Available online at www.sciencedirect.com
ScienceDirect
Procedia Engineering 14 (2011) 204–211
Procedia Engineering
www.elsevier.com/locate/procedia

The Twelfth East Asia-Pacific Conference on Structural Engineering and Construction

Failure Mode of Columns of Existing R/C Building Damaged During the 2007 Niigata Chuetsu-Oki Earthquake

Daisuke KATO[1a], Tetsuo NAGAHASHI[2]

[1] Department of Civil Engineering and Architecture, Niigata University,
[2] SU Design Office, Niigata, Japan

Abstract

The school building investigated was a 3-story reinforced concrete (R/C) building built in 1963. The building suffered from a great deal of damage during both the 2004 Chuetsu Earthquake and the 2007 Chuetsu-Oki Earthquake. The damage of the first and second floors during Chuetsu Earthquake was light and that by Chuetsu-Oki Earthquake was moderate. The previous study revealed that the anticipated design failure modes of most of the columns of the building were flexure although most of them actually failed in shear during the 2007 earthquake. This is an important problem to be studied.

In order to study the reason why the columns failed in shear rather than in flexural, a parametric study was conducted, paying attention to parameters including the strength of concrete, hoop spacing and subjected axial force. But those studies could not explain the real phenomenon clearly. After that the effects of cutoff location of longitudinal reinforcement bars were examined and it was concluded that the diagonal crack generated from cutoff point caused shear failure in these columns.

Keywords: Reinforced concrete building; Earthquake damage; Column; Shear failure; Cutoff of longitudinal reinforcement bar.

[a] Email:dkato@eng.niigata-u.ac.jp

1877–7058 © 2011 Published by Elsevier Ltd. Open access under CC BY-NC-ND license.
doi:10.1016/j.proeng.2011.07.024

1. INTRODUCTION

The Niigata Chuetsu-Oki Earthquake generated with the epicenter depth of 17km on July 16, 2007. An earthquake scale was M6.8 and the maximum seismic intensity recorded upper 6 on the Japanese intensity scale in Nagaoka city, etc. The elementary school building investigated was a 3-story reinforced concrete (R/C) building built in 1963 (referred as S-building) in Oguni town of Nagaoka city.
S-building damaged moderately during the earthquake and the seismic performance of the S-building was studied. The previous study (Nagahashi et al. 2009) revealed that the anticipated design failure modes of most of the columns of the building were flexure although most of them actually failed in shear during the 2007 earthquake. In this study in order to study the reason why the columns failed in shear rather than in flexural, a parametric study was conducted, paying attention to parameters including the strength of concrete, hoop spacing and subjected axial force. Secondary the effects of cutoff location of longitudinal reinforcement bars were examined.

2. OUTLINE OF BUILDING AND DAMAGE

S-building suffered a great deal of damage during both the 2004 Chuetsu Earthquake and the 2007 Chuetsu-Oki Earthquake. The damage of the first and the second floor during Chuetsu Earthquake was light although that by Chuetsu Offing Earthquake was moderate. Figures 1(a)(b) show plans of 1st floor and 2nd floor of S-building, where damage index of each column is also shown. Where, damage index "ⅡB" represents that the damage level is Ⅱ(see Table 1) and the failure mode is flexure. Also damage index "ⅢS" represents that the damage level is Ⅲ and the failure mode is shear. Note that two indices are listed for each column because S-building suffered by both 2004 and 2007 earthquakes. Figure 2 shows section plan of A frame composed of column and beam members with spandrel walls. Table 2 shows strength of concrete and reinforcement using concrete cores and reinforcement obtained from the building.

