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Nonlinear Analysis of a Reinforced Concrete Building with a Soft First Story Collapsed by the 1995 Hyogoken–Nanbu Earthquake

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Abstract

A very new reinforced concrete (RC) building with a soft first story was collapsed by the 1995 Hyogoken–Nanbu earthquake. Nonlinear dynamic response analysis, where strength deterioration was considered in representing member nonlinearity, was conducted to simulate how the building behaved and eventually collapsed during the earthquake. The analysis was found to reproduce the observed damages well, such as residual displacement, mechanism and damages to members. It was also revealed that if first story mechanism might occur, the collapse could be unavoidable even for buildings with a base shear strength of as much as 60% of the total weight.

INTRODUCTION

Many of reinforced concrete (RC) buildings damaged by the 1995 Hyogoken–Nanbu earthquake were those constructed before 1981, when the Japanese building code provisions were extensively revised. However, some RC buildings were found suffering severe damage amongst those constructed after 1981 and most of them were buildings with a soft first story.

In this paper, an example of the damaged soft first story buildings is taken up and inelastic dynamic response analysis is conducted to simulate the building behavior during the earthquake. The building encountered the earthquake only one and a half months later than the completion and collapsed.

Strength deterioration in member hysteresis is usually not considered in the nonlinear dynamic response analysis because of the problem in numerical solution. However, it is considered in this analysis in order to reproduce the process of building collapse as realistically as possible.

OUTLINE OF BUILDING AND DAMAGE

Building

The building was located in the Nada ward of Kobe City, which was one of the most severely affected areas. The building was a RC, seven story apartment house with no basement. The first story was used for parking lots. Figures 1 and 2 show plan views and section views of the north–south (X) direction, respectively. The X-direction, for which major damages occurred, was a frame-wall structure with structural walls in Frame X3 at the first story and in Frames X2 and X3 at the second through seventh stories. There were also nonstructural walls in Frame X1 at the second through seventh stories, which was separated from the edge columns by slits. Other nonstructural walls, such as small walls in Frames X2 and X4 at the second through seventh stories and walls around the stairs were ignored in the analysis. The east–west (Y) direction was a frame structure.

The total weight of the building was 2240 tf. Specified concrete strength was 210 kgf/cm^2 and the nominal yield stress of steel was 3.5 and 3.0 tf/cm^2, respectively, for bars greater than or equal to D19 (number after alphabet D denotes bar diameter in mm) and for bars smaller than D19.

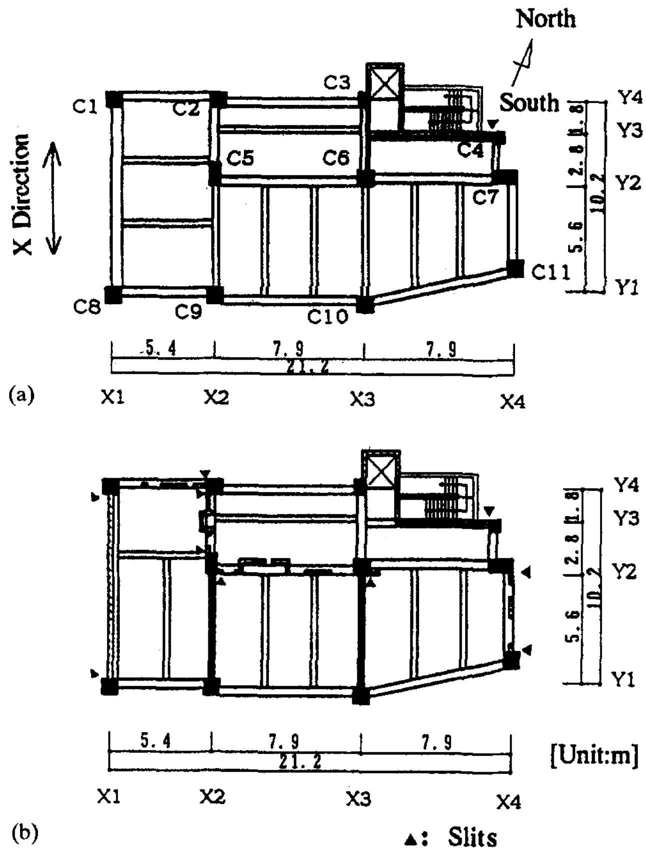

Fig. 1. Plan Views: (a) first story; (b) second through seventh stories.

Figure 3 shows sections of the first story columns. D25 bar was used as main bars and a D16 bar was used as a hoop with 100 mm spacing. Two ties of a D16 bar were provided for C5 for the X-direction and for all columns for the Y-direction. Details of slit are shown in Fig. 4. The slit, although called so, did not perfectly separate the nonstructural wall from the edge column in reality. The slit thickness (50 mm) was only one third of the wall thickness (150 mm).

