between H<sub>I</sub>s value and structural characteristics

The H<sub>I</sub>s values on the ultimate limit in the X direction of the frame models were calculated. Figure 3 shows the relationship between the calculated H<sub>I</sub>s value and the design periods. It is found that the H<sub>I</sub>s value is somewhat larger in the third period compared to the first and second periods because the use frequency of deformed bar of SD 490 is high.

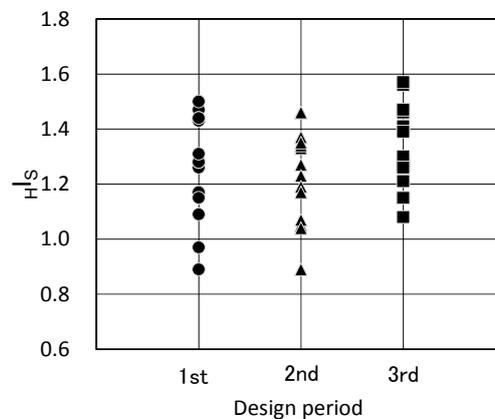


Figure 3. Relationship between H<sub>I</sub>s value and design period

Figure 4 shows the relationship between the calculated  $H_{IS}$  value and the  $C_B \times T_1$ . There is a positive correlation between the  $H_{IS}$  value and the  $C_B \times T_1$ . The  $H_{IS}$  value tends to increase as the  $C_B \times T_1$  increases, but it is not clear enough to estimate the  $H_{IS}$  value from the  $C_B \times T_1$ .

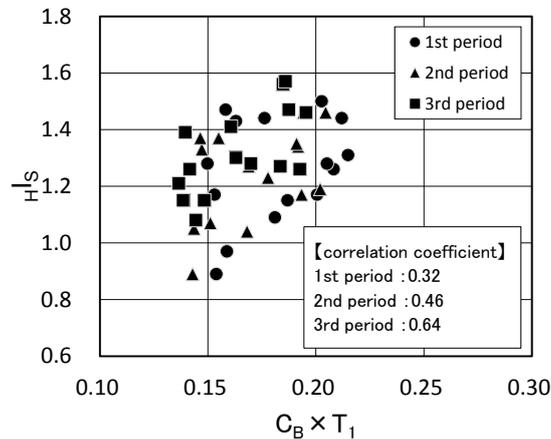


Figure 4. Relationship between  $H_{IS}$  value and  $C_B \times T_1$

Figure 5 shows the relationship between the calculated  $H_{IS}$  value and the  $C_U \times T_e$ .  $C_U$  is the base shear coefficient when the response of the story drift of the frame model reaches the limit story drift angle,  $T_e$  is the equivalent natural period obtained from the equivalent stiffness at that time. A stronger positive correlation is found in the relationship between the  $H_{IS}$  value and  $C_U \times T_e$  than the relationship between the  $H_{IS}$  value and  $C_B \times T_1$  in figure 4. As the  $C_U \times T_e$  increases, the tendency of the  $H_{IS}$  value becoming larger is clear. It is presumed that the  $H_{IS}$  value can be estimated from  $C_U \times T_e$ .

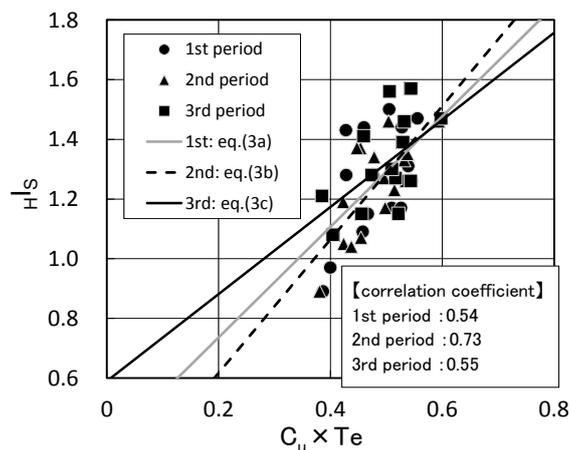


Figure 5. Relationship between  $H_{IS}$  value and  $C_U \times T_e$ .