3. EXAMINATION OF FAILURE MODE OF COLUMNS

In this chapter failure modes of columns are discussed. Figure 5(a) shows ratio of shear strength Q_{su} calculated using Eq.(1) (JBDPA 2001) to flexural strength Q_{mu} of columns failing in shear of 1st floor. The horizontal axis represents damage level of columns. Although all columns failed in shear during the earthquake, the calculated results show anticipated failure modes of those columns were flexure. This is a big problem when evaluating the seismic performance of buildings. Figure 5(b) shows the same relation for columns failing in flexure. Figures 6(a)(b) show the same relation for columns of 2nd floor. Note that effects of cutoff bars were ignored in those figures. It is noticeable that the ratio Q_{su}/Q_{mu} were much higher in case of columns of 2nd floor (Figure 6(a)) comparing with columns of 1st floor (Figure 5(a)), which represents that the gap between calculation and observation becomes considerable for columns of 2nd floor.

$$Q_{su} = Q_{sa} + Q_{st} = \left\{ \frac{0.053 p_t^{0.23} (18 + F_C)}{M/(Q \cdot d) + 0.12} \right\} \cdot b \cdot j + \{0.85 \sqrt{p_w \cdot {}_s\sigma_{wy}} + 0.1\sigma_o\} \cdot b \cdot j \quad (1)$$

Where, p_t is tensile reinforcement ratio(%), p_w is shear reinforcement ratio (p_w =0.012 for p_w>0.012), ${}_s\sigma_{wy}$ is yielding strength of shear reinforcement, F_c is concrete strength (unit: N/mm^2), b and D are width and depth of column, j is distance between centroid of tension and compression forces (default value is 0.8D), M/Q is shear span ratio (default value is h_o/2), d is effective depth of column, h_o is clear height of the column. (unit:mm)

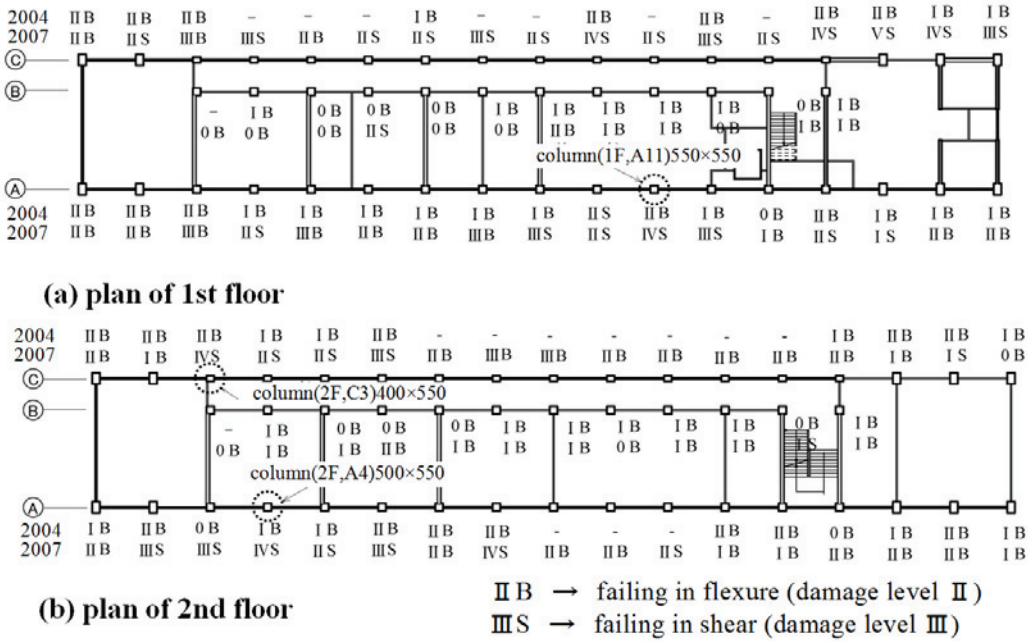


Figure 3 shows reinforcement of 2F-A4 column (column number 4 of A frame of 2nd floor, see Figure 1(b)) and the observed damage. Figure 4 shows isometric drawing and the section of the reinforcement of 2F-A4 column. It must be pointed out that α bar and β bar represent anchorage portion of longitudinal reinforcement of the 1st floor column just under 2F-A4 column. Length between the cutoff point and the top of spandrel wall is 570mm for α bar and 170mm for β bar.