Damage of first story

Damages were observed to concentrate upon the first story of the X-direction and those to the other stories in this direction and to the Y-direction remained slight. A typical first story mechanism formed for the X-direction. Figure 5 shows lateral and vertical residual displacement of the first story columns, which was measured as relative movement of the column top and bottom. The arrow in the figure indicates amplitude and direction of lateral residual displacement. Lateral displacement was extremely large, ranging from 16.3 to 30.1 cm, 23 cm on average. All columns were observed to move to the north: clear evidence of torsion was not found. Vertical displacement (column shortening) was much larger for the north columns (C1 through C4) with 41 cm on average, than for the south columns (C8 through C11) with 11 cm on average.

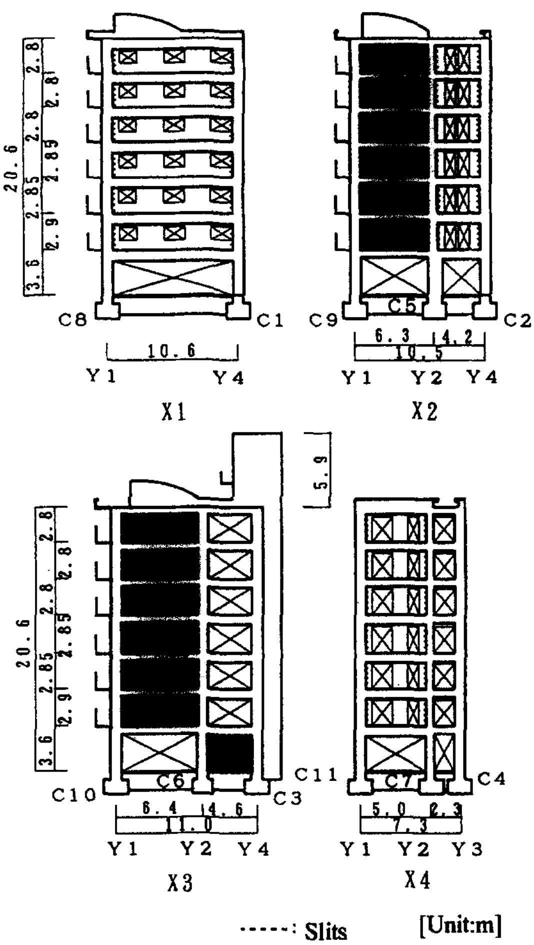

Fig. 2. Section views (X-direction).

Name	C 1	C 4	C 5	C 8
Size	850×850	600×600	600×1200	850×850
X Dir.	9 / 9	7 / 7	7 / 7	8 / 8
Main Bar	32-D25	24-D25	24-D25	28-D25
Hoop	D16@100	D16@100	D16@100	D16@100

[Unit:mm]

Fig. 3. Sections of first story columns.

Plate 1 shows typical damages to the first story members. C1 and C8 were selected as representatives of the north and south columns. The damage was more severe for the north columns than for the south columns: C1 was completely destroyed at the top, while C8 escaped from complete collapse, although severe flexure or flexure–shear failure occurred at both ends and pronounced bond-splitting cracks appeared along the member length. Such a difference in damages to the north and south columns apparently corresponds to the fact that vertical displacement was larger for the former than for the latter. Although various shear-related failure modes, such as flexure–shear failure or bond-splitting failure, were observed for some columns, typical shear failure was not observed for any column. All columns are believed to have failed after experiencing flexure yielding. The first story wall in Frame X3 completely collapsed along with the edge columns at the midheight due to sliding shear. The walls around the stairs also completely collapsed at the first story.

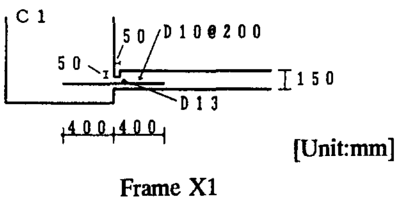

Fig. 4. Details of slit.

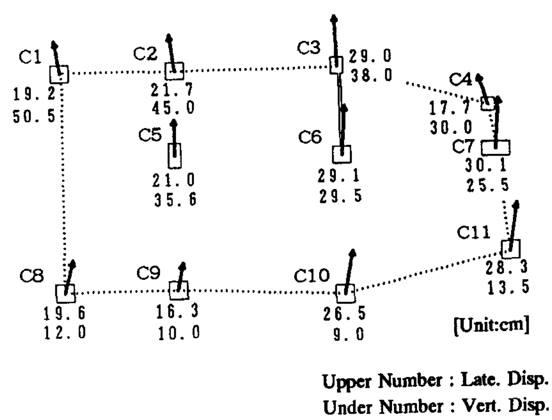

Fig. 5. Residual displacement of first story columns.