Figure 6 shows the relationship between  $T_1$  and  $T_e$ , and Figure 7 shows the relationship between  $C_B$  and  $C_U$ . Since the relationship between  $T_1$  and  $T_e$ , and the relationship between  $C_B$  and  $C_U$  both have a strong correlation, it is found that  $T_e$  can be estimated from  $T_1$ , and  $C_U$  can be estimated from  $C_B$ .

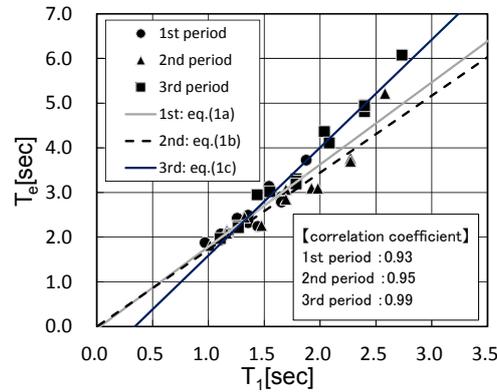


Figure 6. Relationship between  $T_1$  and  $T_e$

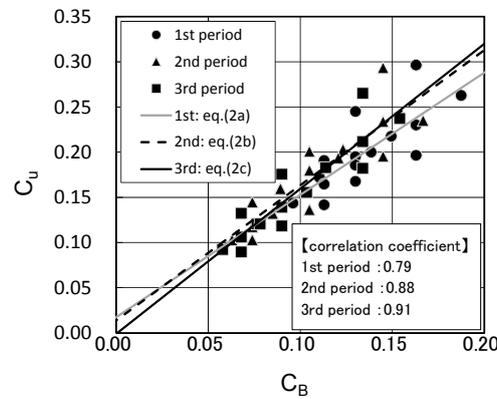


Figure 7. Relationship between  $C_B$  and  $C_U$

### Estimation Method of $H_I S$ Value

From the analysis result of the previous chapter, the estimation equations of  $H_I S$  value using  $C_B$  and  $T_1$  are derived. Equations (1a), (1b), (1c) representing the relationship between  $T_1$  and  $T_e$  are obtained for each design period by the linear approximation of the relationship between  $T_1$  and  $T_e$  shown in figure 6.

$$\text{First period: } T_e = 1.84T_1 - 0.053 \quad (1a)$$

$$\text{Second period: } T_e = 1.72T_1 - 0.001 \quad (1b)$$

$$\text{Third period: } T_e = 2.41T_1 - 0.821 \quad (1c)$$

Where,  $T_1$ (sec) indicates the first natural period and  $T_e$ (sec) indicates the equivalent natural period. The value of  $T_1$  is assumed to be about 1 to 4 seconds. Equations (2a), (2b), (2c) representing the relationship between  $C_B$  and  $C_U$  are obtained for each design period by the linear approximation of the relationship between  $C_B$  and  $C_U$  shown in figure 7.

$$\text{First period: } C_U = 1.35C_B + 0.018 \quad (2a)$$

$$\text{Second period: } C_U = 1.50C_B + 0.014 \quad (2b)$$

$$\text{Third period: } C_U = 1.64C_B - 0.001 \quad (2c)$$

Where,  $C_B$  indicates the designed base shear coefficient and  $C_U$  indicates the base shear coefficient when the response of the story drift angle reaches the limit story drift angle. Equations (3a), (3b), (3c) representing the relationship between the  $H_{Is}$  value and  $C_U \times T_e$  are obtained for each design period by the linear approximation of the relationship between the  $H_{Is}$  value and  $C_U \times T_e$  shown in figure 5.

