1082

SIMULATION OF COLLAPSE OF RC BUILDING SUBJECTED TO SEISMIC LOADS

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SUMMARY

In this paper, the effect of axial compression in beams, which is induced by restraint of their axial elongation, on failure mode of a reinforced concrete building was studied. The building suffered from severe damage due to the 1994 Sanriku-Haruka-Oki Earthquake. Most columns in the first story failed in shear and the first story collapsed, although the building was predicted to form a total yield mechanism of beam-yielding type according to conventional analyses. Static push-over analyses were carried out to simulate failure process of the building and to investigate the effect of the axial compression. Fairly general agreement in the failure mode between the observed damage and the analytical result in case that the axial elongation and the interaction between axial force and flexural moment (N-M interaction) in beams, which were generally ignored in conventional analyses, was obtained.

1. INTRODUCTION

On January 17, 1994, the Sanriku-Haruka-Oki Earthquake of magnitude 7.2 struck Tohoku area located in the northern part of Honshu Island, Japan. Several reinforced concrete buildings in the severely affected area suffered fromserious damage as described in Ref. [Joint Reconnaissance Team, 1994] in details. Major damage to Kanritoh-building of Hachinohe-higashi high school was shear failure in the first story columns, although the building was predicted to form total yield mechanism of beam-yielding type, according to conventional frame analyses [Mizobe and Kitayama, 1997]. In this paper, the focus was laid upon the effect of axial compression in beams, which was induced by restraint of their axial elongation, on failure mode of the building. Multi spring model was introduced into the analytical frame model to consider the axial elongation and the interaction between axial force and flexural moment (N-M interaction) in beams. Static push-over analyses were carried out to simulate failure process of the building and to investigate the effect of the axial compression in beams.

2. DESCRIPTION OF DAMAGED BUILDING

2.1 Summary of the Building

The structure investigated herein is Kanritoh-building of Hachinohe-higashi high school, a three-story reinforced concrete school building, and is located in Hachinohe City, Miyagi Prefecture. The building, designed according to the old seismic design codes and constructed in 1963, experienced the 1968 Tokachi-oki Earthquake of magnitude 7.9, and suffered moderate damage. After the earthquake, the building was repaired and had been used. **Figure 1**, **2** and **3** show floor plans, south elevation, and elevation view of structural frames in longitudinal direction, respectively. The structural type was one of old typical Japanese school type. The building has seven bays in longitudinal direction and one bay in transverse direction. Shear walls were generally placed between classrooms in the transverse direction except some frames.

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Figure 4 shows the reinforcing details in beams and columns. The cross section of a column was 80cm x 50cm in Y1 frame or 100cm x 50cm in Y2 frame. Confinement of the columns was relatively poor because the spacing of lateral reinforcement was 24cm, which is required 10cm or less in the present seismic design codes. The material properties obtained from sample tests are listed in Table 1 [Mizobe and Kitayama, 1997]. The basement was independent footing foundation and was supported by loam layer.

2.2 Damages to the Building

The damage levels of structural members were classified, as shown in **Figure 1**, according to the "Standard for Damage Level Classification of Reinforced Concrete Buildings [JBDPA, 1991]" which is issued from the Japan Building Disaster Prevention Association. In the Standard, the damage level of a structural member is categorized into five classes; The "damage class 1" corresponds to slight damage and The "damage class 5" the severest damage such as shear failure, buckling of reinforcing bars, crush of concrete and so on.

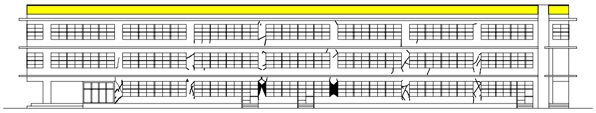


Figure 2: The Southern Elevation of Kanritoh-Building of Hachinohe-Higashi High School

