STOREY SHEAR SAFETY FACTOR FOR RC BUILDINGS

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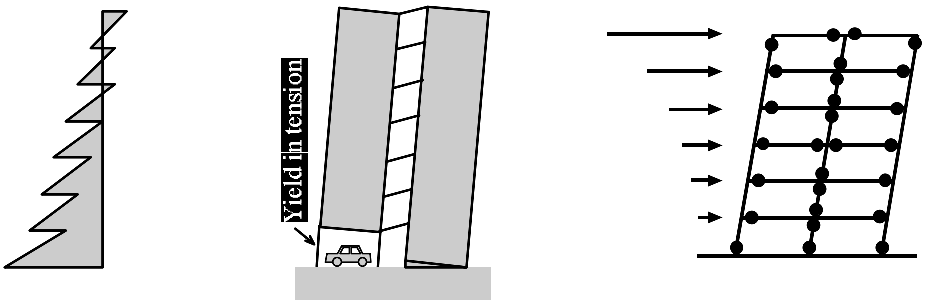
SUMMARY

The concept of a ‘dynamic magnification factor’ proposed by Paulay is currently the only tool for preventing story collapse in buildings. However, this concept is not applicable for existing buildings that have plastic hinges both in the beams and in the columns. Therefore, in the present study, we propose a ‘story-shear-safety-factor’, which can be used to prevent story collapse in buildings of this type. The factor is defined as the ratio of the story shear force when story collapse occurs to the story shear force when total collapse occurs. Through a series of dynamic analyses, simple equations are provisionally proposed to calculate the necessary story shear safety factor that can be used to prevent story collapse.

INTRODUCTION

Capacity design philosophy [Paulay et al 1992] is a powerful tool used to prevent the mechanism responsible for story-collapse. One of the key elements of the philosophy is a dynamic magnification factor that is multiplied to the column moments provided by static analysis to obtain the column design moments. However, when considering various specific structures a number of questions arise regarding the validity of a dynamic multiplication factor. For instance, when the column moment pattern is dominated by cantilever action as shown in Fig. 1a, is a dynamic multiplication factor required? In addition, is a dynamic magnification factor applicable when designing an irregular building such as that shown in Fig. 1b? Furthermore, when we evaluate the seismic vulnerability of an existing structure that fails in mixed mode as shown in Fig. 1c do we need to strengthen the structure to prevent the story-collapse mechanism? The purpose of the present study is to find a criterion to prevent the story-collapse mechanism in the structures shown in Figs. 1a, 1b and 1c.



(a) Column moments with weak beam (b) An irregular building (c) Mixed mode

Fig. 1 Structures to which dynamic magnification factor is not applicable

DEFINITION

The 'story-shear-safety-factor' of the *i*-th story is defined as

$$\alpha_i = \frac{Q_{si}}{Q_{ti}} \tag{1}$$

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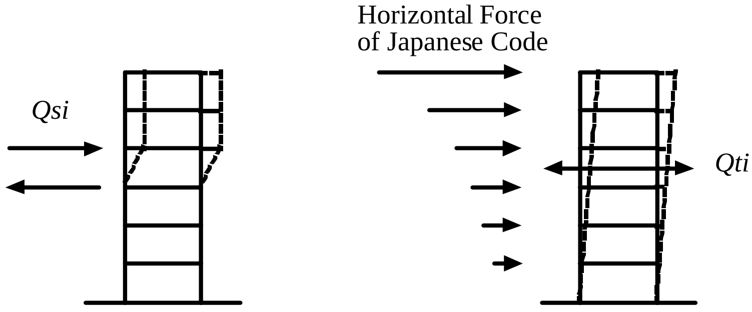
where Q_{si} is the story shear when the story-collapse mechanism occurs under the forces shown in Fig. 2a; and Q_{ti} is the story shear when the structure is subjected to distributed horizontal forces until total collapse as shown in Fig. 2b. The distribution of the forces is tentatively given according to Japanese design code.



