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SEISMIC COLLAPSE TEST OF A FULL-SCALE 4-STORY STEEL FRAME PART 1 - TEST RESULTS

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ABSTRACT

A shaking table test on a full-scale steel building was conducted at the E-Defense three-dimensional shake table facility to evaluate structural and functional performance of the building under design-level ground motions and the safety margin against collapse under exceedingly large ground motions. The specimen is a 4-story moment resisting frame designed and constructed according to the current design specifications and practice, and is attached with non-structural components. This paper describes results concerning the linear and nonlinear responses of the specimen building subjected to 20%, 40%, 60%, and 100% scaled JR Takatori records of 1995 Hyogoken-Nanbu earthquake, respectively. The detailed behavior of the deteriorated structural members and the safety margin of steel moment frames designed in current seismic code against complete collapse are discussed.

Introduction

A full-scale experiment with the objectives to evaluate structural and functional performance of the steel building under design-level ground motions and under exceedingly large ground motions is conducted. This test is a part of the experimental project on steel buildings conducted at the E-Defense shake-table facility. The overview of the project is presented in Kasai et al. (2007). The building specimen was designed following the current Japanese specifications and practices (post 1995 Kobe earthquake). Due to recently adopted improvements, there is little likelihood that moment connections would fracture even under exceedingly large ground motions. However, strain hardening in the beam plastic hinges could increase story shear forces, which in turn, would increase the forces developed in the columns. If the columns are not designed for the increased forces, i.e., if the width-to-thickness ratio of the cross-section is not small enough to develop the increased forces, then local buckling could occur in the columns. Strength deterioration in the lower-story columns could shift the controlling mechanism of the frame from the overall sway mechanism to a weak story collapse mechanism. Based on these and detailed analytical investigations by Tada et al. (2007), weak story collapse mechanism due to deterioration in column strength was identified to be the most likely scenario for collapse of a moment frame constructed according to the current Japanese seismic code. Therefore, the building specimen was expected to show this type of collapse mechanism.

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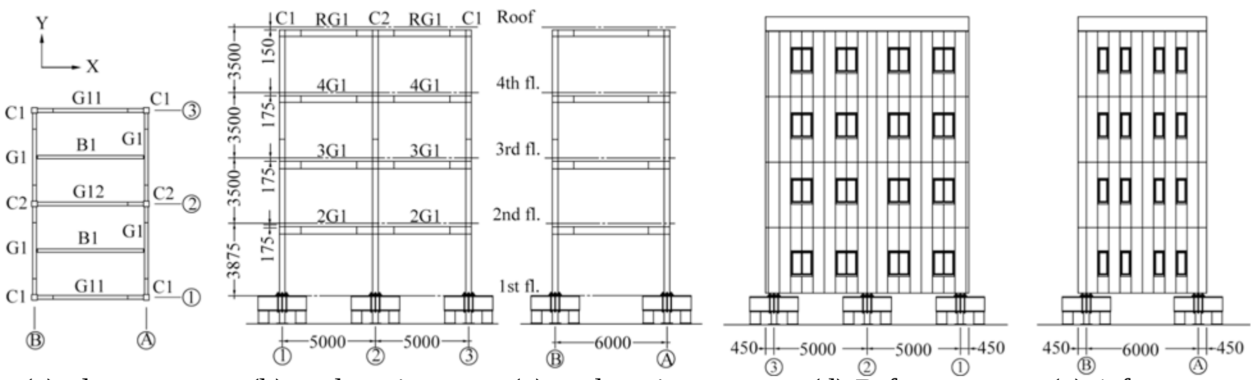
Specimen and Experimental Program

The test structure was a four-story, two-bay by one-bay steel moment frame as shown in Fig. 1, having plan dimensions of 10 m in the longitudinal direction (Y) by 6.0 m in the transverse direction (X). Each story is 3.5 m high, making the overall story height equal to about 14 m. The columns were made of cold-formed square-tubes, beams were made of hot-rolled wide-flanges and through diaphragm connection details were adopted in which short brackets were shop-welded to the columns. Table 1 shows sections of structural members. The wide flange beams ranged from 340 to 400 mm deep, and columns were of square hollow sections of 300 mm wide. Table 2 shows material properties of steel members obtained from coupon tests. The nominal steel strength is 235 and 295 N/mm^2 for the beam and column, respectively. The measured yield strength of columns was rather lower than average actual strength, and the yield strength of wide-flanges for beams are fairly larger than specified values.