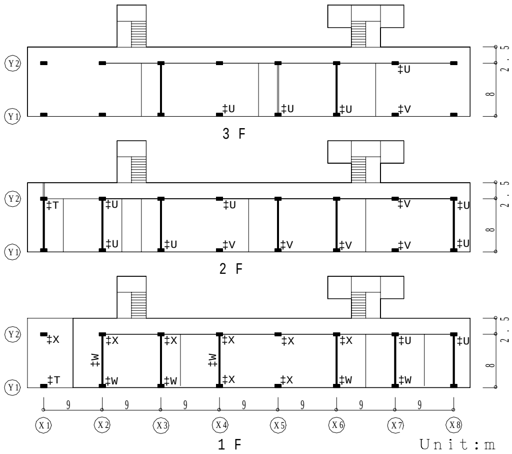


Figure 1: Floor Plans of Kanritoh-Building of Hachinohe-Higashi High School

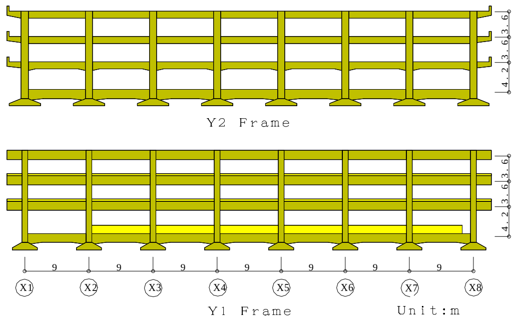
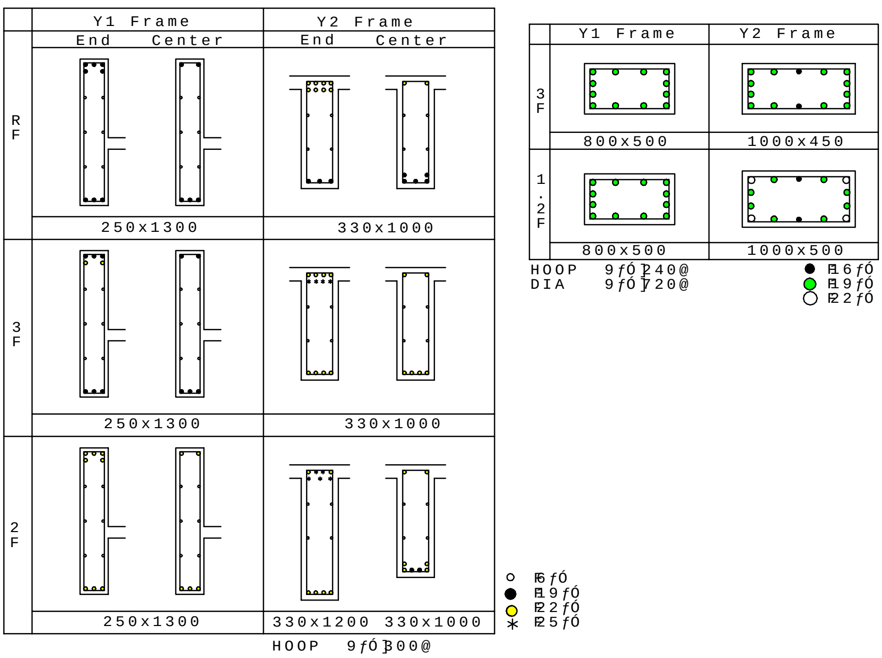


Figure 3: Elevation View of Structural Frames in Longitudinal Direction



(a) Beam **(b) Column**

Figure 4: Reinforcing Details of Members

Cracking patterns were illustrated in the south elevation of **Figure 2**.

Major damage to the building was generally observed in the columns of the first story, as described in Ref. [Joint Reconnaissance Team, 1994] in details. Extensive shear cracks, buckling of longitudinal reinforcement and crush of concrete were observed in the first story columns. Moreover, shear walls in X2 and X4 frames failed in shear as shown in **Figure 1**. Approximately 10cm subsidence of the upper floor was found in X5 frame due to absence of shear wall. According to the Standard for Damage Classification [JBDPA, 1991], damage level of the first story was categorized into "collapse." In the second and third story, the damage was minor compared to the first story, although shear cracks were observed in several columns. Flexural cracks at the critical section were found in some beams, even though, damage to the beams was generally slight.

3. SEISMIC CAPACITY EVALUATION OF DAMAGED BUILDING

To understand the correlation between observed damage and the seismic capacity of the building, seismic evaluation was carried out. In the seismic evaluation of the building, the Japanese "Standard for Seismic Capacity Evaluation of Existing Reinforced Concrete Buildings [JBDPA, 1990]" was applied.