Damage of second through seventh stories

Substantial damages were not detected for structural members above the second floor level except for shear cracks occurring in the walls in Frame X2 at the under stories. Shear cracks were also observed in the nonstructural walls in Frame X1 at under stories, but any damage was not detected near the slits placed between the nonstructural wall and edge column. Since the effectiveness of these slits was deemed questionable, both cases treating the slits as effective

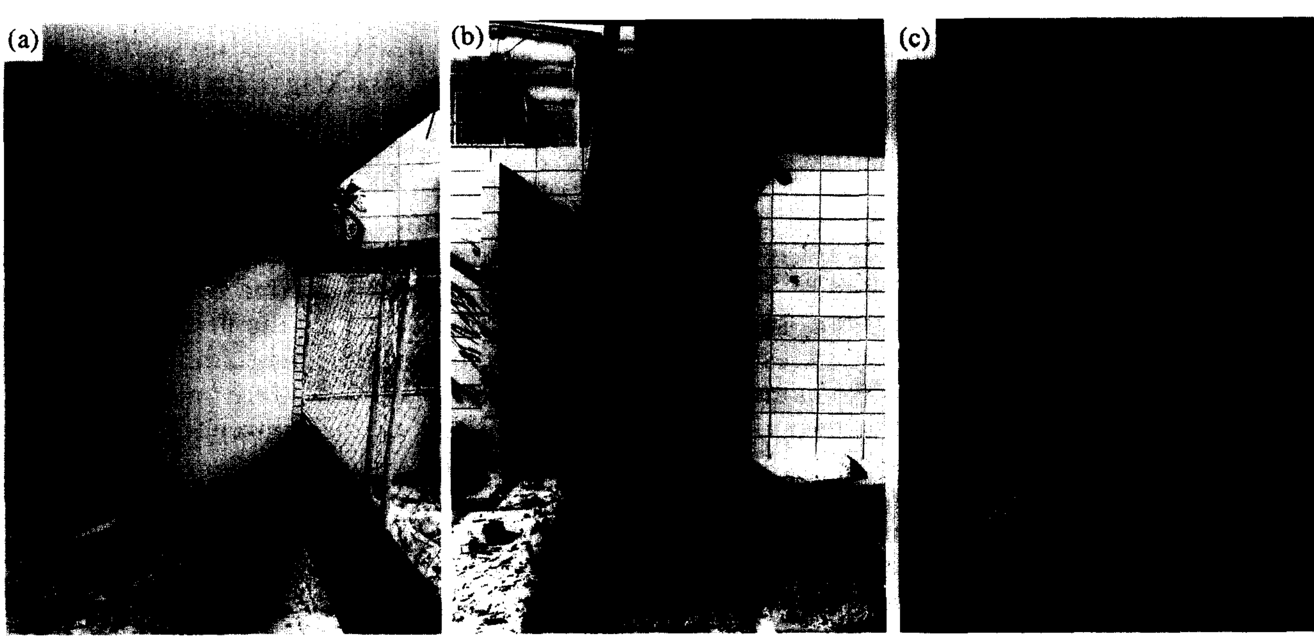

Plate 1. Typical damages: (a) C1 (east face); (b) C8 (east face); (c) first story wall and C3 (west face).

and ineffective, were studied in the following analysis.

METHOD OF NONLINEAR ANALYSIS

For the X-direction, inelastic two-dimensional frame analysis including static analysis and dynamic response analysis using the lateral and vertical ground-motions recorded during the earthquake was performed. The analysis methods are described below.

Modeling of building

(1) The building was represented by a two-dimensional plane frame consisting of Frames X1 through X4 arranged in a line. The small walls in Frames X2 and X4 above the second floor level and walls around the stairs were ignored. The building was assumed to be fixed at the base.
(2) Columns and beams were represented as a line member and a rigid zone was considered for beam–column joints. Walls were represented by a deep column at wall center line with wall properties and a rigid beam at each floor level.
(3) Frame X1 above the second floor level was idealized in two ways as: walls (Model 1), where the slits were considered ineffective; and frames (Model 2), where the slits were considered effective (Fig. 6).
(4) For the dynamic response analysis, lateral mass was assumed to be lumped at each floor level and vertical mass at each beam–column joint.