(a) plan (b) y-elevation (c) x-elevation (d) B-frame (e) 1-frame

Figure 1. Framing and elevation of specimen building (unit: mm)

Table 1. List of member sections

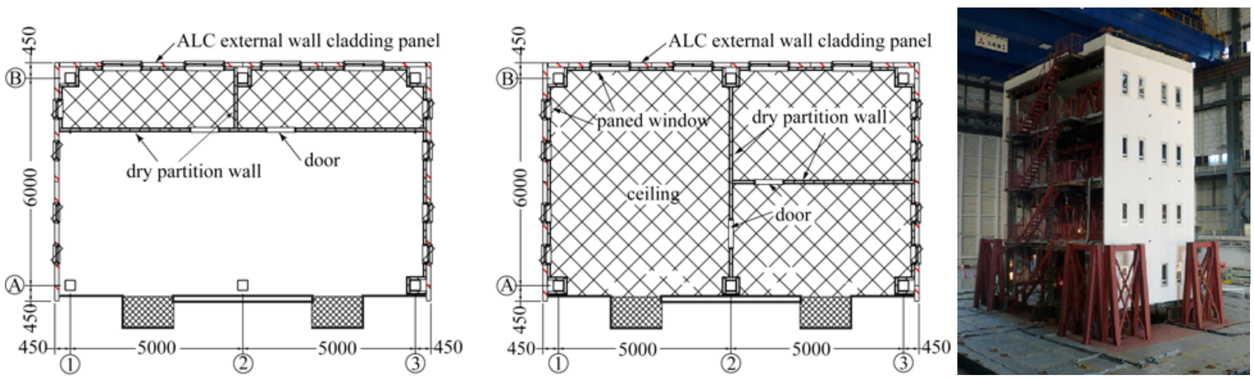
Story	Beam			Column
	G1 (SN400B)	G11 (SN400B)	G12 (SN400B)	C1,C2 (BCR295)
4	H-346x174x6x9	H-346x174x6x9	H-346x174x6x9	SHS-300x300x9
3	H-350x175x7x11	H-350x175x7x11	H-340x175x9x14	SHS-300x300x9
2	H-396x199x7x11	H-400x200x8x13	H-400x200x8x13	SHS-300x300x9
1	H-400x200x8x13	H-400x200x8x13	H-390x200x10x16	SHS-300x300x9

Table 2. Mechanical properties of steel materials

Section	Specified properties		Flange		Web	
	Yield point	Tensile strength	Yield point	Tensile strength	Yield point	Tensile strength
H-340x175x9x14	235	400	309	443	355	468
H-346x174x6x9	235	400	333	461	382	483
H-350x175x7x11	235	400	302	441	357	466
H-390x200x10x16	235	400	297	451	317	458
H-396x199x7x11	235	400	311	460	369	486
H-400x200x8x13	235	400	326	454	373	482
SHS-300x300x9 *	295	400	330	426	-	-
SHS-300x300x9 **	295	400	332	419	-	-

unit: N/mm^2 (* 1st story, ** 2nd – 4th story)

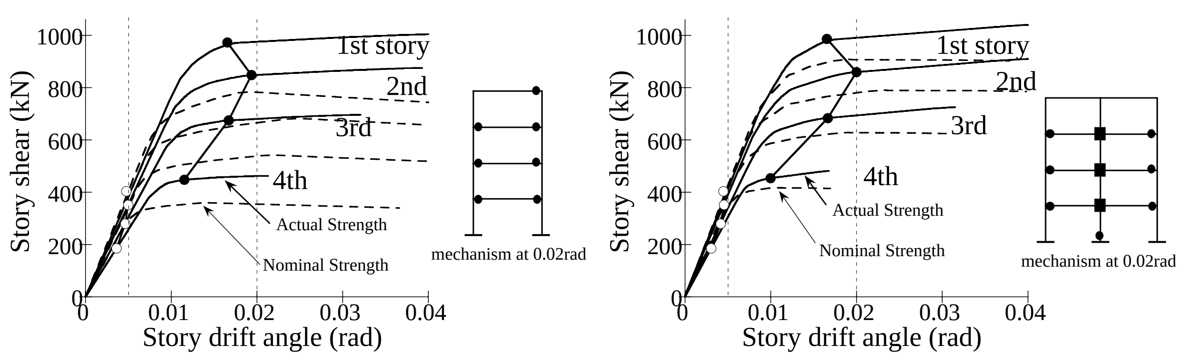
External wall cladding panels of ALC were placed on three sides of the frame as shown in Fig. 1 and 2. The specimen was attached with typical non-structural components used for a steel building, i.e., interior dry partition walls, internal walls and ceilings of gypsum boards were attached through metal-stud framings, and windows and doors are placed on external and partition walls as shown in Fig. 2.