(a) *Qsi* (b) *Qti*

Fig. 2 Definition of story shear safety factor

STORY-SHEAR-SAFETY-FACTOR TO PREVENT STORY COLLAPSE

Inelastic response analyses were performed to investigate the story-shear-safety-factor used to prevent story collapse. The input waves used for the analyses are taken from the Fukiai EW (1995 Kobe Earthquake) and Hachinohe NS (1968 Tokachi-oki Earthquake) records. The input waves are amplified so that the maximum velocity is 100 m/s. The RESP-F program [Kozo 1997] is used together with Giberson's one component model and the Takeda model to analyze 3-, 9-, and 15-story buildings each with a story height of 3 m and span length of 10 m as shown in Fig. 3. The weight of each story is assumed 50 tonnes. The elastic stiffness and the crack moments of the beams and columns are calculated using the dimensions and concrete strengths shown in Table 1. The bending moments due to vertical load are neglected.

The yield moment of the beam at the *i*-th story of Fig. 3 is given by the following equation,

$$M_{byi} = \frac{Q_i + Q_{i+1}}{2} \times \frac{h}{2} \qquad (2)$$

where h is the story height and Q_i is the shear force of the i-th story having base shear force as contained in Table 2 and a distribution given by Japanese design code. The yield moments of the roof beam and the footing beam are assumed sufficiently large. The yield moment of the column at the i-th story is given by the following equation where α_i is the story-shear-safety-factor.

$$M_{cyi} = \alpha_i \times Q_i \times \frac{h}{2} \qquad (3)$$

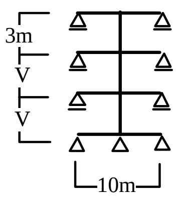


Fig. 3 Analyzed Frame

Table 1. Assumed size and concrete strength

Model	Story	Beam (mm)		Column	Concrete Strength
		Depth	Width	(mm)	(MPa)
3-story	3. 2	645	357	611	22
	1	668	392	645	26
	9. 8	829	459	785	28
9-story	7. 6	859	503	829	34
	5-1	888	548	888	41
	15. 14	946	524	895	32
15-story	13. 12	979	574	946	38
	11-1	1013	625	1013	47

Base shear coefficients of the frames are determined according to one of the following rules:
Rule 1: 0.25 for the 3- and 9-story buildings and 0.238 for the15-story building, which is the minimum requirement under Japanese law. (See Rule 1 in Table 2.)
Rule 2: the base shear coefficients that yield a maximum beam-ductility-factor of 5.0 for input waves based on the Fukiai EW or Hachinohe NS records. These coefficients are obtained from Fig. 4, which shows the maximum ductility factors of beams in buildings with various base shear coefficients. (See Rule 2 in Table 2.) The story-shear-safety-factor at each story is assumed 2.0 in this analysis.

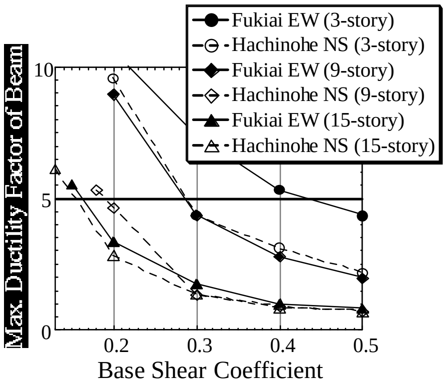


Fig. 4 Maximum ductility factor of beam with various base shear coefficients

Table 2. Base shear coefficients for analyses

	Rule 1 (Minimum)	Rule 2 (μ=5)	
		FukiaiEW	Hachinohe NS
3-story	0.250	0.432	0.288
9-story	0.250	0.286	0.190
15-story	0.238	0.162	0.154

The relationships between the story shear forces and the interstory displacements are plotted in Fig. 5, where the base shear coefficient and the story-shear-safety-factor at each story are assumed 0.3 and 2.0, respectively.