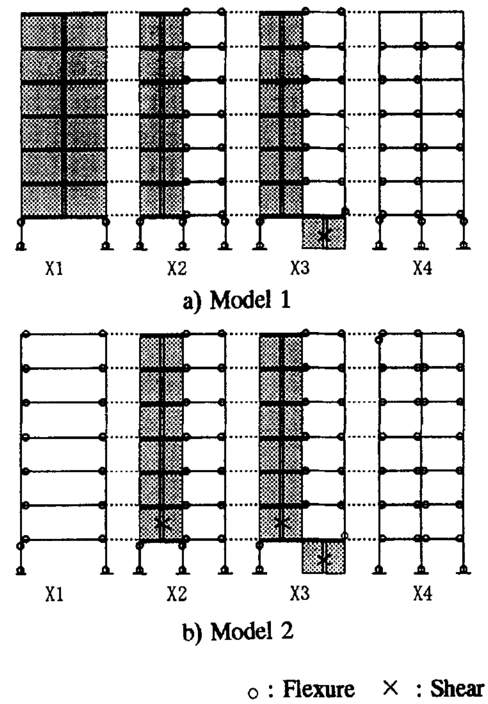

Fig. 6. Modeling of building and location of forming hinges (static analysis).

Modeling of member nonlinearity

(1) For columns and beams, flexure nonlinearity was considered by two springs placed at both ends, the hysteresis of which was represented by Takeda model[1] with different properties for two directions. Shear deformation was taken into account by including it in flexure deformation. For walls, flexure and shear nonlinearity were considered separately: the former was represented by Takeda model with the same properties for two directions and the latter was considered by a spring, the hysteresis of which was represented by origin-oriented model[2] with the same properties for two directions. One-component model was used as a beam model in dealing with flexure nonlinearity. The axial stiffness of columns and walls was assumed elastic.
(2) Member flexure and shear properties were computed based on the specified material properties.
(3) Member flexure properties such as elastic stiffness, cracking strength, yielding strength and yielding rotation, were computed according to the conventional equations[3] in Japan. Post-yield stiffness was assumed as 0.1% of the elastic stiffness except for the case mentioned later. Varying axial load due to lateral loading was considered for the first story columns in computing yielding strength. Member shear properties were also computed according to the conventional equations in Japan.
(4) Table 1 compares flexure and shear strength for the first story columns, where varying axial compression was considered. All columns but C4 and C7 were computed to be controlled by shear strength: the shear to flexure strength

Table 1. Flexure and shear strength of first story columns [Unit:tf]

	C1	*C2*	*C4*	*C5*	*C7*	*C8*	*C9*	*C10*	*C11*
Flexure (1)	226	222	98	200	107	210	189	202	195
Shear (2)	164	165	101	179	158	166	177	170	156
(2)/(1)	0.73	0.74	1.03	0.89	1.48	0.79	0.94	0.84	0.80

ratio of these columns was about 0.8 on average. For these columns it was considered, at first, to evaluate flexure and shear nonlinearity separately like walls. However, it was finally decided to consider flexure nonlinearity alone with flexure strength reduced to the level of shear strength, since flexure yielding was observed to occur prior to the failure, and if flexure and shear nonlinearity were separately evaluated, shear failure would result. Besides, for the outer (north and south) columns, strength deterioration after yielding was considered for the direction where varying axial load was in compression (Fig. 16). Strength deterioration was also considered for the wall shear hysteresis after shear strength (Fig. 15).

(5) Negative member stiffness in the range of strength deterioration may lead to the overall stiffness matrix being singular. To avoid this, the methods[4] shown in Fig. 7 was employed, where member stiffness in this range was assumed zero (exactly speaking very small positive value), and unbalanced force derived from the difference between the real and assumed stiffness was released at next time step by applying equivalent nodal force. Negative stiffness defining the extent of strength deterioration was arbitrarily determined as -2% of the elastic stiffness (SD:-2%). In this case, load would reduce to about 50% of the shear strength at 2% shear strain for the wall shear hysteresis (Fig. 15) and reduce to 80% of the yielding strength at 2% rotation for the column flexure hysteresis (Fig. 16).

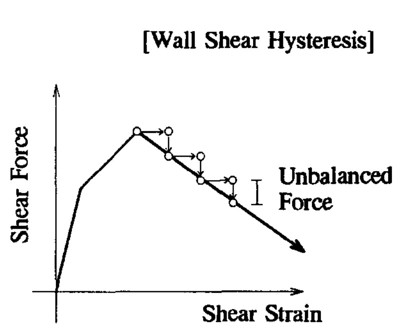

Fig. 7. Method of considering strength deterioration.

STATIC ANALYSIS

Static analysis was performed to assess the inelastic behavior of the building due to monotonically increasing lateral loading. The assumed lateral force distribution along the building height was determined according to the Japanese building code provisions. This force distribution was similar to inversely triangular shape except for a slight top-heavy force at the roof floor. Member strength deterioration was not considered because the decrease in lateral force could not be dealt with in the current static analysis program.