(a) 2nd and 3rd story (b) 4th story (c) overview

Figure 2. Plan and non-structural components (unit: mm)

The lateral resistance of each story satisfies strength and stiffness requirements of seismic codes and the overall sway mechanism at the ultimate state was verified by static pushover elasto-plastic analysis as shown in Fig. 3 in which the dotted lines show results based on the nominal material strength and thick lines show results from the actual material strength obtained from coupon tests. However, strain hardening in the beam plastic hinges could increase story shear forces, which in turn, would increase the forces developed in the columns. If the columns are not designed for the increased forces, i.e., if the width-to-thickness ratio of the cross-section is not small enough to develop the increased forces, then local buckling could occur in the columns. Strength deterioration in the lower-story columns could shift the controlling mechanism of the frame from the overall sway mechanism to a weak story collapse mechanism. Based on these and detailed analytical investigations by Tada et al. (2007), weak story collapse mechanism due to deterioration in column strength was identified to be the most likely scenario for collapse of a moment frame constructed according to the current Japanese seismic code. Therefore, the building specimen was expected to show this type of collapse mechanism.



(a) x-direction (transverse) (b) y-direction (longitudinal)

Figure 3. Story shear versus story drift relationships obtained from pushover analyses

The shake tests were conducted at the E-Defense in September 2007. The specimen was subjected to motions as recorded during the 1995 Kobe earthquake at the JR Takatori train station. The test consisted of repeated application of the records with progressively increasing scale factors from 0.05 to 1.0 as shown in Table 3. This paper focuses on test results by 0.4, 0.6 and 1.0 times Takatori records in which the specimen responded in inelastic manner.

Table 3. Earthquake Scale Factor and Summary of Response

Scale Factor	Building Response
0.05	Linear elastic behavior
0.2	No yielding in steel structural elements. Peak story drift angle less than 0.005. Equivalent to a Japan Level 1 design earthquake (PGV=0.25 m/s).
0.4	Slight yielding. Peak story drift angle about 0.01. Equivalent to a Level 2 earthquake (PGV=0.5m/s).
0.6	Yielding. Peak story drift angle about 0.02 with residual drift ratio about 0.003.
1.0	Collapse in the 1st story.

Inelastic Response By Level 2 Earthquake

Response of Story

As the records equivalent to a Level 2 design earthquake (peak ground velocity is 0.5m/s), 0.4 times Takatori records are used. The shear force and story drift angle relationships in X and Y direction are shown in Fig. 4. The story shear force is estimated as the inertial force obtained from the acceleration record on each floor times mass of the story. Therefore, it equivalents to the sum of restoring forces and damping forces relate to the whole structural and non-structural components of the story. The peak story drift angle is 0.0114 rad at the 1st story and obviously yielded at 1st and 2nd story. The largest inelastic behaviors were observed at panel zones.