3.1 Basic Concept of the Standard

The Standard evaluates the seismic capacity at each story and in each direction of the building by the following index;

$$I_s = E_0 \cdot S_D \cdot T \qquad (1)$$

where,

E_0 = basic structural index calculated by ultimate horizontal strength, ductility, number of stories and story level concerned.

S_D = Structural design index to modify the E_0-index due to the grade of the irregularity of the building shape and distribution of stiffness along the height.

T = time index to modify the E_0-index due to the deterioration of strength and ductility.

The standard values of the S_D- and T-indices are 1.0. The E_0-index for the single structural system can be expressed by the product of the ultimate horizontal strength index in terms of story shear coefficient (C), ductility index (F) and story index ϕ. Story index (ϕ) at the first floor level is 1.0. Therefore, the E_0-index at the first floor level of the simple structure can be defined as;

$$E_0 = C \cdot F \qquad (2)$$

In evaluating F-index in Eq. (2), the shear-span-to-depth ratio, flexural strength, shear strength etc. are considered. F = 1.0 for brittle (shear failure type) members and F = 1.27 to 3.2 for ductile (flexural failure type) members in the Standard.

3.2 Assumptions in Seismic Evaluation

To evaluate the seismic capacity of the building, the following assumptions were employed.

1) Unit weight of each floor was assumed 1.07 ton/m^2 based on the calculation in the original design.
2) *T*-index was assumed 1.0.
3) The strength of concrete and reinforcing bars were assumed equal to material properties obtained from the sample tests. (**Table 1**)
4) In the third level procedure, the effective flange width of floor slab for T-shape beam in Y2 frame was assumed 0.2h (h: clear span length of beams).

Table 1: Material Properties Based on Sampling Test [Mizobe and Kitayama, 1997]

Compressive Strength of Concrete	18.4 MPa
Young's Modulus of Concrete	16.8 Gpa
Yield Strength of Column Reinforcement	333 Mpa
Yield Strength of Beam Reinforcement	304 Mpa
Yield Strength of Hoop	334 Mpa

Table 2: Results of the Seismic Evaluation in Longitudinal Direction

Procedure	Story	E_0	S_D	T	I_S	$C_T S_D$
2nd level	3	0.74	0.98	1.0	0.73	0.57
	2	0.41			0.40	0.40
	1	0.42			0.41	0.41
3rd level	3	0.71			0.70	0.51
	2	0.84			0.82	0.28
	1	0.84			0.82	0.19