Interstory drift-story shear relations and location of forming hinges are shown in Figs 8 and 6 for roof floor displacement of up to 30 cm.

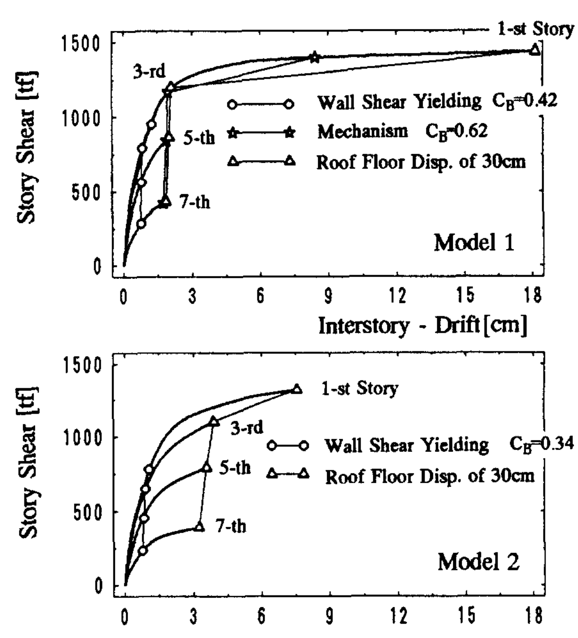

Fig. 8. Interstory drift – story shear relations (static analysis): (a) Model 1; (b) Model 2.

For Model 1, first story mechanism formed at base shear coefficient, C_B, of 0.62 (base shear of 1400 tf) after the first story wall reached shear strength at C_B of 0.42. The first story drift drastically increased in the subsequent loading, reaching 60% of the roof floor displacement at the final step. The fact that this building with base shear strength of as much as 0.62 W (W: total building weight) collapsed, suggests buildings with the first story mechanism need more strength or more deformability compensating insufficient strength. For Model 2, the first story wall reached shear strength and flexure yielding occurred at the bottom of all first story columns, but did not occur at the top of some of them. On the other hand, the second story walls reached shear strength and flexure yielding occurred at nearly all beam ends. These results clearly indicate that complete first story mechanism did not form for Model 2: possible mechanism would be intermediate between the first story mechanism and total mechanism. The first story drift at the final step was 25% of the roof floor displacement, which was much less than the value for Model 1.

DYNAMIC RESPONSE ANALYSIS

Elastic periods and modes are presented in Fig. 9. Fundamental periods are 0.393 and 0.435 s, respectively, for Models 1 and 2. The amplitude of the second mode at the first story is a little larger for Model 1 than for Model 2. This is because the ratio of stiffness of the first story to stiffness of the other stories is lower for Model 1 than for Model 2.

The building was subjected to the NS and UD components (Fig. 10) of the ground motions recorded at Kobe Ocean Meteorological Observatory, Japan Meteorological Agency (JMA). Amplitudes of maximum acceleration were 818 and 332 gal, respectively, for the NS and UD components. Acceleration response spectra are shown in Fig. 11. Although the building site was 5.9 km distant from the observatory, the recorded motions were used in the analysis without modifications. Damping was assumed to be of a viscous type and proportionate to the instantaneous stiffness. A damping factor was assumed 3% with respect to the elastic fundamental frequency.

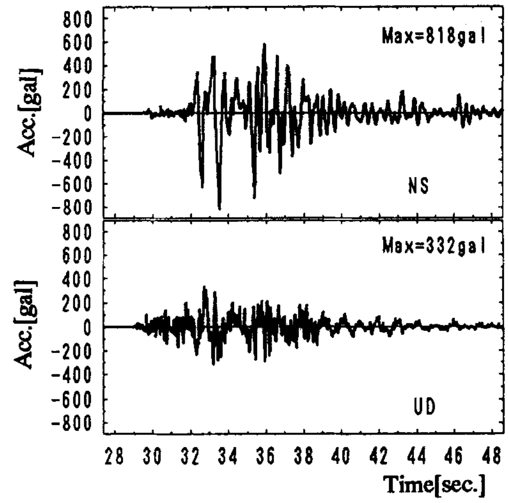

Fig. 10. Ground-motion records (JMA).