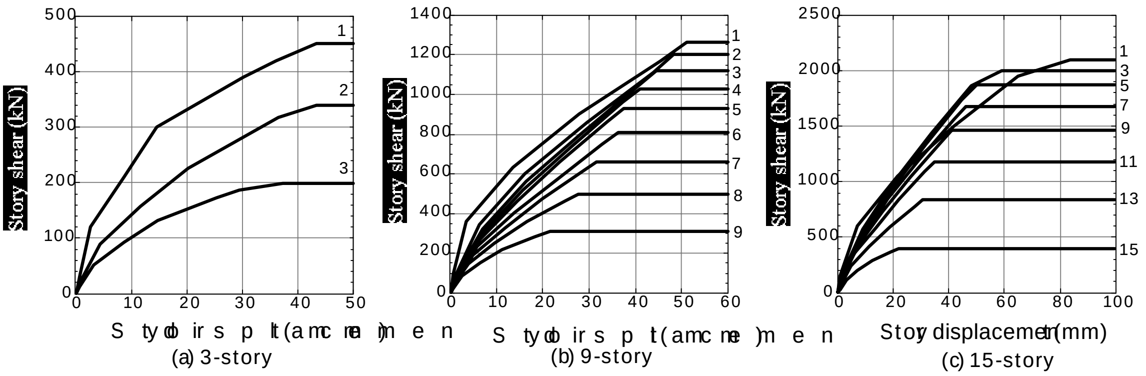


Fig. 5 Relationships between story shear force and interstory displacement

Story collapse may be defined to be when all the columns in a story yield at both the top and the bottom. For example, using this definition story collapse occurs in the first story when the columns yield at the top of the first story. Figure 6 shows the relationship between the maximum ductility factor of the column at the top of the first story and the story-shear-safety-factor (SSSF) of the first story, where the SSSF of the other stories is 2.0. From this figure, the necessary SSSF for the first story is 1.16. Similar analyses were performed for the other stories, and the results are shown in Fig. 7. This figure indicates that the necessary SSSF is largest for the 7^{th} or 8^{th} story and is small for taller buildings.

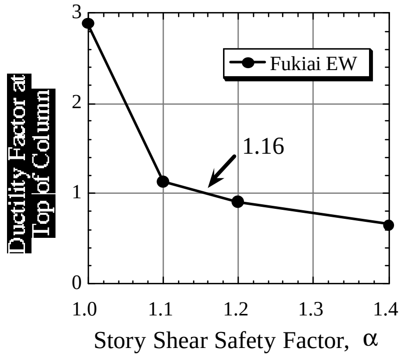
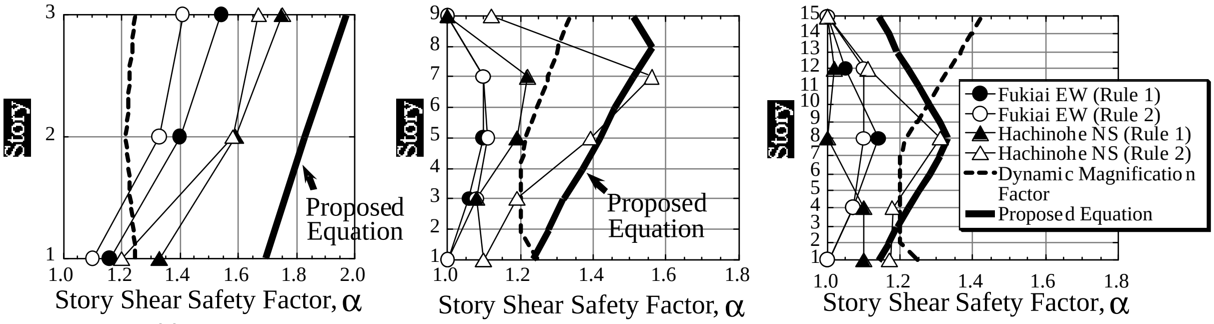


Fig. 6 Maximum ductility factor of the column at the top of the first story

Fig. 7 Necessary story shear safety factor to prevent story collapse



(a) 3-story (b) 9-story (c) 15-story

Noting the above tendencies, we tentatively propose the following equations,

$$\alpha_i = 1 + \frac{0.1}{T_1}\left(1 + \frac{i}{4}\right) \qquad \text{for } i \le 8 \tag{4}$$

$$\alpha_i = 1 + \frac{0.1}{T_1}\left(5 - \frac{i}{4}\right) \qquad \text{for } i \ge 9 \tag{5}$$