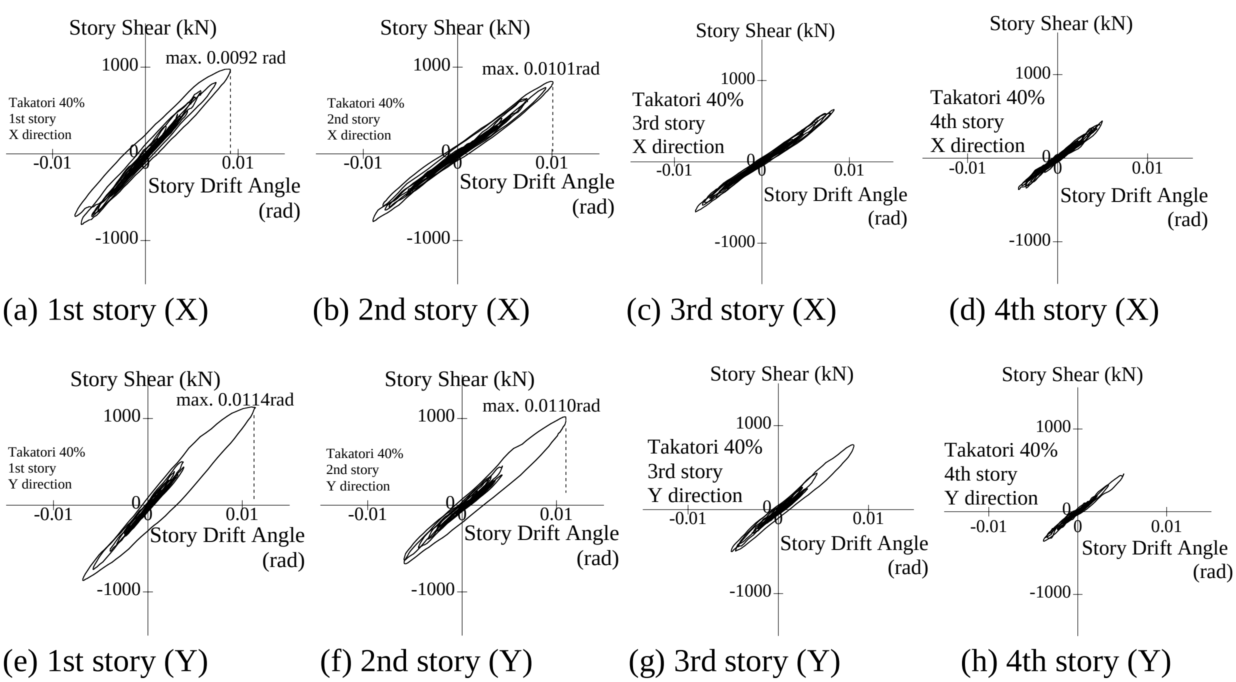
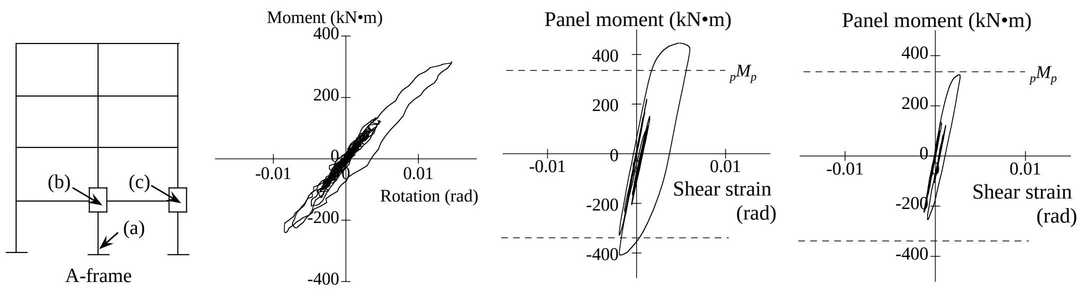


Figure 4. Story shear force and interstory drift relationships by 0.4 Takatori records

Behavior of Structural Members

Fig. 5 shows response of primary structural members in Y-direction at the 1st and 2nd story. In these figures, the rotation is defined between the inflection point and the end of the member, and the panel moment is defined as shear force of the panel multiplied by the depth. All columns of the first story are slightly yielded, as shown in Fig. 5(a), at the base and remained in elastic at the top side. All beams behaved in elastic manner because of composite action with concrete slabs and remarkably large actual yield strength of beams compared with other structural members. The primary plastic deformation is observed in the panels of the center columns at the 2nd and 3rd floor level as shown in Fig. 5(b). At the side columns, panels are also yielded but panel moments are slightly less than the full plastic strength. Both in X and Y direction, the frame behaved in a overall sway mechanism as intended in the design



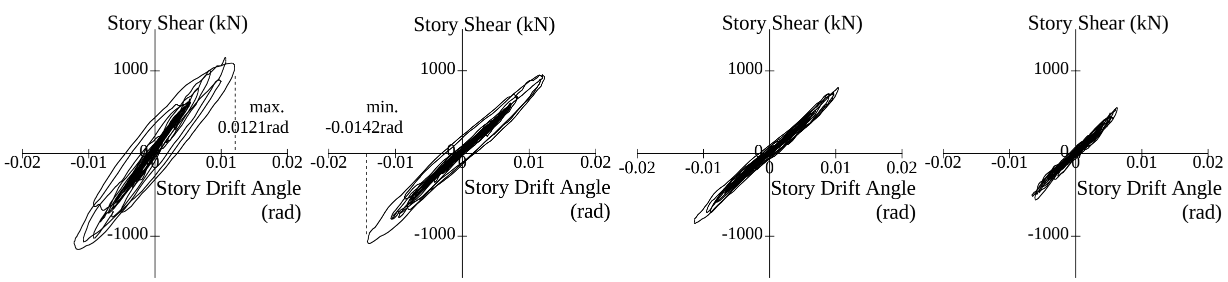
(a) Column at the base (b) Panel of center column (c) Panel of side column

Figure 5. Hysteresis behavior of column and panel by 0.4 Ta-katori records

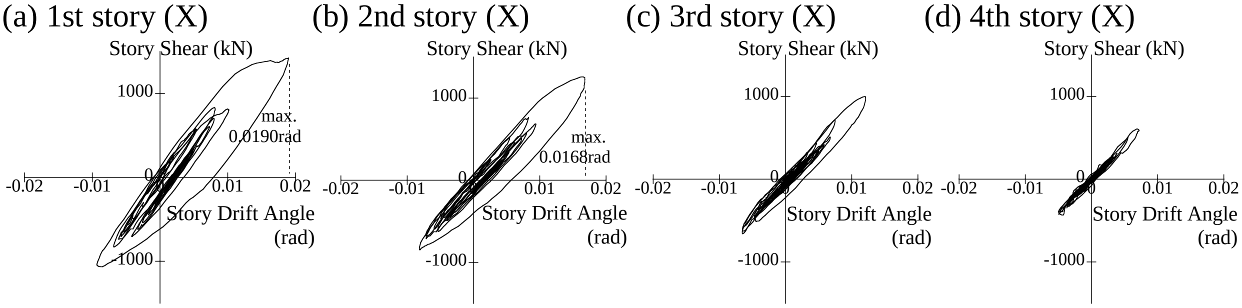
Inelastic Response By Over Level 2 Earthquake

Response of Story

Fig. 6 shows the story shear force and interstory drift relationships by 0.6 times Takatori records. The peak ground velocity was 0.75m/s, 1.5 times larger than the Level 2 earthquakes.



(a) 1st story (X) (b) 2nd story (X) (c) 3rd story (X) (d) 4th story (X)

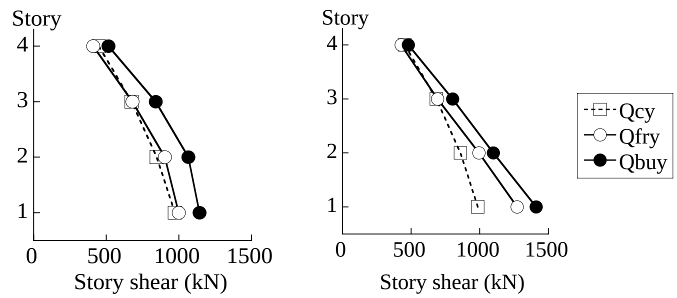


(e) 1st story (Y) (f) 2nd story (Y) (g) 3rd story (Y) (h) 4th story (Y)

Figure 6. Story shear force and interstory drift relationships by 0.6 Takatori records

The peak story drift angle increased to 0.019 at 1st story in Y-direction and inelastic hysteresis relationships were observed in the 1st to 3rd stories, which means that the overall sway mechanism proceeded. However, from the comparison between Fig. 4(a) and Fig. 6(a), the peak story shear at the 1st story increased about 1.3 times larger than the response by the 0.4 times Takatori.

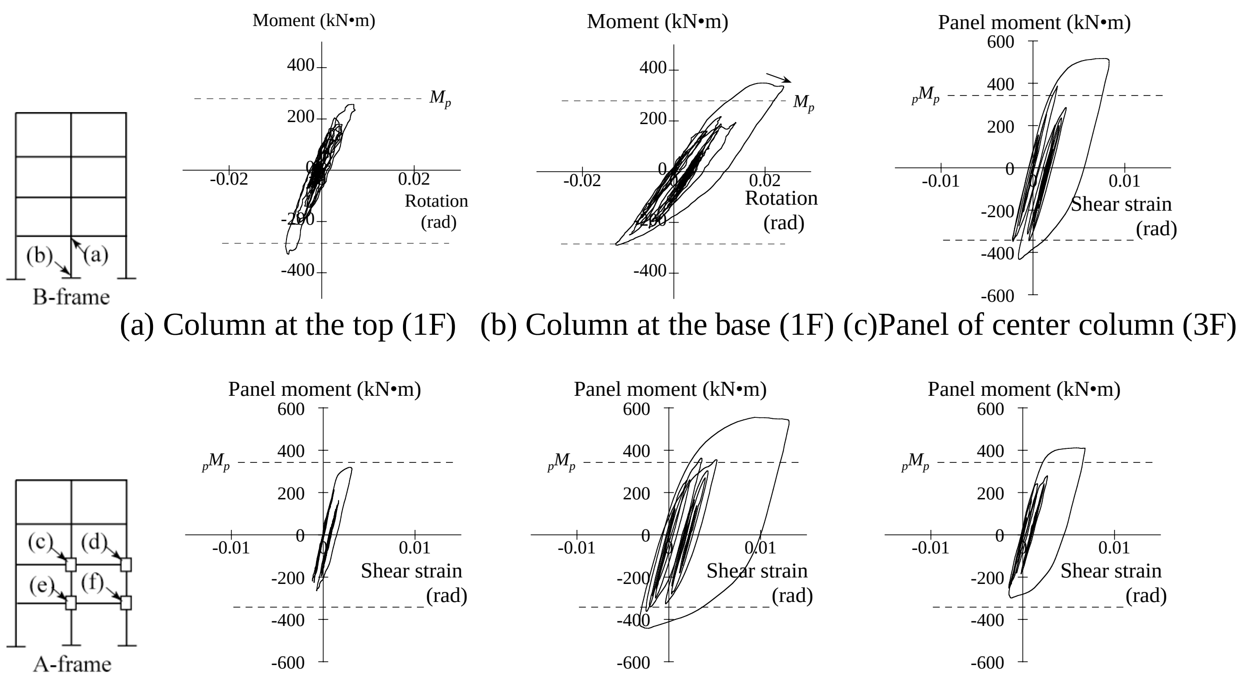
Fig. 7 shows peak story shear forces by 0.6 times Takatori. The open circles show story shear forces carried by the steel frame only and these values correspond to the results obtained from push-over analysis at 0.02 rad drift angle. The solid circles indicate the inertial forces which correspond to the shear forces carried by a whole building including non-structural components. The peak story shear force at the 1st story increased about 1.25 times larger than the response by 0.4 Takatori records in Y direction. The lateral strength of the 1st story reaches its maximum limit and small degradation of the strength is observed as shown in Fig. 6(e). These results indicate that the frame attained its ultimate strength level by the over-all sway mechanism.



(a) X-direction (b) Y-direction

Figure 7. Maximum story shear force of each story by 0.6 Ta-katori records

Behavior of Structural Members



(a) Column at the top (1F) (b) Column at the base (1F) (c)Panel of center column (3F)

(d)Panel of side column (3F) (e) Panel of center column (2F) (f) Panel of side column (2F)

Figure 8. Hysteresis behavior of column and panel by 0.6 Ta-katori records

Fig. 8 shows response of primary members at the 1st and 2nd stories. The inelastic behavior observed not only in panel zones but also in columns. In the center column, yielding occurred at the both of top and bottom ends and the obvious deterioration of the strength is observed in Fig. 8(b). From visual observation after the test, residual out-of-plane deformation of the square hollow section was found in the vicinity of the column base. The primary plastic deformation is exhibited in the panels of the 2nd and 3rd stories. The panels, which experienced large plastic strain, were extended to side columns of the 2nd floor and the center column of the 3rd floor compared with the response by 0.4 Takatori records. Therefore, during the response by 0.6 Takatori, a collapse mechanism by yielding of panels in the 2nd and 3rd floor and the column bases at the 1st story is developed.

Collapse Behavior By Takatori Record

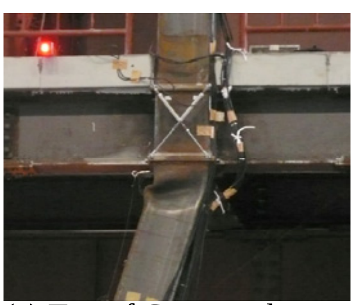
Behavior of frame and story

The collapse occurred by 1.0 times Takatori records, peak ground velocity is 1.28m/s, i.e., 2.5 times larger than the level 2 earthquake. The collapse mode was a side-sway with a mechanism in the first story as shown in Fig. 9(a) and 9(b). Plastic hinging and local buckling occurred at both the top and base of the columns as shown in Fig. 9(c) and 9(d). There was yielding in other members (columns above first story, beams, panel zones), but these did not govern the collapse.

Fig. 10 shows the story shear force and inter-story drift relationship during 1.0 times Takatori records. The peak story drift angle at 1st story were 0.08 and 0.19 in X and Y direction respectively. On the other hand, the peak story drift angle above the 2nd stories were about 0.01 to 0.02 rad and increase of drift angle from previous test were small. The in-crease of story drift is concentrated at the first story.


(a) Overall of A-frame


(b) Collapse of 1st story


(c) Top of Center column of 1st story


(d) Column base of center column

Figure 9. Collapse of specimen after test by 1.0 Takatori re-cords

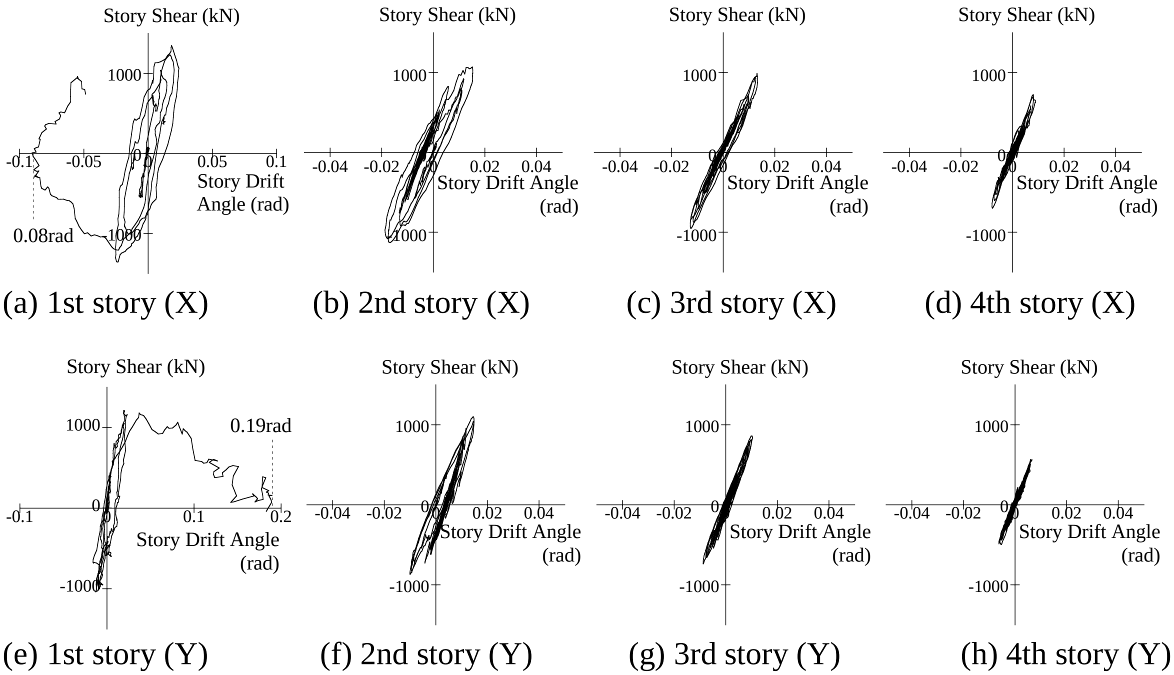


Figure 10. Story shear force and interstory drift relationships by 1.0 Takatori records

The change of the peak story drift during all tests are shown in Fig. 11. In the X-direction, the maximum drift occurred at the 2nd story until 0.6 Takatori records, and in the Y-direction, the maximum drift of 1st story increased larger than above stories by 0.2 Takatori. These change of the maximum drift profile indicates that the transition of controlling mechanism of the specimen frame occurred during 0.6 to 1.0 Takatori records.

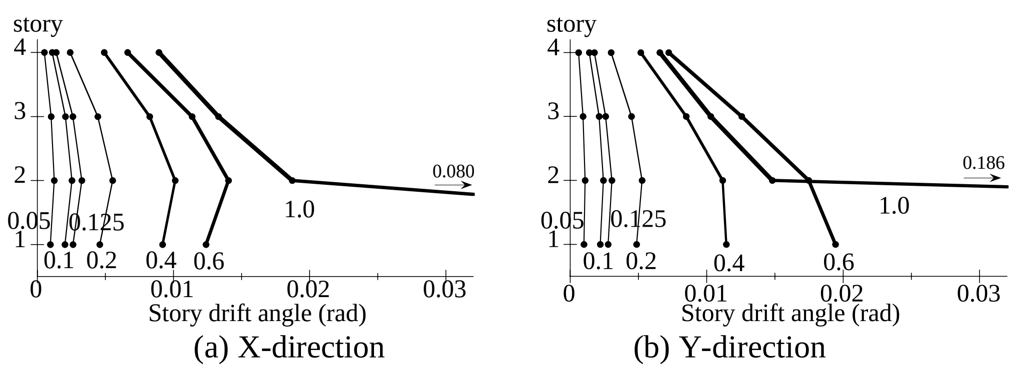
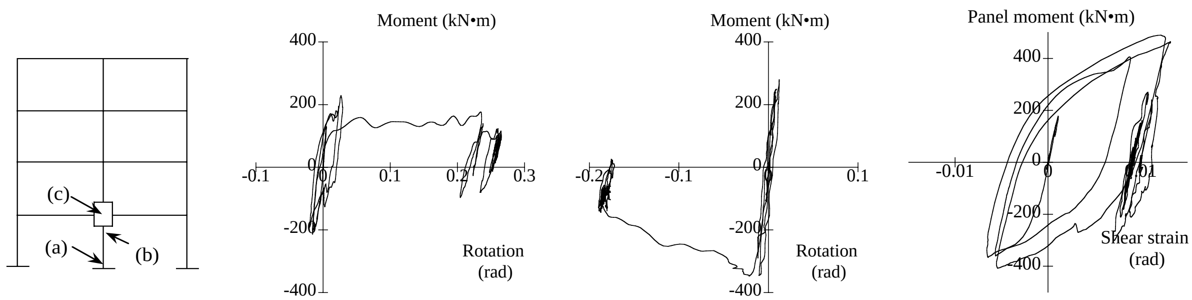


Figure 11. Profile of maximum story drift angle at each excita-tion level by Takatori records

Behavior of structural members and transition of collapse mechanism

The transition of the mechanism of the frame from a overall sway mechanism to a weak story mechanism is verified from the behavior of structural members. Fig. 12 shows the response of primary members at the 1st and 2nd stories by 1.0 Takatori records. At the base of the center column of the 1st story, the peak moment of the column shown in Fig. 12(a) is less than the results of the previous test by 0.6 Takatori records shown in Fig. 8(b) due to deterioration. At the top side of the column, the bending moment remarkably deteriorated as shown in Fig. 12(b). On the other hand, the panel showed stable hysteresis behavior, but soon after the deterioration of the column strength, unloading of the panel moment occurred as shown in Fig. 12(c) and the base shear of the frame decreased fatally. From these hysteresis relationships, the instance at the failure of the primary members are detected.

Fig. 13 shows the orbit of story drift at the 1st story by 1.0 Takatori records. The instances of the degradation of the column strength and the unload of the panel are marked by circles on the line of the orbit. The change of the mechanism initiated by the deterioration of the center column at 5.89s elapsed from the start of Takatori records, and a weak story mechanism is completed at 5.97s by the degradation of the side column. As shown in Fig.13, the shift of mechanism occurred in mere 0.08 seconds and the specimen completely collapsed and settled on the safe guard frame in the 1st story at 6.57s elapsed from the start of Takatori records. Thus, the deterioration of the column due to local buckling after several cyclic plastic deformation in large amplitude is likely one of the scenarios for collapse of a steel moment frame designed in current seismic codes. The excitation level of ground motion to collapse was twice and half larger than the level 2 design level.



(a) Column at the base (b) Column at the top (c) Panel of center column

Figure 12. Hysteresis behavior of column and panel by 1.0 Takatori records

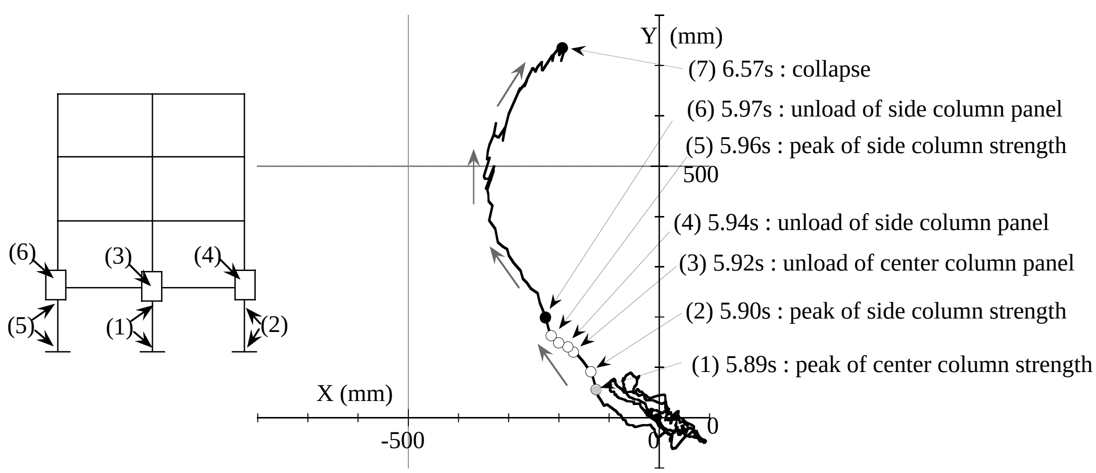


Figure 13. Story drift orbit of 1st story and sequence of collapse by 1.0 Takatori records

Conclusions

A full-scale shake table test on a four-story steel building was conducted to evaluate structural and functional performance of the steel building under design-level ground motions and under exceedingly large ground motions.

1) From the shake test by level-2 design earthquake, the specimen exhibited stable response behavior and a overall sway mechanism of the moment frame corresponded to the form

intended in the seismic design.

2) At the 1.5 times over level-2 excitation test, the primary plastification of the frame extended and a mechanism by yielding of panels in 2nd and 3rd floors and the column base at the 1st story is devel-oped. The frame behaved stably showing increased story shear resistances.

3) By the 2.5 times over level-2 excitation test, the specimen completely collapsed and the deterioration of the column due to local buckling after several cy-cles of large plastic deformation caused the change of the mechanism to a weak story collapse mecha-nism. This is a possible scenario of collapse for steel frames designed by current design codes under over design level earthquakes

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