Comparison of analysis results with observations

Maximum interstory drift along the building height are plotted in Fig. 12 for Models 1 and 2. Almost all displacement concentrated on the first story for Model 1 with first story drift (FSD) of 31 cm (drift angle of 8.6%), while such extreme displacement concentration did not occur for Model 2 with FSD of 14 cm (drift angle of 3.8%), less than half of the value from Model 1. The hinge formation, though not

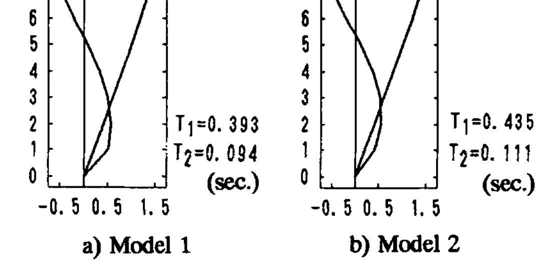

Fig. 9. Elastic periods and modes.

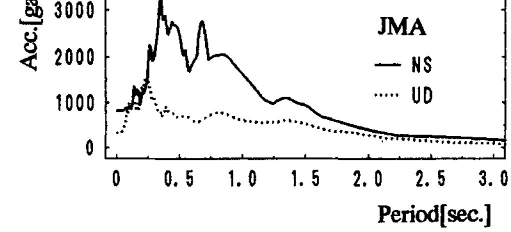

Fig. 11. Acceleration responce spectra.

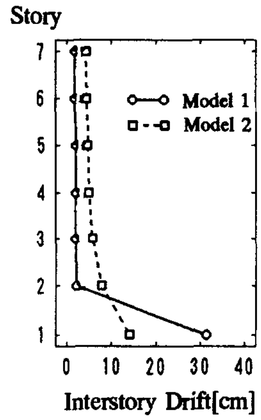

Fig. 12. Maximum interstory drift.

shown, was close to the static analysis results (Fig. 6): first story mechanism formed for Model 1, while definite mechanism did not form for Model 2. Comparison of the analysis results with the observations clearly indicates Model 1 assuming the slits as ineffective could reproduce the observed results well, while Model 2 assuming the slits as effective did not. In other words, if the slits had been effective, the collapse of the first story might have been avoidable. Problems on nonstructural walls are often discussed from a view point of their failure or their effect on adjoining structural members. However, it should be noted that there are cases like this building, where the existence of nonstructural walls may govern the overall behavior of buildings such as mechanism. Model 1, alone, is considered in the subsequent discussions.

Figures 13 and 14 show time histories of FSD and FSD–base shear relations. Shear force–shear strain relations of the first story wall and moment–rotation relations of the first story columns are also plotted in Figs 15 and 16.

In Fig. 13 (SD: -2%), after large amplitude vibration to the north at 33.8 s, FSD was

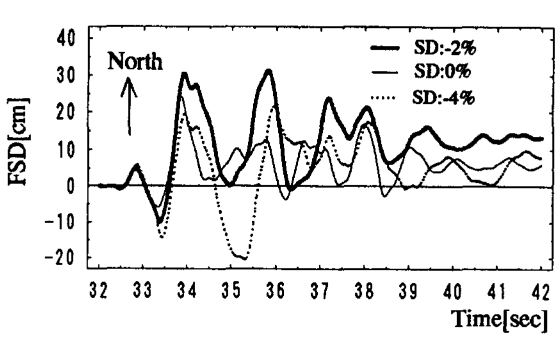

Fig. 13. Time histories of FSD.

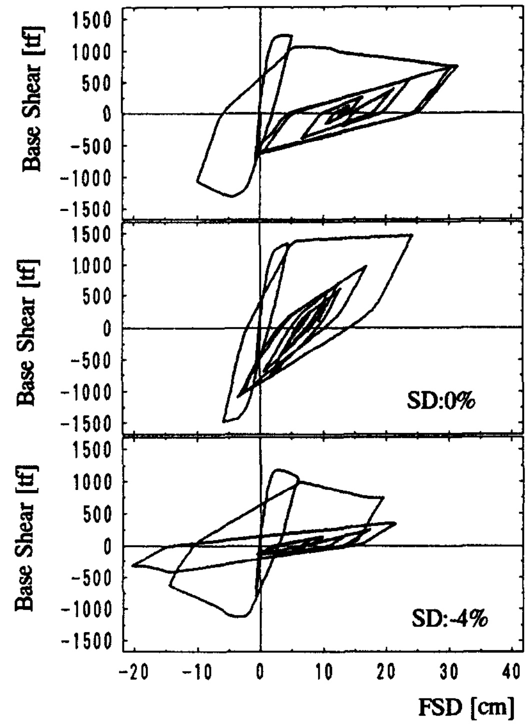

Fig. 14. FSD — base shear relations.