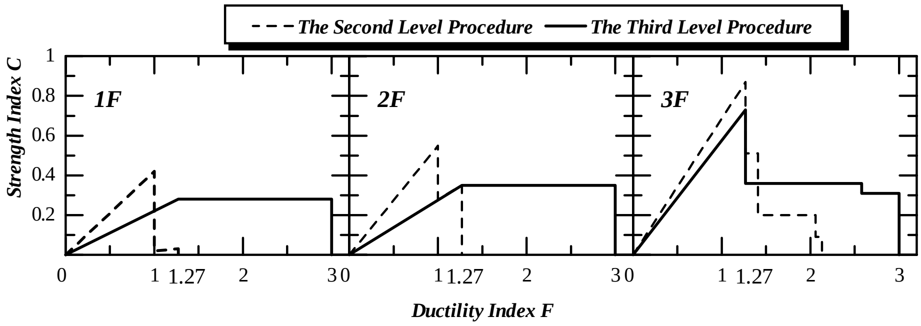


Figure 5: Strength Indices *C* vs. Ductility Indices *F*

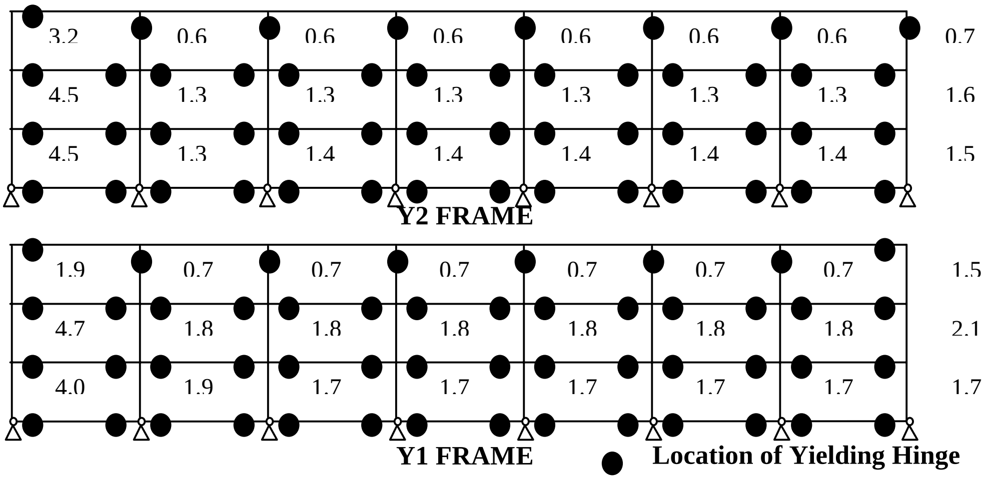


Figure 6: Predicted Locations of Yield Hinges in the Third Level Procedure and Column-to-Beam Strength Ratio

3.3 Evaluation Results

The seismic capacity in the longitudinal direction was evaluated according to the Standard for Evaluation. In this study, the second and third level procedures were adopted. Evaluation results in the longitudinal direction were summarized in **Table 2**. **Figure 5** shows the relationship between the strength index (C) and the ductility index (F).

In the second level procedure, beams are assumed to be infinitely stiff and strong to enable analysis to be made simple, since much damage has been reported in vertical members. Seismic performance index *Is* was therefore evaluated based on only vertical members such as columns and walls. Almost all the columns in first story were estimated as shear type (F=1.0) as can be seen in **Figure 5**. The columns in the second story consisted of shear types and ductile flexural types (F=1.27). F-index of the columns in the third story ranged from 1.27 to 2.2. *Is*-Index was significantly lower than 0.6 in the longitudinal direction of the first and the second stories. *Is*-Index of 0.6 is recommended for the judging criterion to prevent from serious damage in the Standard for Evaluation based on the study from the past earthquake damage. The insufficient *Is* value of 0.41 and brittle shear type columns in the first story corresponded to observed damage.

Flexural and shear strengths in beams, in addition to those in the vertical members, are also considered in the third level procedure, to evaluate more reasonable and accurate seismic performance indices *Is* especially for beam-yielding type structures. **Figure 6** shows the location of yield hinges predicted by the calculation and column-to-beam nodal moment ratio (the sum of column nodal moments divided by the sum of column nodal moments). As can be seen in the figure, flexural yielding at the beam ends was predicted to precede shear failure and flexural yielding in the columns, since column-to-beam nodal moment ratio was larger than 1.0 at the second and third floor levels. In this case, the columns are defined as the *column governed by flexural beams*, of which *F*-Index is 3.0, according to the Standard for Evaluation. As a result, the building attained larger *Is* values for the first and second stories than in the second level procedure, although the lateral strength index $C_T \cdot S_D$ was lower. The larger *Is* values, which were approximately twice as those in the second level procedure and were larger than the criterion of 0.6, disagreed with the observed damage. As mentioned earlier, the criterion of *Is*=0.6 is recommended to prevent serious damage in the Standard for Evaluation. This overestimation of the seismic performance resulting from mistaking of failure mode of the structure is fatal to the future works of seismic retrofit to screen seismically vulnerable buildings.

4. FRAME ANALYSIS CONSIDERING VARIABLE AXIAL LOAD OF BEAM

As has been stated above, in the third level seismic capacity evaluation, disagreement in the failure mode of the structure between the calculation and the damage investigation was found. The possible reasons for such disagreement may be attributed to; (1) mode shape during the earthquake different from those assumed in the Standard, which is based on the fundamental mode shape (dynamic effect), (2) deterioration of shear strength in the column due to axial tension induced by the input into the vertical and transverse direction, and so on. Another reason is the effect of the varying axial compression in beams induced by the restraint of their elongation, on which the focus in this paper was laid. In general, a reinforced concrete member tends to elongate after flexural cracking and/or yielding of tensile longitudinal bars. The axial elongation is restrained by surrounding structural members such as connecting columns, shear walls and floor slabs, especially in multi bay frame structure. The restraint of the axial elongation provokes increase in flexural yielding strength of beams [Wada and Hayashi et al., 1990]. In the following section, frame analyses, considering the axial elongation effect, were performed to simulate the behavior of the building.