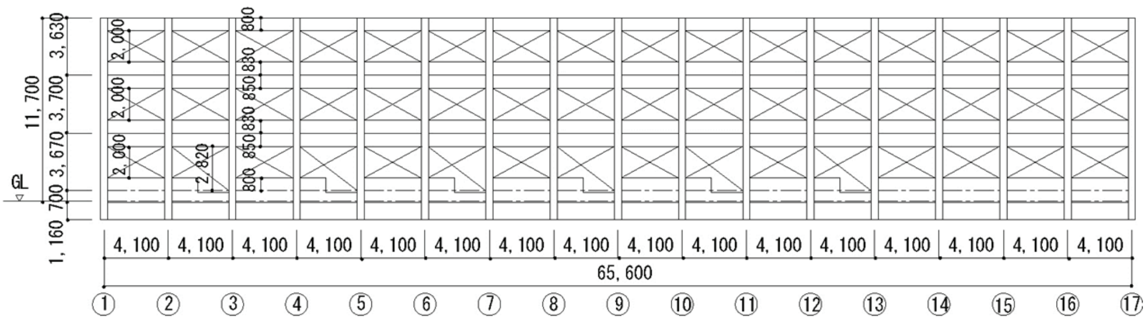


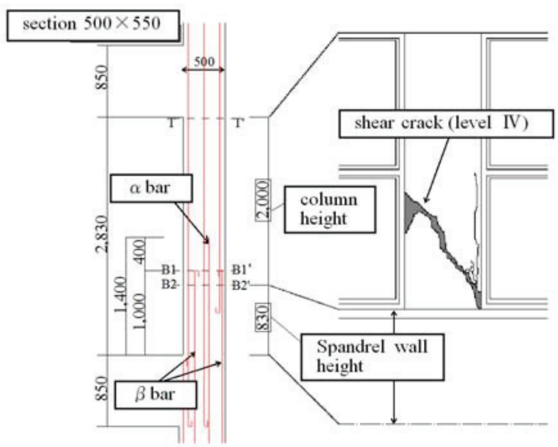
Figure 2: Section plan of A Frame

Table 1: Damage level

damage level	damage
I (1)	slight damage
II (2)	light damage
III (3)	moderate damage
IV (4)	heavy damage
V (5)	collapse

Table 2: Strength of materials

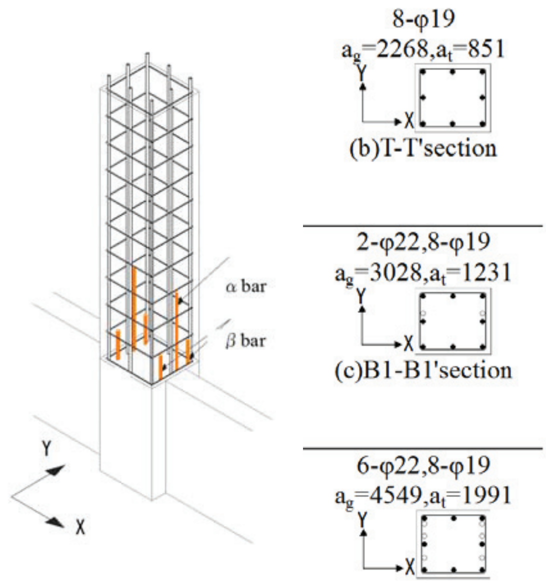
concrete strength	32.7 N/mm^2
yield strength of main bar	354 N/mm^2
yield strength of hoop	339 N/mm^2



(a) Reinforcement (span directin)

(b) Damage (ridge direction)