$$\text{First period: } H_{Is} = 1.85(C_U \times T_e) + 0.366 \quad (3a)$$

$$\text{Second period: } H_{Is} = 2.24(C_U \times T_e) + 0.170 \quad (3b)$$

$$\text{Third period: } H_{Is} = 1.46(C_U \times T_e) + 0.590 \quad (3c)$$

Estimation equations (4a), (4b), (4c) of  $H_{Is}$  value are obtained by substituting equations (1a), (1b), (1c) and equations (2a), (2b), (2c) into  $T_e$  and  $C_U$  of equations (3a), (3b), (3c), respectively.

$$\text{First period: } H_{Is} = 4.60(C_B \times T_1) - 0.132C_B + 0.061T_1 + 0.364 \quad (4a)$$

$$\text{Second period: } H_{Is} = 5.78(C_B \times T_1) - 0.003C_B + 0.054T_1 + 0.170 \quad (4b)$$

$$\text{Third period: } H_{Is} = 5.77(C_B \times T_1) - 1.966C_B - 0.004T_1 + 0.591 \quad (4c)$$

By using the equations (4a), (4b), (4c), it is possible to estimate the seismic capacity index value ( $H_{Is}$  value) of existing high-rise RC buildings only from the designed base shear coefficient  $C_B$  and the first natural period  $T_1$ . As shown in Figure 5, the values calculated by equations (4a), (4b), (4c) are the average of the relationship between the  $H_{Is}$  value and  $C_B \times T_1$ . Therefore, in order to take into account the variation in the frequency distribution of the  $H_{Is}$  values, the intercepts of Equations (3a), (3b), (3c) are increased or decreased to cover the upper and lower limits of the distribution for each period as shown in Figure 8.

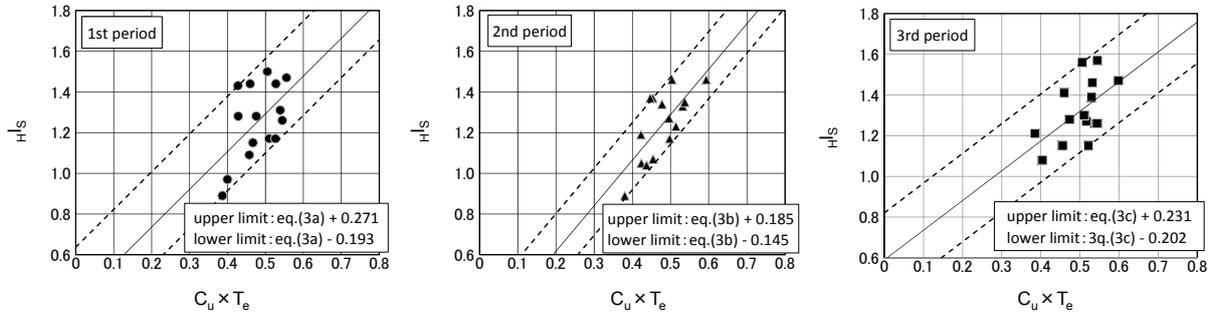


Figure 8. Consideration of variation in estimation of  $H_{Is}$  value

### Frequency Distribution Estimation of $H_{Is}$ Value

The frequency distributions of the  $H_{Is}$  value on the ultimate limit state of 373 existing high-rise RC buildings were estimated using equations (4a), (4b), (4c). Figure 9 shows the frequency distributions of the estimated  $H_{Is}$  value of the ultimate limit state and Table 2 shows the representative value (average and median) of the frequency distributions of the  $H_{Is}$  value. Figure 10 shows the cumulative distribution of the  $H_{Is}$  values for each design period. Since the number of buildings in the first period is smaller than that in the second and third period, the  $H_{Is}$  value of the first period is a reference. As shown in figure 9, it can be seen that the frequency distribution

of the  $H^I_s$  value of each period has a mountain shape which has single peak. As shown in table 2, since the average is slightly larger than the median, it can be judged that the peak shifts slightly toward the small side of the  $H^I_s$  value. The average and the median of the frequency distribution of the third period are relatively large compared with those of the other period. As shown in figure 10, there are many buildings with large  $H^I_s$  values in the third period compared with other periods.

