Seismic Performance of Moment-Resisting Reinforced Concrete Building Systems during an Intense Earthquake Ground Motion

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ABSTRACT

Seismic performance of low-, medium- and high-rise reinforced concrete buildings are evaluated and compared with one another obtained when subjected to intense earthquake ground motions. A nonlinear dynamic response analysis has been performed, and seismic capacities of building are directly evaluated by the peak ground velocity of ground motion. Four types of building models are generated in accordance with practice in Japan. Critical conditions with which the buildings suffer serious damage and lead to collapse are specified in terms of both interstory and structural deflection responses. Seismic capacities of buildings are examined and discussed in a quantitative manner. The results obtained in the study can be extended utilized to assess the vulnerability of a group of buildings within an urban area.

INTRODUTION

Within this study presented herein, seismic resisting performance of low-, medium- and high-rise buildings is evaluated and compared with one another obtained when subjected to intense earthquake ground motions. Attention is focused on the engineered reinforced concrete buildings that have been structurally designed in accordance with the seismic design code proposed in Japan. Examined is the seismic resisting performance for the moment-resisting reinforced concrete frame buildings. Four types of buildings have been discussed within the study, whose story number is four, 12, 30 and 60, respectively corresponding to the low-, medium- and high-rise reinforced concrete buildings in Japan. Performing inelastic dynamic response analysis, the seismic resisting capacities of those buildings are evaluated in a quantitative manner.

Through the study, within the urban area in Japan within which a large number of buildings with various building height have been constructed, it can be evaluated what types of buildings would be most seriously damaged against intense earthquake ground excitation.

SEISMIC DESIGN PROCEDURE FOR PRACTICE IN JAPAN

Seismic design procedures in the Japanese practices are briefly reviewed. The high-rise buildings of which height is greater than 60 in meter should be verified their seismic capacities by dynamic response analysis. The strength of frames should be obtained so as the responses of the building do not exceed the specified criteria. If the strength of frame is prescribed less than required, the resulting responses obtained against the specified seismic excitation might exceed the specified design criteria, requiring greater frame strength so as to yield the seismic responses falling within the specified criteria. For the low- and medium-rise buildings, height of which is less than 60m, one is not required essentially to check the seismic performance through dynamic response analysis. For the low- and medium-rise buildings, the so-considered minimum frame strength is required as summarized in the followings.

Design Procedures for the Low- and Medium-rise Buildings

In a brief expression, design story shear forces would be determined by the story shear coefficients Ci through

$$C_i = ZR_tA_iC_o \qquad (1)$$

where coefficients Ci, Z, Rt, Ai and Co denote, respectively, as listed below:

- Ci: story shear coefficient at the i-th story level;
- Z: zone coefficient for seismic activity;
- Rt: coefficient taking the soil-building interaction into account, which is given by the design fundamental period of building and that determined from the subsoil condition;
- Ai: coefficient which specified the distribution of story shear force along the height of building; and
- Co: standard shear coefficient specified in the building code.

The required ultimate story shear force Qun is given by

$$Q_{un} = D_s F_{es} Q_{ud} \tag{2}$$

in which Ds: reduction coefficient for the required ultimate story shear force taking the ductility of frame into consideration;

Fes: coefficient taking the irregularity of building into account, increasing the required ultimate story shear force; and

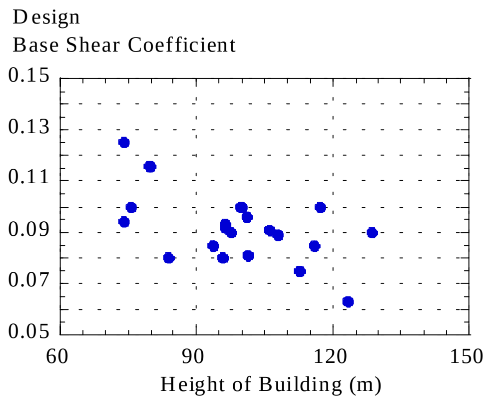
Qud: design lateral force against seismic action, given by substituting 1.0 for Co in Eq. (1).