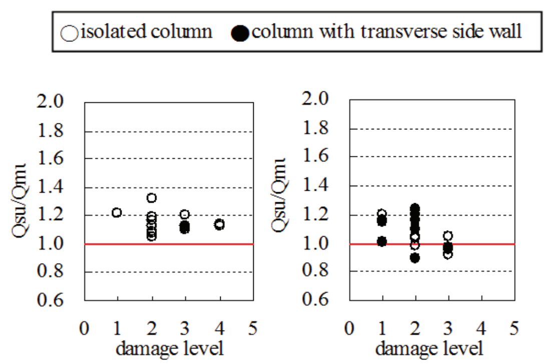
Figure 3: Reinforcing arrangement of 2F-A4 column and the damage



(a) Column of 2F-A4

(b) B2-B2' section

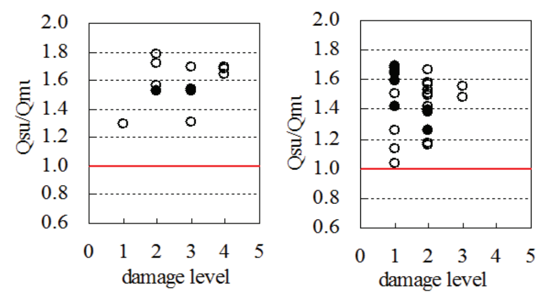
Figure 4: Reinforcing arrangement and the section of 2F-A4 column.