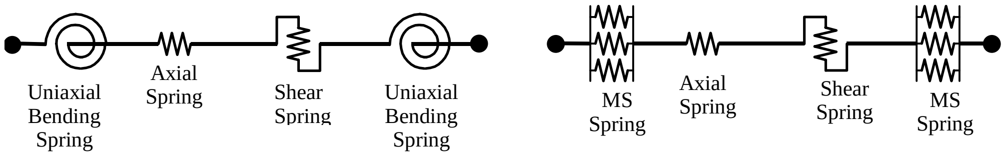
4.1 Analytical Model of the Building

Two frame models were introduced to simulate the behavior of the building in longitudinal direction. Beams and columns were idealized by the one component model, which consisted of bending spring at its end, shear spring and axial spring at its mid-span, as shown in **Figure 7**. Each spring was connected by linear element. The frame was assumed to be fixed on pin supports at the first story column base. The beam-column joint was assumed to be rigid and represented by rigid zones, and the rigid zone length was supposed from rigid joint to critical section. The mass was concentrated at each node. A general purpose computer program CANNY-E, developed by Dr. Kang-Ning Li [Li, 1996], was employed in the calculation.

To investigate the effect of the axial elongation in beams on the failure mode of the structure, two cases were studied; In Case1, axial elongation in beams, which is generally neglected in conventional analyses, was not taken into consideration and rigid floor slab was assumed. An uni-axial bending spring model was introduced into beams (**Figure 7(a)**). In Case2, multi spring model (MS model) [Li, 1996] was employed to consider the axial elongation and the N-M interaction in beams (**Figure 7(b)**). MS model was also used for bending spring of the columns in both cases. The MS model consists of concrete and steel longitudinal spring elements as shown in **Figure 8**.

Hysteresis rules for the concrete and steel elements of the MS model were shown in **Figure 9** and in **Figure 10**, respectively. The length of the MS spring elements was assumed D/2 (D: depth of the member) so that the initial stiffness of the MS spring without axial force agreed with that of the uni-axial spring. The parameter λ, which represents deterioration of concrete after reaching compressive strength *Fc*, was assumed 0.9 for core concrete and 0.7 for shell concrete. Stiffness degradation was considered for steel spring, as shown in **Figure 10**, in order to represent gradual extension of plastic hinge region after flexural cracking. Displacement of steel spring at the

yielding point was selected $3.0\,d_{sy}$ so that the yielding rotation of the MS spring without axial loads agreed with that of the uni-axial spring.
Takeda model [Takeda, Sozen and Nielsen, 1970] was used for hysteresis rule of the uni-axial bending spring and shear spring. Axial stiffness model [Kabeyasawa et al., 1983] was used for the axial spring. Resistance and deformation at the cracking and yielding point in the hysteresis rule were determined based on structural geometry and material properties of each member, according to the "Standard for Structural Calculation of Reinforced Concrete Structures [AIJ, 1991]."



(a) Beams in Case1 **(b) Beams in Case2 and Columns**

Figure 7: Idealization of Members

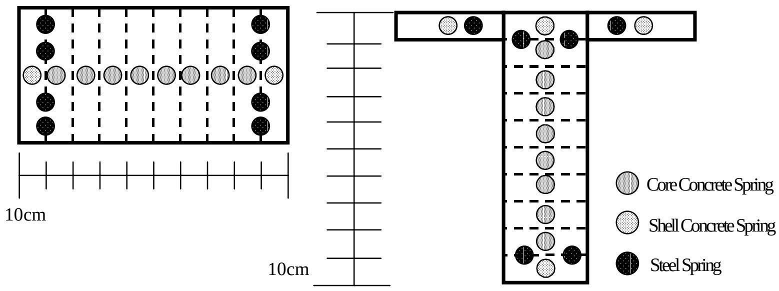
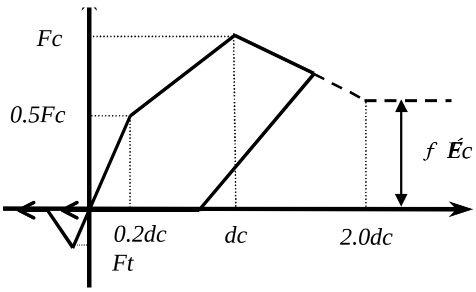


Figure 8: Location of Spring Element in MS Model



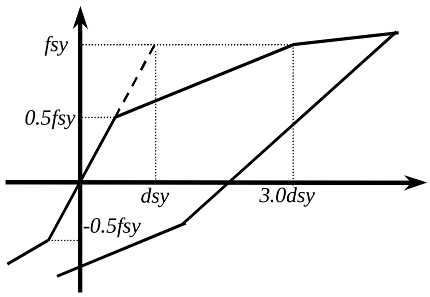


Figure 9: Hysteresis Rule of Concrete Spring Figure 10 : Hysteresis Rule of Steel Spring

4.2 Static Pushover Analysis

Static step-by-step pushover analyses were carried out for both cases. **Figure 11** is showing relationships between story shear coefficient and drift angle. The location of flexural yielding and shear failure in Y2 frame at the first story drift angle of 1/300rad. and 1/100rad. was shown in **Figure 12**.
In Case1, in which axial the elongation and the N-M interaction in beams were neglected, flexural yielding occurred at the bottom of a few second floor beam at the story drift angle of 1/500rad. Then the yield hinges developed at most ends of second and third floor beams at the story drift angle of approximately 1/300rad. Finally total yielding mechanism of beam-yielding type was formed as can be seen in **Figure 12(a)**. Shear failure in columns was never observed until the story drift angle exceeded 1/100rad. As mentioned above, the failure mode of the frame predicted by the analysis in Case1 did not agreed with that observed at the site (i.e., shear failure in columns in the first story was major damage) but corresponded with that estimated by the third level procedure of the seismic evaluation.

Figure 13 indicates the distribution of axial force ratio (axial stress normalized by concrete compressive strength) in each floor for Case2, in which axial elongation, varying axial force and N-M interaction in beams were considered. Flexural yielding occurred at the bottom of a few second story beams at the story drift angle of 1/500rad. The axial elongation in beams occurred after cracking of concrete spring and then axial force was

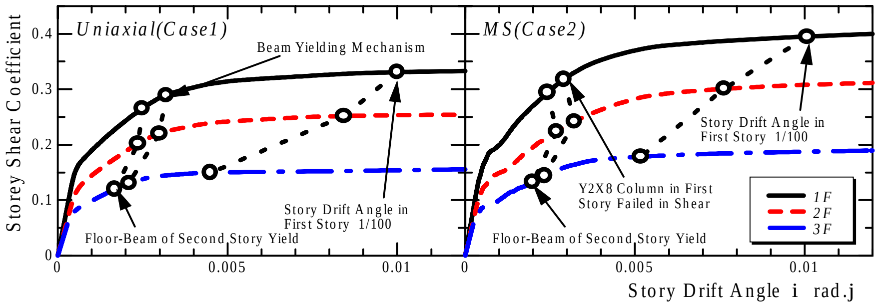
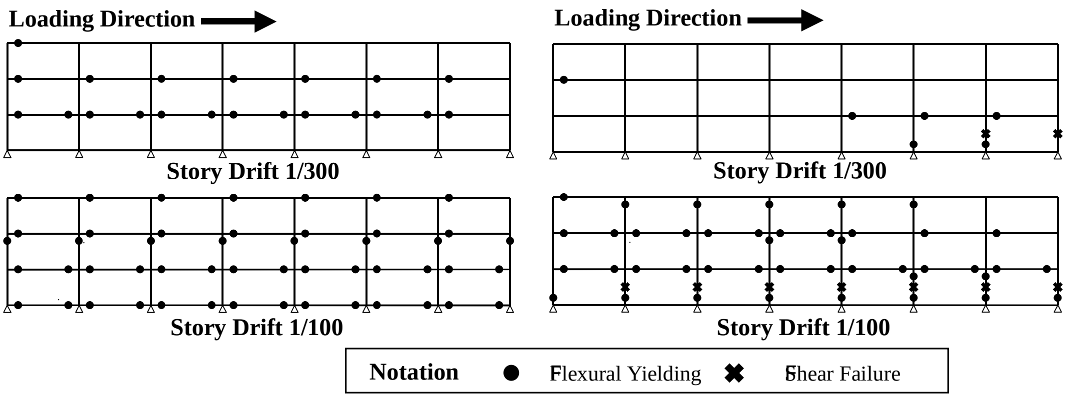


Figure 11: Story Shear vs. Story Drift Angle



(a) Case1 (Uniaxial Bending Spring) (b) Case2 (MS Spring)