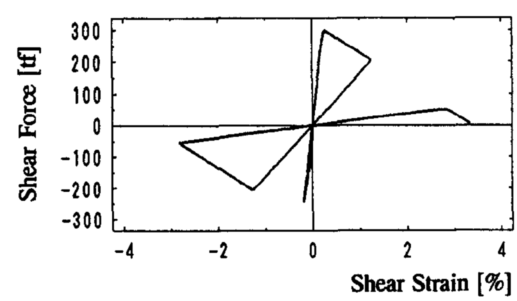

Fig. 15. Shear force – shear strain relations (first story wall).

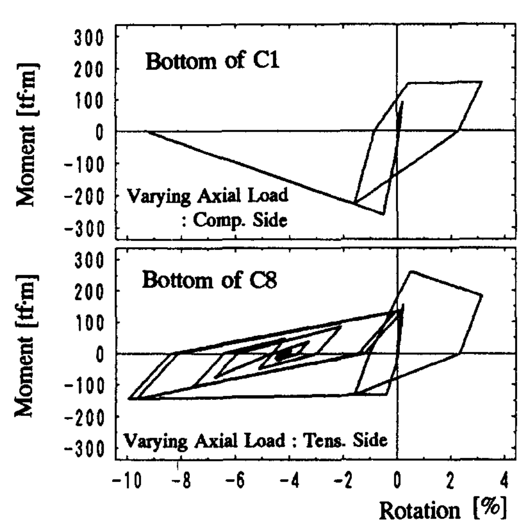

Fig. 16. Moment–rotation relations (first story columns).

observed to shift to this direction, causing residual displacement of 14 cm to the north at the end of the earthquake. Such result coincides with the observed residual displacement of 23 cm to the north, though the amplitude was a little different. The first story wall completely lost shear strength and C1 (north column), which was subjected to axial compression when displaced to the north, also completely collapsed. However, C8 (south column), which was subjected to axial tension when displaced to the north, did not lose flexure strength much. These analysis results on member behavior also agree with the observation stated in Section 2.2. As shown in Fig. 14 (SD: −2%), base shear reduced to 60% of the maximum value due to the member strength deterioration.

To study the influence of member strength deterioration, two additional cases were analyzed, where strength deterioration was not considered (SD: 0%) and the extent of it was doubled (SD: −4%). Results of these cases are shown in Figs 13 and 14.

For SD:0%, Maximum and residual values of FSD reduced to 24 cm (drift angle of 6.7%) and 7 cm, as compared to SD: −2% (Fig. 13), which was apparently because in this case strength deterioration was not considered, therefore, maximum strength of the building was maintained (Fig. 14). However, note even for SD:0% maximum drift angle reached 6.7%. It is practically impossible to provide members with deformability sustaining that amount of drift, suggesting if the first story mechanism might occur, the collapse could be unavoidable even for buildings with base shear strength as much as 0.62 W. For SD: −4%, because of large amplitude vibration to the south at 35.4 s, the extent of shift to the north became small, resulting in both maximum and residual values of FAD being lower than those from SD: −2% (Fig. 13). As suggested by Fig. 14 (SD: −4%), where base shear reduced to nearly zero for both directions, many columns such as C1 and C2 (north) and C8, C9 and C10 (south) completely lost flexure strength. The above discussions on the influence of member strength deterioration imply SD: −2% agree with the observations better than the other cases.

To study the influence of the level of ground motions, two additional cases were analyzed, where the level of ground motions was decreased and increased by 20%. These results are shown in Fig. 17 along with the original case. As compared to the original case, maximum value of FSD reduced to 16 cm for the case with a 20% decrease and residual value of FSD reduced to almost zero for the case with a 20% increase, which was due to the same reason as stated for SD: −4%. The original case seems to agree with the observations better than the other cases.

Axial compression of first story columns and effect of vertical motions

Maximum axial compression of the first story columns including permanent load are listed in Table 2 (Vert. mot. considered). The amplitudes of axial stress divided by specified concrete strength (axial stress index) reached as much as 0.42–0.56 for C1, C8, C9 and C10,

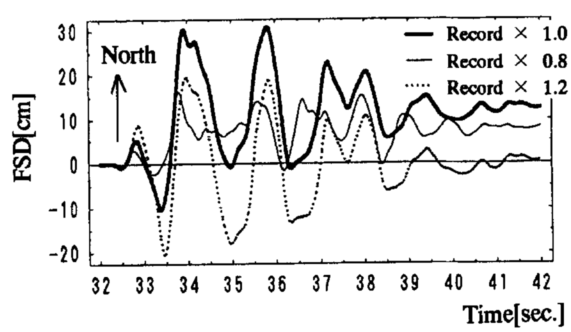

Fig. 17. Time histories of FSD.