(a) failing in shear

(b) failing in flexural

Figure 5: Failure mode of 1F columns



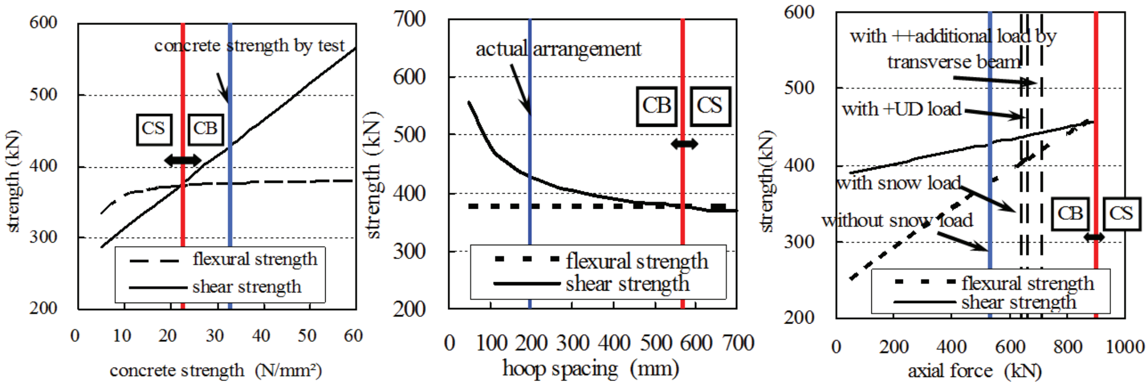
(a) failing in shear

(b) failing in flexural

Figure 6: Failure mode of 2F columns

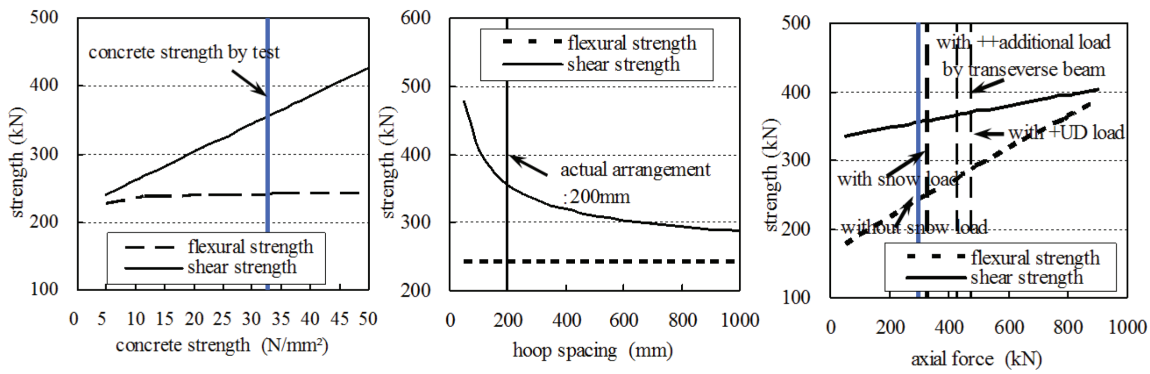
In order to discuss the reason why those columns failed in shear although almost all columns were evaluated as flexural failing columns, the parametric study was conducted, paying attention to three parameters; i.e. strength of concrete, hoop spacing and subjected axial force. Note that hoop spacing represents the effects of hoop. Figure 7(a) shows effects of concrete strength on shear strength and flexural strength of 1F-A11 column of 1st floor. Figures 7(b)(c) show effects of hoop spacing and subjected axial load, respectively. From these figures it can be concluded that it is not impossible to explain that the gap between calculation and observation was caused by the fluctuation of concrete strength, hoop effects and subjected axial force, etc.

On the other hand, Figures 8(a)(b)(c) show the same relation with Figures 7(a)(b)(c) for 2F-A4 column of 2nd floor. It must be noted that flexural strength is lower than shear strength in all range in all cases, which leads to the conclusion that it is impossible to explain that the gap between calculation and observation was caused by the fluctuation of concrete strength, hoop effects and subjected axial force, etc. Therefore effects of cutoff bars should be considered to examine the failure mode of columns of 2nd floor.