Figure 12 : Location of Yield Hinge and Shear Failure in Y2 Frame

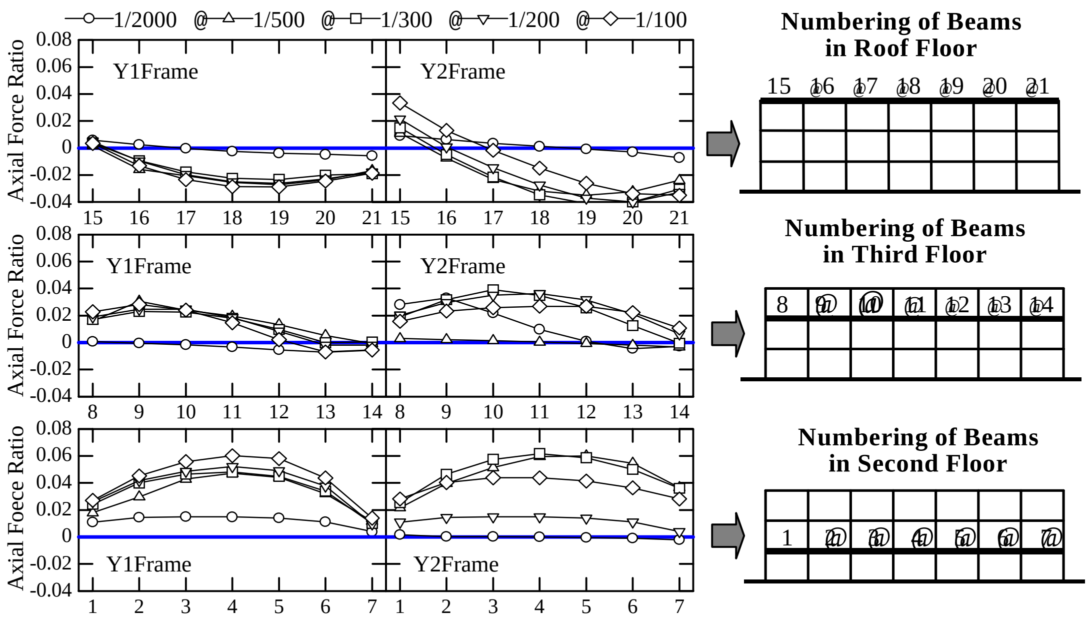


Figure 13: Axial Force Ratio in Beams

induced gradually to beams at the second and third floor levels. Axial compressive force ratio ranged from approximately 0.02 to 0.05 in the second floor level at the drift angle of 1/300rad. Higher restraint induced higher axial compression for interior beams. The axial compression caused increase in flexural yielding moments of the beams. At the story drift angle of 1/300rad., X8 column in the first story of Y2 frame failed in shear, although, the number of the yield hinge was less than that in Case1 as shown in **Figure 12(b)**. Flexural yielding moments of beams in the second floor level were approximately 1.2 to 1.4 times as those without axial compression. Shear failure in the X8 column preceded failures in the other columns since additional shear force was induced due to the reaction of the axial compression from connecting beam. Then, shear failure extended from X8 column to X1 column. Finally, most columns in the first story failed in shear at the story drift angle of 1/100rad, as can be seen in **Figure 12(b)**.

As described above, there was fairly general correspondence in the failure mode of the frame between the observed damage and the analytical result in which the axial elongation and the N-M interaction in beams were considered. Therefore, one of the possible reasons why the damage to the investigated building centered on the first story columns may be attributed to the increase in the beam yielding strength caused by the restraint of the beam axial elongation.

5. CONCLUSIONS

Seismic performance of a RC school building damaged due to the 1994 Sanriku-Haruka-Oki Earthquake was investigated and the effect of the axial elongation in beams on the behavior of the building was discussed. Major findings in this paper can be summarized as follows.

1) Conventional analyses, in which the axial elongation and compression in beams were neglected, estimated the failure mode of the frame different from observed damage, and overestimated the seismic capacity of the investigated building.
2) Fairly general correspondence in the failure mode of the frame between the observed damage and the analytical results, in case that the axial elongation and the N-M interaction in beams were considered, was found.
3) One of the possible reasons why the damage to the investigated building centered on the first story columns may be attributed to the increase in the beam yielding strength caused by the restraint of the beam axial elongation.

6. ACKNOWLEDGEMENTS

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