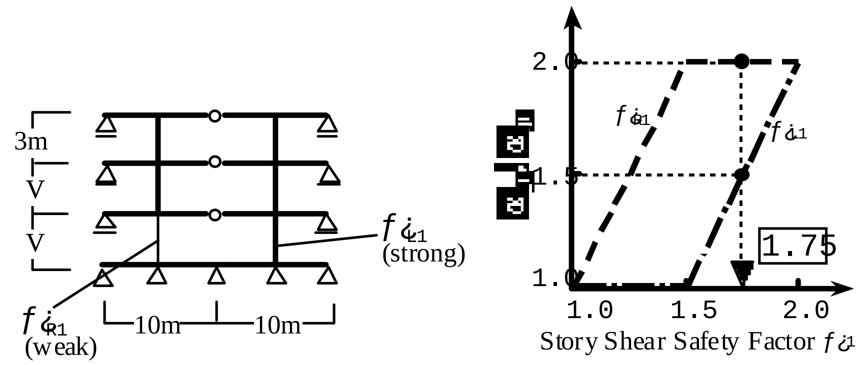
where T_1 is the fundamental period of the building, which we approximated by multiplying the total height of the building in meters by 0.02. The results obtained using the above equations are indicated by the bold lines in Fig. 7. The broken lines indicate the magnification factors given in the Japanese Design Guidelines [AIJ 1997]. Compared with the magnification factors of the AIJ, the proposed values are large for smaller buildings. For the 15-story building, the proposed values are only higher than the magnification factors of the AIJ for the middle stories.

Similar analyses were performed for frames with columns of different strength as shown in Fig. 8a. In this case, the column strengths of the left and right frame are given by the following equations.

Left $$M_{cyi} = \alpha_{Li} \times \frac{Q_i}{2} \times \frac{h}{2} \tag{6}$$

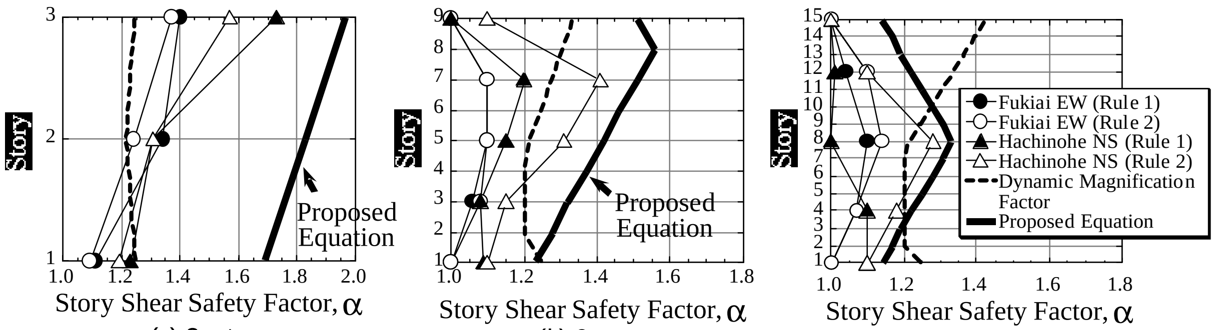
Right $$M_{cyi} = \alpha_{Ri} \times \frac{Q_i}{2} \times \frac{h}{2} \tag{7}$$

The factors, α_{Li} and α_{Ri}, for the weak story are given according to the relationship shown in Fig. 8b. For example, for a story-shear-safety-factor of 1.75, α_{Li} = 1.5 and α_{Ri} = 2.0. The factors used for the other stories are α_{Li} = 2.0 and α_{Ri} = 2.0. Note that Fig. 8b includes α_{Li} = 1.0 (column hinging in the left frame) and α_{Ri} > 1.0 (beam hinging in the right frame). The obtained story-shear-safety-factors are shown in Fig. 9. The results are all within the proposed equations, which indicates that the proposed criteria are applicable for the mixed mode shown in Fig. 1c.