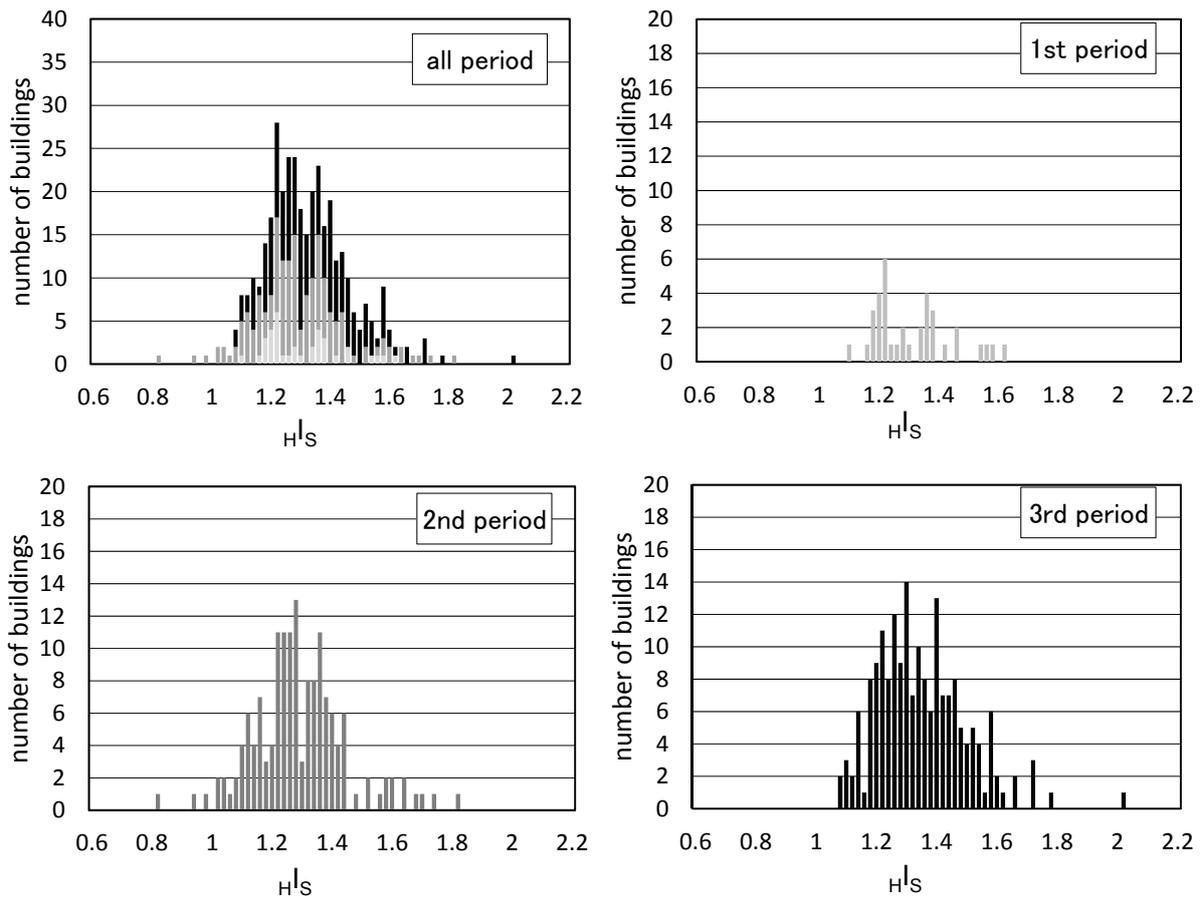


Figure 9. Frequency distributions of estimated  $H^I_s$  value of ultimate limit state

Table 2. Representative value (average and median) of frequency distributions of  $H^I_s$  value

	all pereiod	1st period	2nd period	3rd period
number	373	36	151	186
minmum	0.82	1.10	0.82	1.08
maximum	2.01	1.60	1.82	2.01
average	1.31	1.30	1.28	1.34
median	1.29	1.27	1.27	1.32

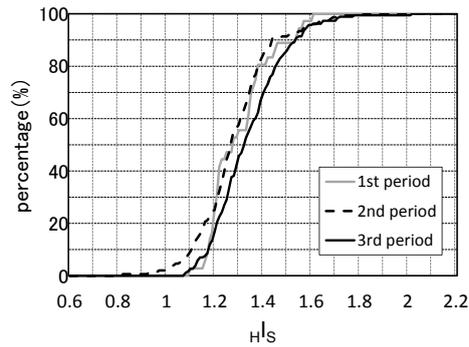


Figure 10. Cumulative distribution of  $H_I$ s value for each design period

Figure 11 shows the cumulative distribution of the  $H_I$ s values of ultimate limit state calculated taking into account the variation in estimation equations. Seismic capacity of high-rise RC buildings for each design period are examined with reference to the  $H_I$ s value of 1.0. The  $H_I$ s value of 1.0 means that the ultimate limit will be reached when inputting the standard earthquake motion (BCJ-L2). From figure 11, the percentages of buildings with the  $H_I$ s value less than 1.0 is 0% to 19.4% in the first period, 0% to 16.6% in the second period and 0% to 16.6% in the third period. In the  $H_I$ s value estimation method presented in this paper, it is found that the percentages of the building which has the  $H_I$ s value under 1.0 are from 0% to 20%.

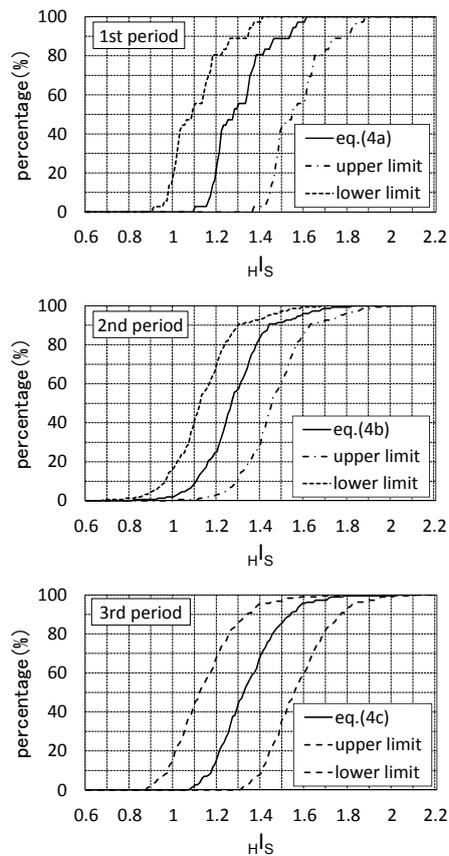


Figure 11. Cumulative distribution of  $H_I$ s value taking into account variation of the equations

## Conclusions

In this paper, equations are proposed in order to estimate the seismic capacity index of the ultimate limit state for existing high-rise RC buildings in Japan. The estimation equations are constructed from the designed base shear coefficient and the first natural period. The designed base shear coefficient and the first natural period are obtained easily because those data are released in the performance evaluation sheet. The seismic capacity indexes of 373 existing high-rise RC buildings in Japan are calculated by the estimation equations and the frequency distributions of the seismic capacity index of the buildings are obtained. Then, the distribution shapes of the frequency distributions and the variation of the estimation equations are examined. As a results, following conclusion are obtained.

- (1) The frequency distributions of the seismic capacity index of ultimate state for existing high-rise RC buildings in Japan exhibit a mountain shape which has single peak. The peak shifts slightly toward the small side of the seismic capacity index.
- (2) The average and the median of the frequency distribution of the third period are relatively large compared with those of the other period.
- (3) Considering the variation of the estimation equations, the percentages of the building which has the seismic capacity index value under 1.0 are from 0% to 20%.

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