Table 2. Maximum axial compression of first story columns [Unit:tf]

	C1	*C2*	*C4*	*C5*	*C7*	*C8*	*C9*	*C10*	*C11*
Vert. mot. Considered	716 (0.47)*	547 (0.36)	391 (0.52)	636 (0.40)	292 (0.17)	633 (0.42)	849 (0.56)	766 (0.50)	384 (0.25)
Vert. mot. Neglected	672 (0.44)	490 (0.32)	372 (0.49)	557 (0.37)	236 (0.13)	639 (0.42)	873 (0.58)	760 (0.50)	372 (0.25)
Permanent Load only	184 (0.12)	181 (0.12)	89 (0.12)	222 (0.15)	191 (0.12)	207 (0.14)	267 (0.18)	285 (0.19)	175 (0.12)

*Axial stress divided by concrete strength.

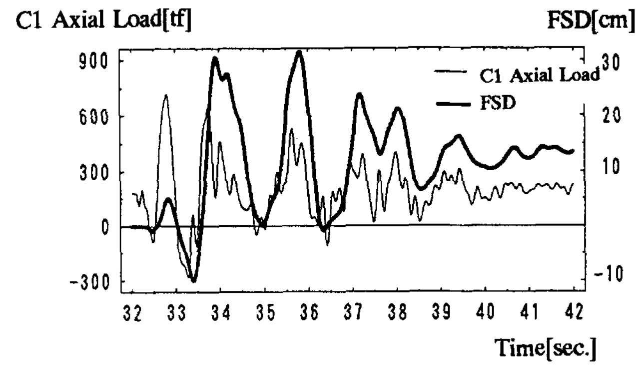

Fig. 18. Time histories of C1 axial load and FSD.

which were outer columns supporting walls. These amplitudes were much greater than those due to permanent load only. Figure 18 shows time histories of C1 axial load and FSD. C1 was computed to have collapsed near at 33.7 s, when this column underwent axial compression of 600 tf (axial stress index of 0.40) and displacement more than 20 cm. One should note that, although maximum axial compression occurred at 32.8 s, displacement at this time was very small. The north columns, including C1, were believed to have collapsed due to large lateral displacement combined with high axial compression.

Results for the case neglecting vertical motions (UD component) are shown in Table 2. The difference between the cases considering and neglecting vertical motions was very small: maximum difference was only 79 tf for C5. This was probably because vertical mass was assumed to be lumped at each beam–column joint and column axial stiffness was assumed elastic. Such assumptions might lead to the estimation of building vertical period being short (computed fundamental elastic period of 0.135 s) and as a result of it, the building was considered to have responded to vertical motions like a rigid body. For example, supposing the building vertical period was zero, maximum axial load of C5 due to vertical motions was evaluated as a product of permanent load (222 tf, Table 2) and maximum vertical acceleration (0.339 g, Fig. 10), resulting in 75 tf, which was very small and close to the aforementioned value of 79 tf.

CONCLUSIONS

The inelastic analysis of the building with soft first story collapsed by the 1995 Hyogoken–Nanbu earthquake was conducted, in which member strength deterioration was considered to simulate the process of collapse as realistically as possible. The major findings from the analysis are as follows:

(1) The observed behavior of the building, such as residual displacement, mechanism and damages to members were well simulated by the dynamic response analysis using the recorded ground-motions. The first story columns collapsed due to large lateral displacement combined with high axial compression.

(2) Nonstructural walls occasionally affect overall behavior of buildings such as mechanism. For this building, since the slits placed between the nonstructural walls provided above the second floor level and adjoining columns were ineffective, these nonstructural walls behaved as structural walls, causing the first story mechanism. If the slit had been effective, the collapse of the first story might have been avoidable.

(3) Within the limits of the studies presented herein, if the first story mechanism might occur, the collapse could be unavoidable even for buildings with base shear strength as much as 60% of the total weight.

REFERENCES

1. Takeda, T., Sozen, M. A. & Nielsesn, N. N., Reinforced concrete response to simulated earthquakes. *Journal of Structural Division, ASCE*, **96** (ST12) (1970) 2557–2573.
2. Hisada, T., Nakagawa, K. & Izumi, M., Earthquake response of structures having various restoring force characteristics. *Proceedings of Japan Earthquake Engineering Symposium*, November 1962, pp. 63–68.
3. Architectural Institute of Japan. *Ultimate Strength and Deformation Capacity of Buildings in Seismic Design*, October 1990 (in Japanese).
4. Hirosawa, M., Kitagawa, Y., Yamazaki, Y. & Teshigawara, M., Damages to Namioka municipal hospital due to 1983 Nihonkai–Chubu earthquake and its analysis. *Concrete Journal, Japan Concrete Institute*, **23** (12) (1985) 117–133 (in Japanese).