(a) Frame (b) Factor for each column

Fig. 8 Frame with columns of different strengths



(a) 3-story (b) 9-story (c) 15-story

Fig. 9 Necessary story shear safety factor for columns of different strengths

VERIFICATION OF THE STORY-SHEAR-SAFETY-FACTOR

In addition to the above, another series of analyses were performed in order to verify the proposed factor. The base shear coefficients are given by Rule 1 (minimum requirement under Japanese law). The yield moments of the columns are given by equations 3, 4 and 5, i.e., the minimum requirement according to the proposed story-shear-safety-factor theory. The yield moment of the beam at the *i*-th story is given by

$$M_{byi} = \frac{Q_i}{\sum_{i=2}^{n} Q_i}\left(h\sum_{i=2}^{n} Q_i - 2M_{cy1}\right) \tag{8}$$

where h is story height and Q_i is the story shear force at the *i*-th story required under Japanese design code. The other characteristics of the structures are the same as those shown in Fig. 3 and given in Table 1.

The solid circles shown in Fig. 10 represent the bending moments at the top and bottom of the columns under push-over analyses, respectively. The distributions of the horizontal forces used are those given by Japanese design code. The yield moments of the columns provided by story-shear-safety-factor (SSSF) theory are indicated by open circles, and the yield moments required using a dynamic magnification factor (DMF) are indicated by open triangles. Note that the moments provided by SSSF theory are larger than those required using a DMF in the 3-story building but are smaller in the 9- and 15-story buildings.

The maximum responses of the interstory displacement for input waves taken from the Takatori NS record (1995 Kobe Earthquake) are plotted by the solid circles in Fig. 11 compared with the static response (the open circles) when the failure mechanism is formed. The inelastic deformation is large especially in 3-story building. The maximum responses of the beam ductility factor and column ductility factor are plotted in Figs. 12 and 13. For those stories where large inelastic deformation occurred in the top (or the bottom) of the column, the bottom (top) of the column remained elastic. In other words, although the columns yielded story collapse did not occur.

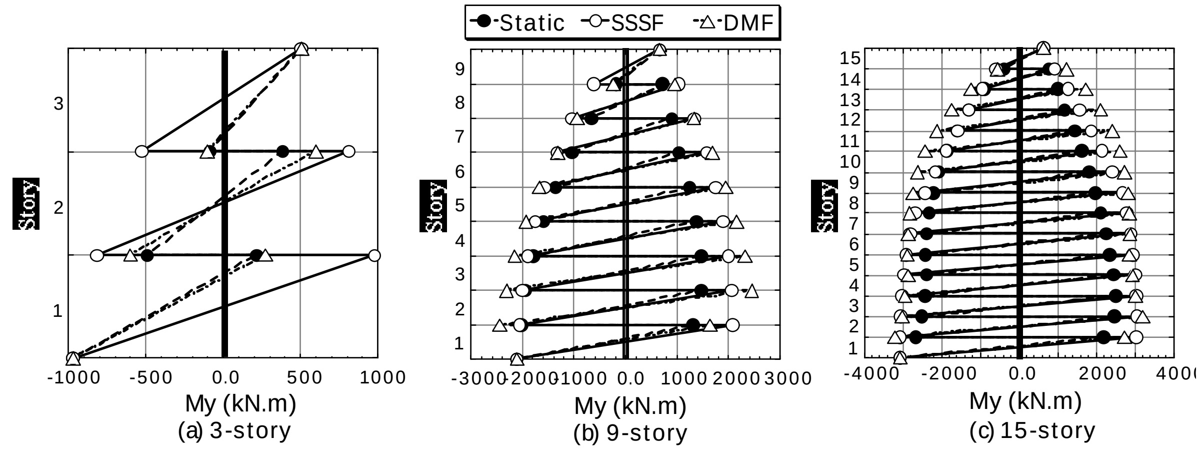


Fig. 10 Bending moments of the columns under push-over analyses (10m)

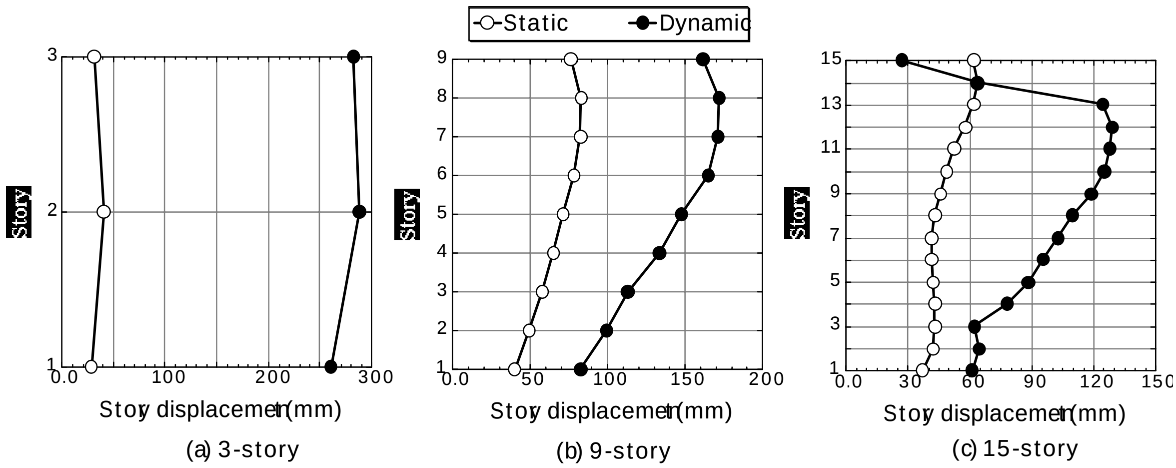


Fig. 11 Maximum response of interstory displacement to the Takatori NS record (10m)

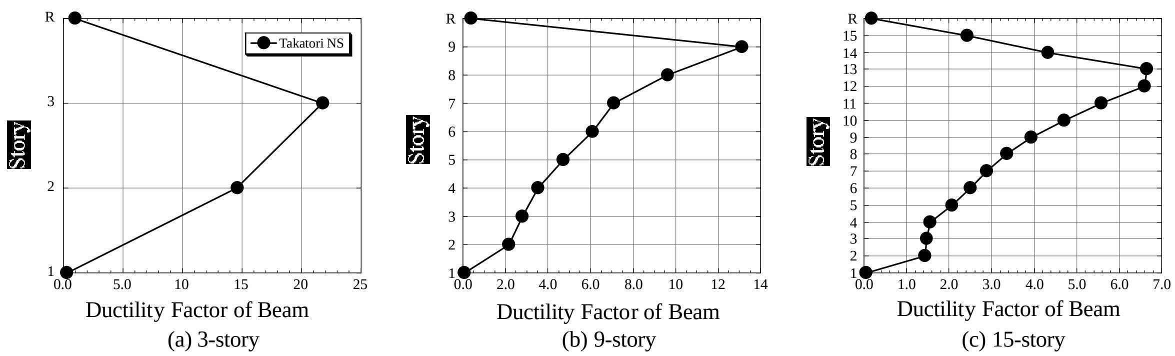


Fig. 12 Maximum responses of beam ductility factor to the Takatori NS record (10m)

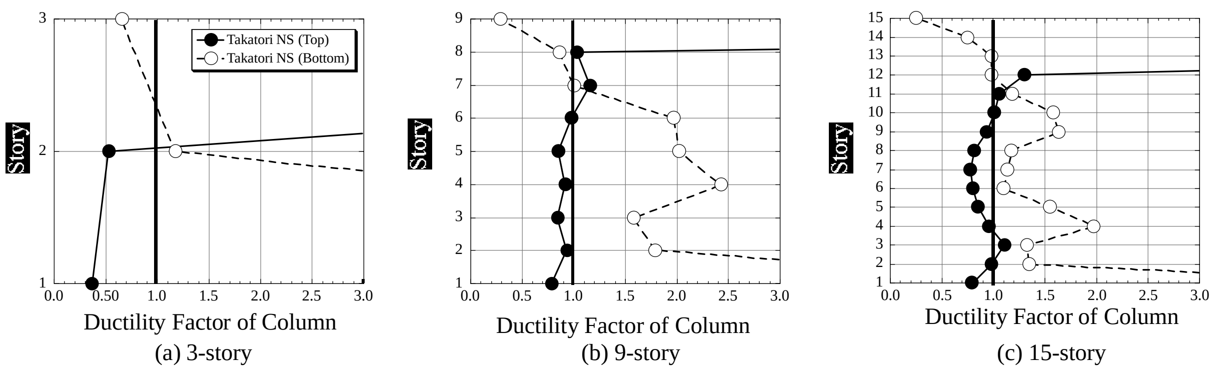


Fig. 13 Maximum responses of column ductility factor to the Takatori NS record (10m)

Figure 14 shows the results of similar push-over analyses where the beam span is changed from 10 m to 5 m. The different beam span affected the column moments required using a DMF but did not affect the moments provided by the SSSF theory. Figures 15 through 17 show the maximum responses of the beam ductility factor and column ductility factor. Story collapse again did not occur. Thus, we conclude that the proposed equations are valid

5. CONCLUSIONS

For the cases studied in the present paper, the story-collapse mechanism can be prevented if a structure is designed so that the story-shear-safety-factor defined in equation 1 is greater than that obtained from equations 4 and 5. The proposed criteria are applicable for structures where beam hinging and column hinging occur simultaneously.

REFERENCES

Architectural Institute of Japan (1997): *Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings Based on Inelastic Displacement Concept*, Tokyo, Japan

Kozo Keikaku Co. (1997): RESP-F, *Inelastic Response Analysis Program for Two-dimensional Buildings*, Version 8.00, Tokyo, Japan

Paulay, T and Priestley, M. J. N. (1992): *Seismic Design of Reinforced Concrete and Masonry Buildings*, John Wiley & Sons, Inc., New York, USA.

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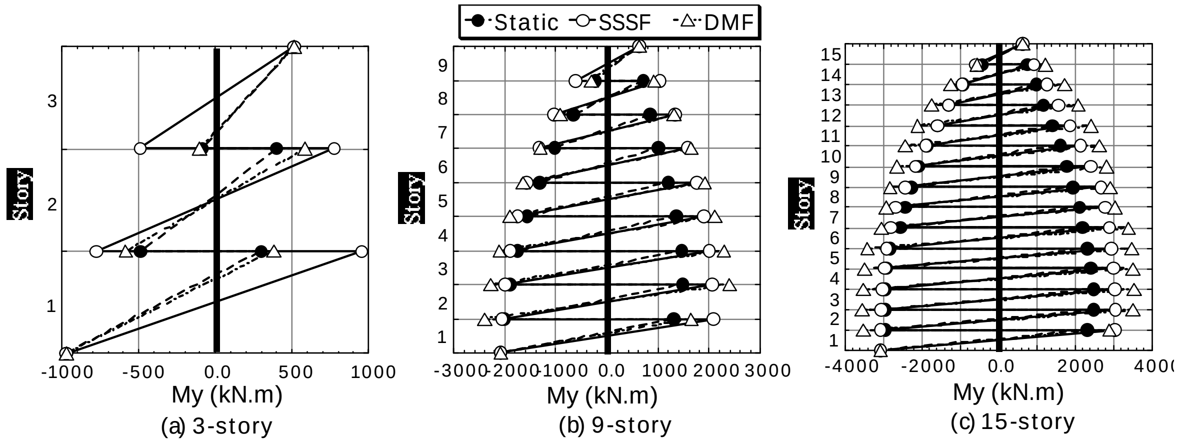

Fig. 14 Bending moments of the columns under push-over analyses (5m)