(a) concrete strength (b) hoop spacing (c) subjected axial load

Figure 7: Effects of parameters on failure mode of 1F-A11 column

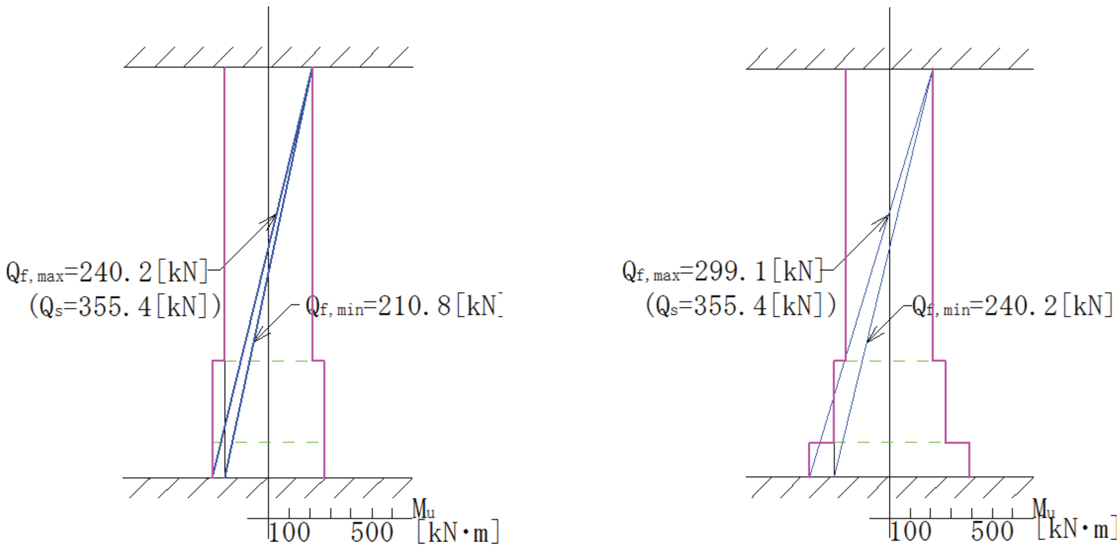


(a) concrete strength (b) hoop spacing (c) subjected axial load

Figure 8: Effects of parameters on failure mode of 2F-A4 column

4. EFFECTS OF CUTOFF BARS ON FAILURE MODE OF COLUMNS

In this chapter effects of cutoff bars of columns of 2nd floor are discussed. Figures 9(a)(b) show moment strength distribution along the column axis of 2F-A4 column paying attention to cutoff bars. Figure 9(a) shows the distribution assuming that α bars are perfectly effective and β bars are perfectly ineffective (case 1). On the other hand Figure 9(b) shows that assuming all bars are perfectly effective (case 2). Both figures indicate that shear strength is higher than flexural strength even though all cutoff bars are assumed to be effective. The gap between calculation and observation cannot be explained either from this view point.



(a) Case 1 (b) Case 2

Figure 9: Distribution of moment strength

Finally crack patterns are discussed. Crack patterns of 2F-A4 column shown in Figure 3(b) indicate that the crack generated at the cutoff point of α bar developed into the column end diagonally. In other words it is assumed that the original crack was caused by flexural moment at the cutoff point but the developed crack formed diagonally and influenced the deformation capacity of the column just like shear cracks.

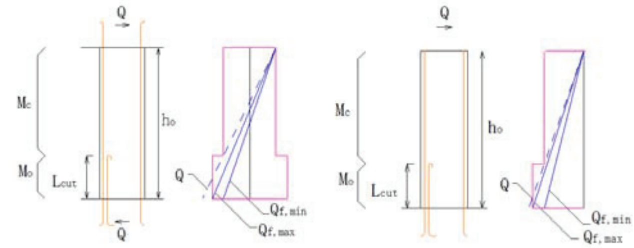
This phenomenon analogizes to failure mechanism of pier columns with cutoff main bars. From this view point the 2F-A4 column is compared with experimental data using simple beam specimens (Ozaka et al. 1986). Figures 10(a)(b) show moment strength distributions for building column (column subjected to bending at both ends) and cantilever column (half portion of simple beam). In the figures possible moment distribution are also shown, where L_{cut} represents length between cutoff point and column end, Q represents shear force at flexural yielding of cutoff point, $Q_{f,min}$ represents that ignoring cutoff bars and $Q_{f,max}$ represents that considering cutoff bars at the column end which represents the maximum shear force . Figure 11 shows relations between Q (normalized by $Q_{f,max}$) and L_{cut} (normalized by h_o), where Q reaches the ceiling of $Q_{f,max}$ for long L_{cut} range.

In order to compare building columns with cantilever pier columns or simple beams, influential factor α of cutoff bar on failure mode, which is defined as b/a in Figure 11, is introduced as expressed by Eq. (2).

$$\alpha = \frac{1}{\dfrac{Q_{f,\max}}{Q_{f,\min}} - 1} \times \frac{1}{\dfrac{h_o}{L_{cut}} - 1} \tag{2}$$

$$Q_{f,\max} = \begin{cases} (M_c + M_o)/h_o & \text{(column subjected to bending at both ends)} \\ M_o/h_o & \text{(cantilever column or simple beam)} \end{cases}$$

$$Q_{f,\min} = \begin{cases} 2M_c/h_o & \text{(column subjected to bending at both ends)} \\ M_c/h_o & \text{(cantilever column or simple beam)} \end{cases}$$