The coefficient Ds for a moment-resisting frame building can be taken as a less figure provided that a superior ductility of frame should be ensured. In practice, the coefficient Ds is taken as 0.3 for a moment-resisting reinforced concrete frame. In the following analysis, coefficients Z and Fes are taken unity so as to consider the seismic activity is normal, and building shape would be generally regular. And other two coefficients Ai and Rt have been evaluated according to the design code.

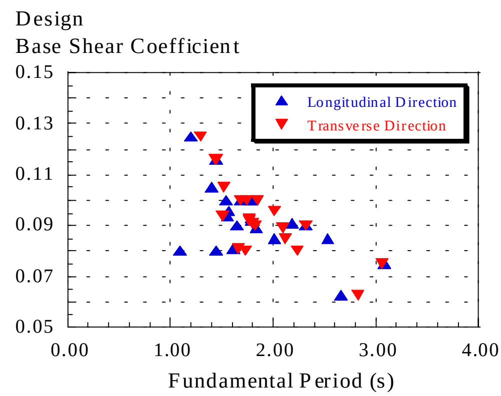
Design Procedures for the High-rise Buildings

The seismic performance of high-rise buildings is essentially examined by dynamic response analysis. Building is designed so as to obtain a certain level of frame strength. If the frame strength is less than required, excessively large responses will be produced generating a large number of hinges within the constituent columns and beams of the building having low frame strength. A certain level of frame strength is necessary for the building to reveal moderate inelastic responses against the specified design seismic action.

Figure 1(a) illustrates the correlation between the building height and design base shear coefficient C_B for 18 high-rise buildings recently designed. The building height is greater than 60 in meter, constructed by reinforced concrete with a moment-resisting frame system exclusively. The axes x and y represent the height of the building and the design base shear coefficient employed in the seismic design, respectively. The buildings examined herein have satisfied the response condition required in the Japanese practice. Figure 1(b) shows the correlation between the fundamental period and design base shear coefficient for those buildings.



(a) Correlation between building height and design base shear coefficient.



(b) Correlation between fundamental period and design base shear coefficient.

Figure 1 Design base shear coefficient for moment-resisting high-rise reinforced concrete buildings in Japan: (a) Correlation with building height; and (b) Correlation with design fundamental period of building.

ANALYTICAL BUILDING MODELS

Dimension of Building and Constituent Members

Four analytical model buildings, of which story number is four, 12, 30 and 60, are produced herein. The buildings are consequently 13.75m, 39.95m, 99.95m, and 198.8m in height, respectively. We will discuss the performance of moment-resisting ductile frame buildings that have been structurally planned with weak-beam and strong-column concept.

The common procedures to establish the model buildings are as follows:

(1) building dimensions such as those of the longitudinal and transverse directions in plan and story height in elevation are determined based on the Japanese practice, resulting in 40m by 40m with five spans of eight meter length in both longitudinal and transverse directions, and about 3.3m for story height;

(2) building weight is roughly determined from the assumption of unit weight of 1.2×10^4 N/m^2 on an average;

(3) column dimensions are determined so as the average compressive stress σ_o to equal about one third of design strength Fc of concrete;

(4) depth of beams and/or girders D is roughly determined as about one tenth of span length, and width of beams and girders B would be determined from examination on design practice in Japan.

The dimensions of model building examined within this study for the four-, 12-, 30- and 60-story buildings are tabulated in Tables 1(a) through 1(d), respectively.

TABLE 1. Dimension of Constituent Members: -Columns and Beams-

(a) Four-story low-rise building

Height of Story	Dimension of Column B and D	Design Concrete Strength	Dimension of Beam B and D
2F ~ 4F 3.25 (m)	1F ~ 4F 0.75 x 0.75 (m)	1F ~ 4F 24 (N/mm^2)	3F and 4F 0.45 x 0.95 (m)
			2F 0.50 x 0.95 (m)
1F 4.00 (m)			1F 0.50 x 1.10 (m)

(b) Twelve-story medium-rise building

Height of Story	Dimension of Column B and D	Design Concrete Strength	Dimension of Beam B and D
6F ~ 12F 3.25 (m)	8F ~ 12F 0.95 x 0.95 (m)	10F ~ 12F 24 (N/mm^2)	10F ~ 12F 0.45 x 0.95 (m)
	1F ~ 7F 1.00 x 1.00 (m)	6F ~ 9F 27 (N/mm^2)	6F ~ 9F 0.50 x 0.95 (m)
2F ~ 5F 3.30 (m)		1F ~ 5F 30 (N/mm^2)	4F and 5F 0.50 x 1.00 (m)
			2F and 3F 0.55 x 1.00 (m)
1F 4.00 (m)			1F 0.65 x 1.10 (m)

(c) Thirty-story high-rise building

Height of Story	Dimension of Column B and D	Design Concrete Strength	Dimension of Beam B and D
17F ~ 30F 3.25 (m)	28F ~ 30F 1.05 x 1.05 (m)	26F ~ 30F 36 (N/mm^2)	27F ~ 30F 0.50 x 0.95 (m)
	16F ~ 27F 1.10 x 1.10 (m)	16F ~ 25F 45 (N/mm^2)	21F ~ 26F 0.55 x 0.95 (m)
			17F ~ 20F 0.60 x 0.95 (m)
8F ~ 16F 3.30 (m)		6F ~ 15F 54 (N/mm^2)	11F ~ 16F 0.65 x 1.00 (m)
	1F ~ 15F 1.15 x 1.15 (m)		8F ~ 10F 0.70 x 1.00 (m)
3F ~ 7F 3.35 (m)		1F ~ 5F 63 (N/mm^2)	3F ~ 7F 0.75 x 1.05 (m)
1F and 2F 4.00 (m)			1F and 2F 0.75 x 1.30 (m)

(d) Sixty-story high-rise building

Height of Story	Dimension of Column B and D	Design Concrete Strength	Dimension of Beam B and D
34F ~ 60F 3.25 (m)	54F ~ 60F 1.10 x 1.10 (m)	56F ~ 60F 36 (N/mm^2)	49F ~ 60F 0.45 x 0.95 (m)
	39F ~ 53F 1.15 x 1.15 (m)	46F ~ 55F 45 (N/mm^2)	44F ~ 48F 0.50 x 0.95 (m)
		36F ~ 45F 54 (N/mm^2)	39F ~ 43F 0.55 x 0.95 (m)
	24F ~ 38F 1.20 x 1.20 (m)		36F ~ 38F 0.60 x0.95 (m)
		26F ~ 35F 63 (N/mm^2)	34F and 35F 0.65 x 0.95 (m)
18F ~ 33F 3.30 (m)		16F ~ 25F 72 (N/mm^2)	26F ~ 33F 0.65 x 1.00 (m)
	1F ~ 23F 1.25 x 1.25 (m)	11F ~ 15F 81 (N/mm^2)	18F ~ 25F 0.70 x 1.00 (m)
3F ~ 17F 3.35 (m)		1F ~ 10F 99 (N/mm^2)	3F ~ 17F 0.80 x 1.05 (m)
1F and 2F 4.00 (m)			1F and 2F 0.75 x 1.30 (m)

Modeling of Constituent Members -Columns and Beams-

Strength of columns and beams is determined from stress analysis on the model building. Geometrical dimensions of columns and beams have been determined in the previous steps. Static stress analysis is carried out against the specified seismic design load specified in the seismic code, evaluating the required flexural strength for each constituent member.

In seismic resisting planning, the buildings are designed to appear a moment-resisting frame with the weak-beam strong-column concept. The flexural strength of beams is established by the evaluated strength from the stress analysis above, yielding the frame strength required necessary in the building code. The flexural strength of columns is determined multiplying the obtained strength from the stress analysis by appropriate coefficients not to produce yielding hinges during dynamic seismic action. Shear strength of each member is taken great so as not to reveal shear failure. Herein the study, strength capacities of members are determined, while the specification and arrangement of reinforcing bars are not designed.

The load-deflection characteristics of members are specified by the tri-linear model, representing stiffness degradation caused by flexural yielding and cracking generated within the member. To establish the tri-linear model, the following four quantities are introduced: (a) yielding strength given from the stress analysis; (b) cracking strength determined as four tenths and one third of yielding strength for columns and beams, respectively; (c) initial stiffness determined from dimensions of member; and (d) stiffness deterioration ratio of cracked stiffness to initial stiffness, which is assumed to be 0.13 for columns and 0.15 for beams, respectively.

Modeling of Low- and Medium-rise Buildings

The coefficient Ds in Eq. (2) for a reinforced concrete moment-resisting frame building can be taken as a less figure provided that a superior ductility of frame should be ensured. In the following analysis herein, the coefficient Ds is taken as 0.3 for a ductile moment-resisting frame. Coefficients Z in Eq. (1) and Fes in Eq. (2) are taken unity so as to consider the seismic activity is normal, and building shape would be generally regular. Other two coefficients Ai and Rt have been evaluated according to the design code.

Coefficient Rt obtained for a building positioned at the site with the normal subsoil condition becomes 1.0 for the four-story low-rise building, design fundamental period of which is 0.28 seconds, and 0.98 for the 12-story medium-rise building, that of which is 0.80 seconds, respectively.

Hereinafter the study, the 12-story medium-rise building model is established as a reference model among the four types of buildings examined within this study.

Modeling of High-rise Buildings

Considering the practices in Japan illustrated in Figs. 1(a) and 1(b), the base shear coefficient C_B, which will determine the strength of frame, is taken as 0.10 for the 30-story high-rise building, and 0.05 for the 60-story high-rise building as well.

Specifying the distribution of story shear forces either from Ai's defined in Eq. (1) or from an appropriate analysis, a stress analysis has been performed to determine the member strength required necessary to ensure the frame strength against the lateral forces produced during the seismic excitation.

ANALYTICAL PROCEDURES EVALUATING THE SEISMIC RESISTING CAPACITY OF BUILDING

Seismic resisting capacity of building is directly evaluated through dynamic response analysis. The criteria with which the building reaches the ultimate condition are prescribed. The intensity of ground motion is evaluated, with which the dynamic inelastic responses of building reach the prescribed criteria. Within the study herein, the intensity of motion is expressed by the peak ground acceleration and/or ground velocity.

Strong Earthquake Ground Motions Utilized in the Analysis

The following five strong earthquake ground motion records are employed in the dynamic response analysis: the El Centro motion obtained during the Imperial Valley earthquake of 1940 (hereinafter abbreviated as ELC), Taft motion obtained during the Kern County earthquake of 1952 (TFT), Hachinohe motion obtained during the Tokachi-oki earthquake of 1968 (HCH), Tohoku University motion obtained during the Miyagi-ken-oki earthquake of 1978 (THU) and the TCU068 motion obtained during the Chi-Chi, Taiwan earthquake of 1999 (TWA). Two horizontal components of each record, the North-South (NS) and East-West (EW) components in general, are utilized in analysis.

The amplitudes of motion are scaled so as the dynamic responses to reach the specified criteria, the peak of which indicate the intensity of motion with which the building is found to reach the ultimate conditions.

SEISMIC PERFORMANCE EVALUATION ON THE BUILDINGS (CONDITION-A)

Criteria for the Buildings to Reach the Ultimate Conditions (Condition-A.1)

A ductile moment-resisting frame building reveals its seismic resisting capacities dissipating a large amount of hysteretic energy against seismic action. Therefore, it may produce fairly large inelastic responses against strong earthquake ground motion excitation.

Herein the study, in the fist assumption, the following conditions are established common among the four types of buildings studied herein, with which the buildings are yielded to reach the ultimate stage: the interstory deflection angle reaches 1/100 at any story level.

Results of Analysis for the Specified Criteria (Condition-A.1)

Table 2 summarizes the intensity of ground motion, with which the responses of building reach the specified criteria. The buildings are considered to be in their ultimate condition when subjected to the ground motion, peak amplitude of which scaled to the listed value in Table 2. For example, the 60-story high-rise building discussed within the study produces the interstory deflection angle of 1/100 when subjected to the El Centro S00E component of which peak velocity is adjusted to become 82.1cm/s while the original velocity is reported as 34.2cm/s. In the other expression, the 60-story building is found to reach the ultimate condition when subjected to the El Centro S00E component modified 2.4 (=82.1/34.2) times as large as the real component.

Figures in the parenthesis ( ) indicate the ratio of intensity of each building to the reference building of the 12-story building, and figures in the right hand side column denote the statistical evaluation of the intensity ratios taken to those of the reference building.

TABLE 2. Intensity of Ground Motion in Velocity for the Buildings to Ultimate Conditions (Condition-A.1)

		Component of Earthquake Strong Ground Motion										Statistical Quantities
		ELC S00E	ELC S90W	TFT N21E	TFT S69E	HCH NS	HCH EW	THU NS	THU EW	TWA NS	TWA EW	μ (ν)
Peak Velocity of Real Motion		34.2	37.1	15.5	17.7	34.9	38.9	36.5	27.8	208.0	201.9	
4-story Building	R = 1/100	32.8 (0.48)	57.8 (0.76)	47.2 (0.76)	44.6 0.93)	52.7 (0.65)	53.6 (0.81)	33.5 (0.86)	30.8 (0.53)	192.8 (1.42)	101.8 (0.78)	0.797 (0.324)
12-story Building	R = 1/100	67.7 (1.0)	76.5 (1.0)	62.2 (1.0)	48.0 (1.0)	81.7 (1.0)	65.8 (1.0)	39.1 (1.0)	57.8 (1.0)	136.1 (1.0)	131.2 (1.0)	1.00 (-)
30-story Building	R = 1/100	59.9 (0.88)	65.5 (0.86)	60.2 (0.97)	79.5 1.66)	55.8 (0.68)	51.2 0.78)	34.4 (0.88)	46.6 (0.81)	147.7 (1.09)	92.5 (0.70)	0.930 (0.303)
60-story Building	R = 1/100	82.1 (1.21)	50.8 (0.66)	69.8 (1.12)	78.8 (1.64)	66.8 (0.82)	62.7 (0.95)	45.9 (1.17)	51.4 (0.89)	59.7 (0.44)	77.3 (0.59)	0.950 (0.372)

unit: cm/s
Nomenclatures μ and ν designate the mean and coefficient of variation of the statistical quantities, respectively.

Criteria for the Buildings to Reach the Ultimate Conditions (Condition-A.2)

The second conditions in the following are established: interstory deflection angle of 1/50 for the 4-story low-rise and 12-story medium-rise buildings, while that of 1/80 for the 30-and 60-story high-rise buildings. Since the axial column stresses in the low- and medium-rise buildings are generally less than those in the high-rise buildings, greater deflection responses can be produced.

Results of Analysis for the Specified Criteria (Condition-A.2)

Table 3 tabulates the evaluated ground motion intensity. Figures 2 and 3 show the responses of interstory deflection angle and inelastic responses of beam ends expressed in term of ductility factor for moment-rotational angle responses.

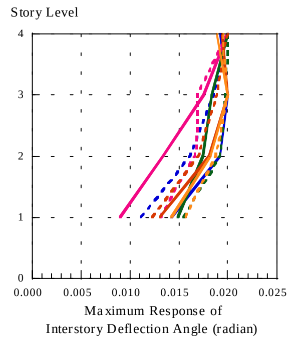
TABLE 3. Intensity of Ground Motion in Velocity for the Buildings to Ultimate Conditions (Condition-A.2)

		Component of Earthquake Strong Ground Motion										Statistical Quantities
		ELC S00E	ELC S90W	TFT N21E	TFT S69E	HCH NS	HCH EW	THU NS	THU EW	TWA NS	TWA EW	μ (ν)
Peak Velocity of Real Motion		34.2	37.1	15.5	17.7	34.9	38.9	36.5	27.8	208.0	201.9	
4-story Building	R = 1/50	82.2 (0.79)	103.9 (0.79)	58.2 (0.67)	72.5 (0.80)	79.9 (0.56)	72.3 (0.66)	48.3 (0.72)	61.1 (0.70)	268.9 (1.56)	173.1 (0.87)	0.812 (0.341)
12-story Building	R = 1/50	103.8 (1.0)	131.5 (1.0)	87.3 (1.0)	91.0 (1.0)	142.2 (1.0)	109.7 (1.0)	66.7 (1.0)	87.8 (1.0)	172.6 (1.0)	198.2 (1.0)	1.00 (-)
30-story Building	R = 1/80	87.6 (0.84)	71.2 (0.54)	68.8 (0.79)	91.9 (1.01)	60.0 (0.42)	108.9 (0.99)	42.7 (0.64)	76.5 (0.87)	158.6 (0.92)	101.6 (0.51)	0.754 (0.280)
60-story Building	R = 1/80	128.6 (1.24)	58.6 (0.45)	105.6 (1.21)	92.3 (1.01)	79.4 (0.56)	78.9 (0.72)	52.2 (0.78)	90.2 (1.03)	66.7 (0.39)	89.3 (0.45)	0.784 (0.413)

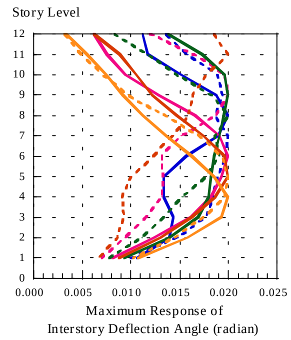
unit: cm/s
Nomenclatures μ and ν designate the mean and coefficient of variation of the statistical quantities, respectively.

SEISMIC PERFORMANCE EVALUATION ON THE BUILDINGS (CONDITION-B)

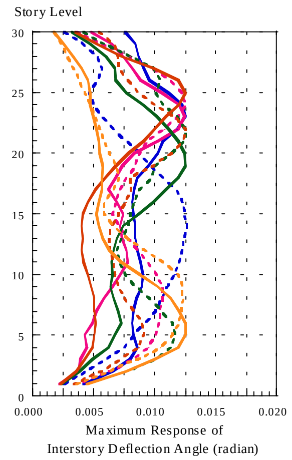
The design concept of weak-beam strong-column is essentially adopted producing flexural yield hinges at the ends of beams. Beams will appear flexural yielded, while columns shall remain unyielded against an intense seismic action. For a multi-story building, a large response at specific story levels might not produce critical and significant damages to the entire building. In the study hereinafter, high-rise buildings with multi-story levels are judged severely suffered damages leading to be collapsed, provided that the lateral displacement at the point of center of resultant lateral loads applied to the building exceeds the critical response. A response deflection angle, named herein as a building deflection angle, is defined for the response displacement produced at the center of resultant lateral loads.



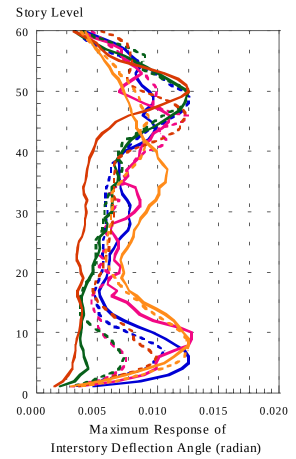
(a) Interstory response of the four-story building.



(b) Interstory response of the 12-story building.



(c) Interstory response of the 30-story building.



(d) Interstory response of the 60-story building.

Figure 2 Maximum response of interstory deflection angle of building: (a) the four-story low-rise building; (b) the 12-story medium-rise building; (c) the 30-story high-rise building; and (d) the 60-story high-rise building.

When the distribution of lateral loads applied during the seismic action can be well represented by the inversed triangular, the point of resultant seismic lateral loads is positioned at the two-thirds of the total building height. The coefficients Ai's in Eq. (1) specifying the shear force distribution in the Japanese design code would correspond well to the lateral load distribution represented well by the inversed triangular shape as defined in the other codes in the world.

Criteria for the Buildings to Reach the Ultimate Conditions (Condition-B.1)

The buildings are found seriously damaged when the building deflection angle response reaches 1/100. The deflection angles of 1/100 are taken common among the four buildings examined herein regardless the number of stories.

Results of Analysis for the Specified Criteria (Condition-B.1)

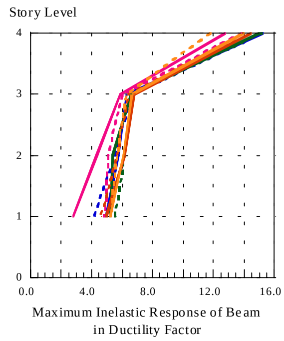
Due to the limitation of space of the paper, herein this paper the results are not discussed in detail obtained for the case when the critical building deflection angles of 1/100 are taken.

Criteria for the Buildings to Reach the Ultimate Conditions (Condition-B.2)

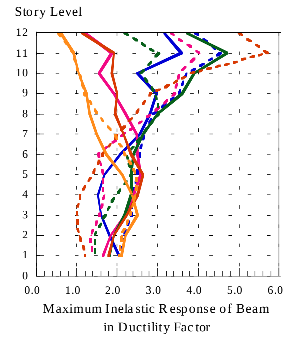
The buildings are judged seriously collapsed down when the responses exceed the critical building deflection angle of 1/50 for the 12-story medium-rise building and 1/80 for the 30- and 60-story high-rise buildings, respectively.

Results of Analysis for the Specified Criteria (Condition-B.2)

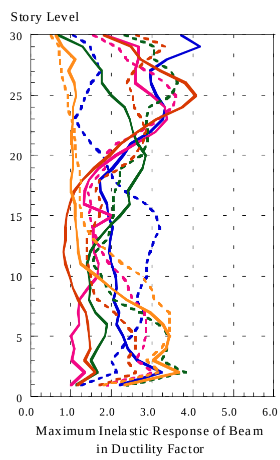
The critical intensity of ground motion with which the building is considered collapsed down is tabulated in Table 4. Expressions of the table are identical to those of Tables 2 and 3, while the building deflection angle R' is taken instead of the



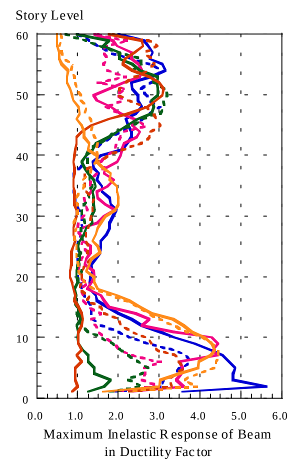
(a) Beam ductility response of the four-story building.



(b) Beam ductility response of the 12-story building.



(c) Beam ductility response of the 30-story building.



(d) Beam ductility response of the 60-story building.

Figure 3 Maximum inelastic response of flexural response at the end of beam at each story level: (a) the four-story building; (b) the 12-story building; (c) the 30-story building; and (d) the 60-story building.

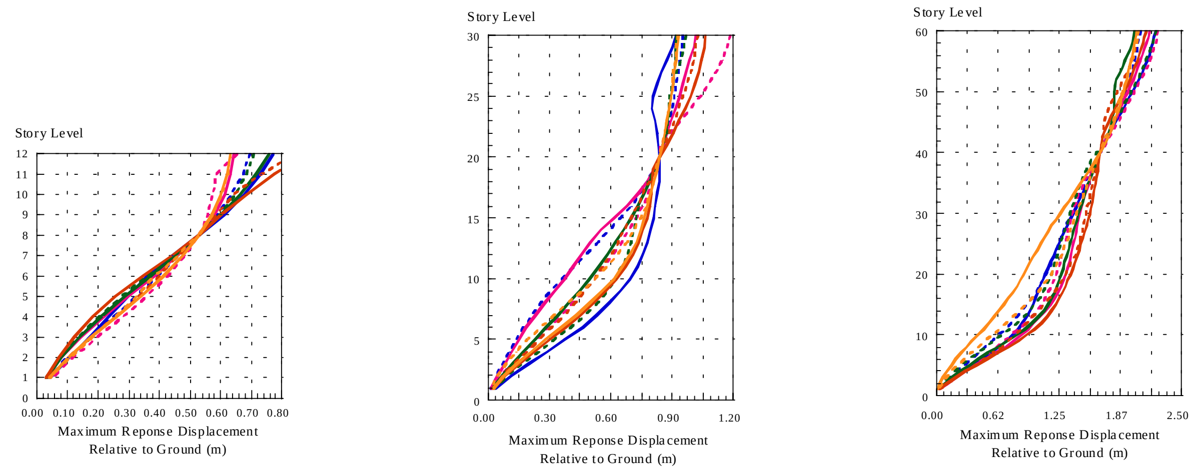
TABLE 4. Intensity of Ground Motion in Velocity for the Buildings to Ultimate Conditions (Condition-B.2)

		Component of Earthquake Strong Ground Motion										Statistical Quantities
		ELC S00E	ELC S90W	TFT N21E	TFT S69E	HCH NS	HCH EW	THU NS	THU EW	TWA NS	TWA EW	μ (ν)
Peak Velocity of Real Motion		34.2	37.1	15.5	17.7	34.9	38.9	36.5	27.8	208.0	201.9	
12-story Building	R'= 1/50	130.1 (1.0)	141.5 (1.0)	100.5 (1.0)	179.9 (1.0)	156.7 (1.0)	124.2 (1.0)	123.3 (1.0)	127.3 (1.0)	210.1 (1.0)	228.1 (1.0)	1.00 (-)
30-story Building	R'= 1/80	167.2 (1.29)	93.9 (0.66)	130.8 (1.30)	146.7 (0.82)	118.3 (0.76)	184.4 (1.48)	148.9 (1.21)	124.5 (0.98)	177.8 (0.85)	148.0 (0.65)	0.999 (0.300)
60-story Building	R'= 1/80	196.0 (1.51)	142.1 (1.00)	226.3 (2.25)	205.3 (1.14)	225.1 (1.44)	214.7 (1.73)	220.1 (1.78)	267.7 (2.10)	80.8 (0.38)	120.6 (0.53)	1.387 (0.450)

unit: cm/s

Nomenclatures μ and ν designate the mean and coefficient of variation of the statistical quantities, respectively.

interstory deflection angle R within the table. Figures 4(a) through 4(c) illustrate the deflection responses of the building indicating the response mode shape during seismic excitation, the legends of which are identical to those of Figs. 2 and 3. Note that the responses have been evaluated and discussed for the 12-story medium-rise, and the 30-story and 60-story high-rise buildings. The responses for the four-story low-rise building have not examined herein.



(a) Building response displacement of the 12-story building.

(b) Building response displacement of the 30-story building.

(c) Building response displacement of the 60-story building.

Figure 4 Maximum response of building deflection relative to the ground: (a) the 12-story medium-rise building; (b) the 30-story high-rise building; and (c) the 60-story high-rise building.

CONCLUDING REMARKS

Examined is the seismic resisting performance of moment-resisting ductile reinforced concrete building systems obtained when subjected to intense earthquake ground motions. Four types of model buildings are designed in accordance to the practice in Japan. A four-story, 12-story, 30-story and 60-story building model in each story height has been established representing a low-, medium- and high-rise building in a high seismic activity zone, respectively. Criteria with which the building is judged heavily damaged and/or collapsed down are specified. A nonlinear dynamic response analysis has been carried out, and the peak ground velocity (PGV) of ground motion with which level the building responses reach the conditions prescribed in the criteria. The seismic capacities of the buildings are directly correlated with the evaluated PGV of ground motions, with which the buildings produce great responses beyond the prescribed response deflection angle.

Analytical results obtained within this study can be summarized in the following:

(1) When the buildings are judged significantly damaged and/or collapsed down in terms of interstory deflection angle, the 12-story medium-rise building reveals highest seismic performance when compared to the 4-story low-rise, 30- and 60-story high-rise buildings specified herein the study in accordance with the practice in Japan.

(2) The evaluated seismic performance capacities obtained for the condition that the specific interstory deflection angle reaches 1/100, however, are almost identical with one another among the four types of buildings examined herein.

(3) When the buildings are considered seriously damaged and collapsed down in terms of structural deflection angle, the evaluated seismic performance capacity of the 60-story building is greatest. Those evaluated for the 12-story building and for the 30-story building are identical with each other.

(4) The performance obtained with the criteria defined in terms of the structural deflection angle is greater than that obtained with the criteria defined in terms of the interstory deflection angle. Note that criteria for the interstory deflection angle indicate the condition that a specific large interstory deflection response may produce the collapse of building, and those for the structural deflection angle correspond to the condition that the total deflection response will generate serious damages of building designed with weak-beam and strong-column concept. This tendency will be significant for the case in the high-rise buildings. The performance with the structural deflection angle is 1.3 times as large as that with the interstory deflection angle for the 12-story building, while that with the structural deflection angle is about twice as large as that with the interstory deflection angle for the 60-story building.

Within the study presented herein, seismic performance capacities of moment-resisting reinforced concrete buildings designed in accordance to the Japanese practice have been examined and discussed. The procedures introduced herein can be applied to buildings designed by other seismic codes. The results obtained herein can be extended utilized to assess the vulnerability of a group of buildings in an urban area located within a high seismic activity zone.