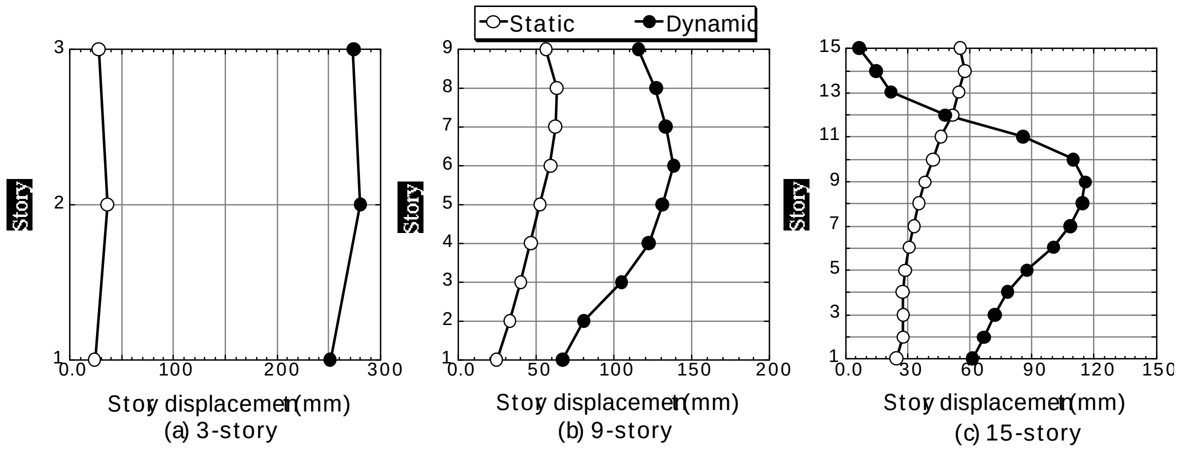

Fig. 15 Maximum response of interstory displacement to the Takatori NS record (5m)

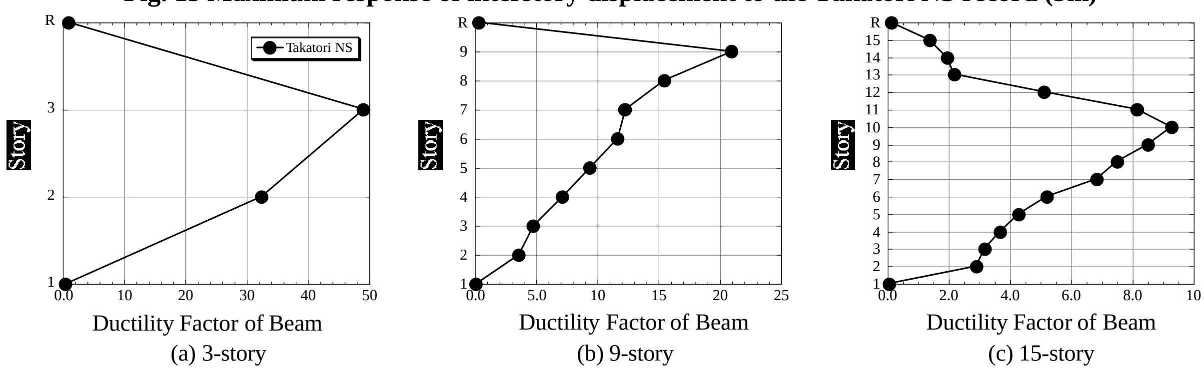

Fig. 16 Maximum responses of beam ductility factor to the Takatori NS record (5m)

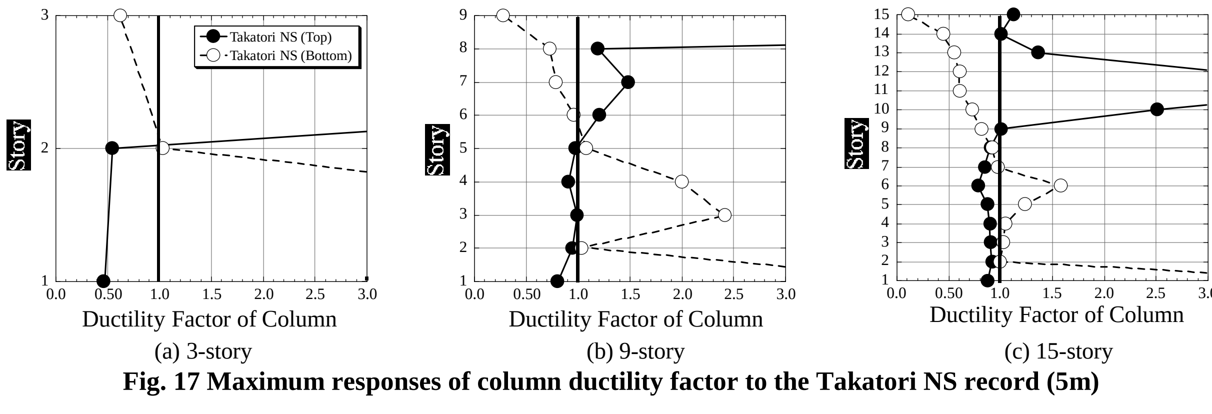

Fig. 17 Maximum responses of column ductility factor to the Takatori NS record (5m)