(a) column subjected to bending at both ends

(b) cantilever column

Figure 10: Distribution of moment strength of columns with cut off bars

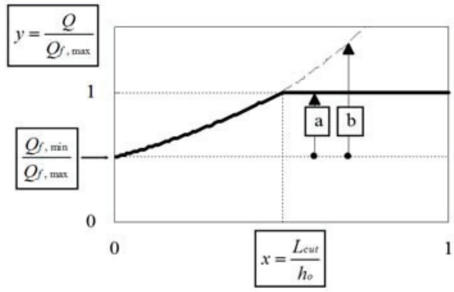
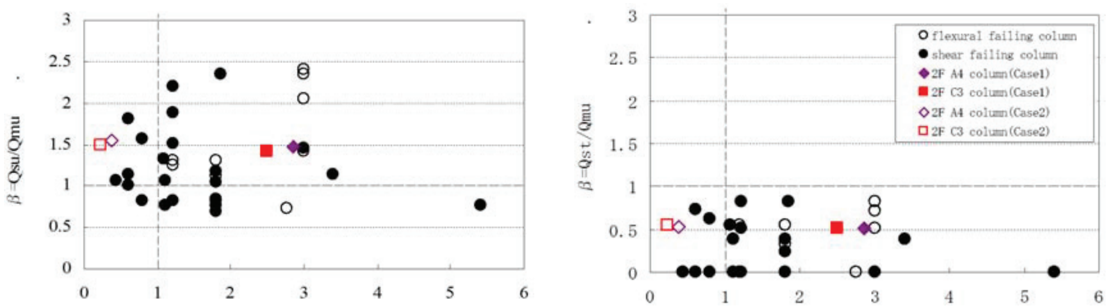


Figure 11: Relation between Lcut and shear force at yielding of column with cutoff bar

Figure 12(a) shows relationship between influential factor α and shear strength margin ratio ($\beta = Q_{su}/Q_{mu}$) of S-building columns comparing to simple beam specimens with cutoff bars (Ozaka et al. 1986). In the figure two cases shown in Figure 9 are considered for S-building columns and failure modes are shown by symbols for simple beam specimens. The figure indicates that failure modes of almost simple beam specimens are shear failure although the shear strength margin ratios are higher than 1 and the S-building columns are located among the beam specimens, which leads to the conclusion that the shear failure of 2nd floor columns of S-building were caused by cutoff bars.

Shear resisting mechanism is assumed to change after diagonal crack occurs, which means the degradation of shear strength due to the extinction of the contribution by arch action. From this view point in Figure 12(b) Q_{su} is replaced by Q_{st} which represents shear strength by truss action only as expressed by Eq. (1). The figure indicates that calculated failure modes of these members are shear failure on the assumption that shear is carried by truss action only. In other words shear design using Q_{st} can be effective in order to prevent shear failure.



Influential factor α
(a) β=Qsu/Qmu

Influential factor α
(b) β=Qst/Qmu

Figure 12: relation between shear margin factor β and index α

5. CONCLUING REMARKS

The 2nd floor columns of S-building failed in shear during 2007 earthquake although anticipated failure modes were flexure. The reason of this gap was concluded that the crack generated at the cutoff point developed into the column end diagonally and the diagonal crack reduced the shear strength of the column.

This phenomenon analogizes to failure mechanism of pier columns with cutoff main bars. But the difference is that cutoff bars are necessary in case of pier columns, while cutoff bars are not necessary in

case of S-building, which means cutoff bars of S-building were the anchorage portion of 1st floor column just under the objective column.
Shear design using Q_{st} can be effective in order to prevent shear failure.

REFERENCES

[1] Nagahashi T., Kokubo T., Nakamura Y. and Kato D. (2009). Seismic Performance of a R/C School Building Suffered during the Chuetsu and Chuetsu Oki earthquake. Journal of Structural Engineering, Architectural Institute of Japan and Japan Society of Civil Engineering. Vol 55B , pp. 459-468.(in Japanese)

[2] Ozaka Y., Suzuki M., Terasawa M. and Kobayashi S. (1986). Effect of Bar Cutoff on Shear Strength of Reinforced Concrete Beams. Journal of Structural Mechanics and Earthquake Engineering, Japan Society of Civil Engineering. Vol 366/V-4 , pp. 133-142.(in Japanese)

[3] The Japan Building Disaster Prevention Association (2001). Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings