COLLAPSE EXPERIMENT ON 4-STORY STEEL MOMENT FRAME PART 2 DETAIL OF COLLAPSE BEHAVIOR

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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 it is attached with non-structural components. This paper is an extension of a companion paper that describes objectives, design of the specimen, protocols of the excitation and the outline of the results. This paper presents further detailed results concerning the nonlinear response of the specimen building subjected to JR Takatori records of 1995 Hyogoken-Nanbu earthquake. By 40% and 60% scaled Takatori records, which corresponds to the level 2 and larger seismic load, maximum inter-story drift exceeds 0.01 radian and some structural members experienced yielding and the mechanism of the moment frame was overall sway mechanism. By 100% Takatori records, the mechanism of the specimen changed to a weak story type mechanism in the first story by strength deterioration of columns due to local bucklings at the top and bottom ends and collapsed. 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.

KEYWORDS: Steel moment frame, Shaking table test, Full-scale specimen, Collapse behavior

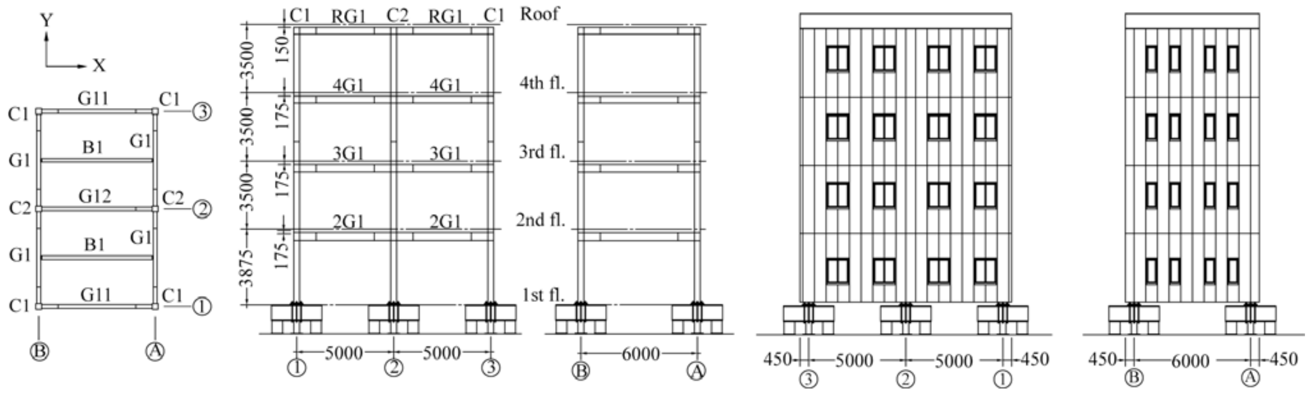
1. INTRODUCTION

1.1. Scope of Full-Scale Collapse Experiment

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.

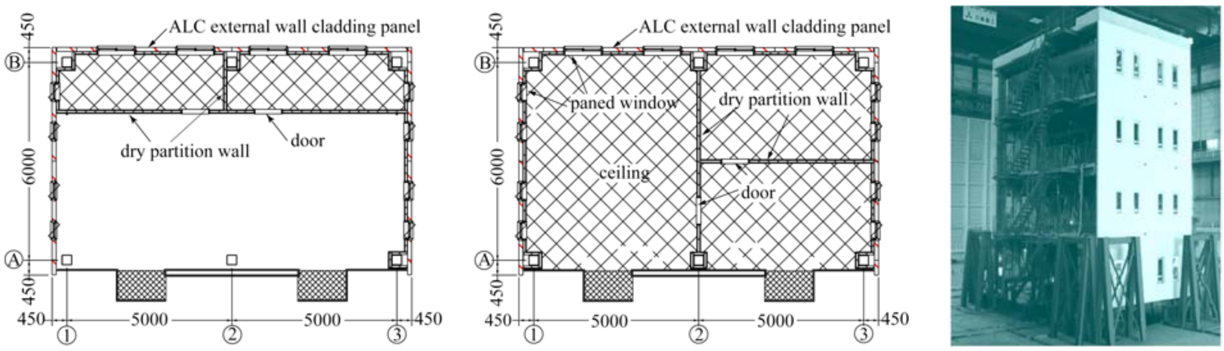
1.2. Brief Description of Specimen and Experimental Program

The test structure was a four-story, two-bay by one-bay steel moment frame as shown in Figure 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. External wall cladding panels of ALC were placed on three sides of the frame as shown in Figure 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 Figure 2.



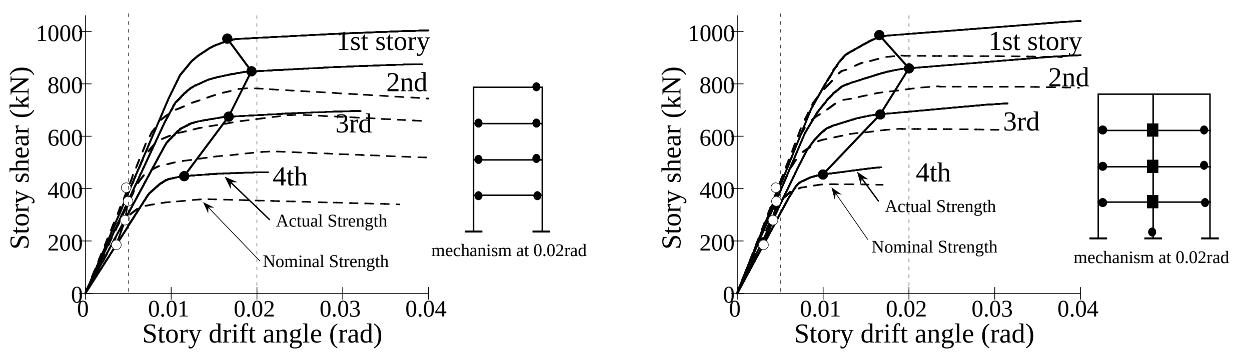
(a) plan (b) y-elevation (c) x-elevation (d) B-frame (e) 1-frame

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



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

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



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

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

The structure was designed following the most common design considerations exercised in Japan for post-Kobe steel moment frames. Figure 3 shows the results of pushover analysis of the test frame. The dotted lines show results based on the nominal material strength and solid lines show results from the actual material strength obtained from coupon tests. From the analyses, 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.
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 1. The details of the specimen, plan of experiment, instrumentation and excitation are described in the companion paper by S. Yamada et al. (2008). 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. In the response under 0.2 times Takatori, the peak story drift angle is less than 0.005 and no yielding is observed in both structural and non-structural elements except for minimal local deformation of partition walls at the connection with frames of doors and sashes. The specimen behaved generally in elastic. The detail of elastic response and dynamic characteristics of the specimen are also shown in the companion paper by S. Yamada et al. (2008).

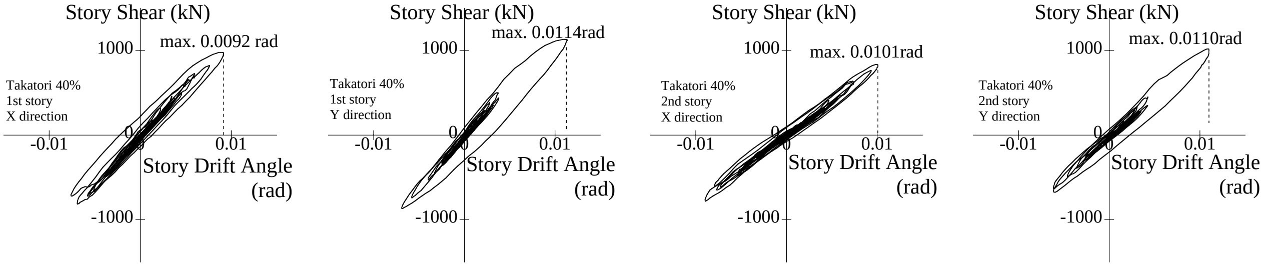
Table 1 Test Earthquake Scale Factors and Summary of Responses

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.

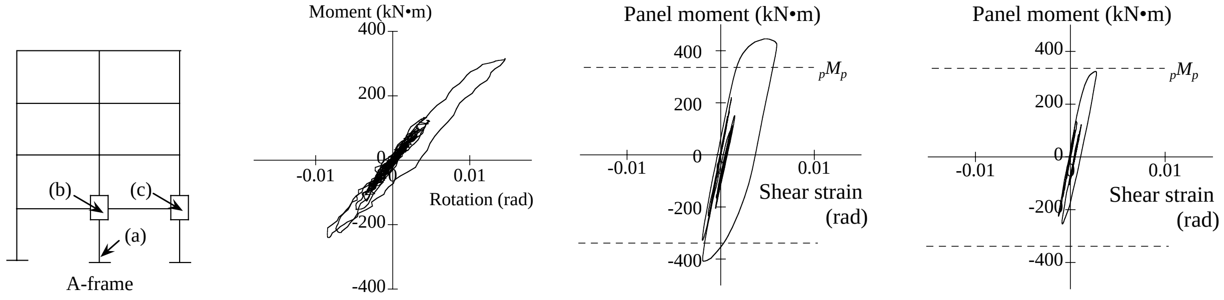
2. INELASTIC RESPONSE BY LEVEL 2 AND OVER-LEVEL 2 EARTHQUAKE

2.1. Inelastic Response by Level 2 Earthquake

As the records equivalent to a Level 2 design earthquake (peak ground velocity is 0.5m/s), 0.4 times the Takatori records are used. The shear force and story drift angle relationships in X and Y direction at the 1st and 2nd story are shown in Figure 4. In the present paper, 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 related to the whole structural and non-structural components of the sptory. The peak story drift angle is 0.0114rad at the 1st story and obviously yielded at 1st and 2nd story. The largest inelastic behaviors were observed at panel zones. Figure 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 (Figure 5a) at the base and remained in elastic at the



(a)1st story, x-direction (b)1st story, y-direction (c)2nd story, x-direction (d)2nd story, y-direction
Figure 4 Story shear force and inter-story drift relationships by 0.4 Takatori records

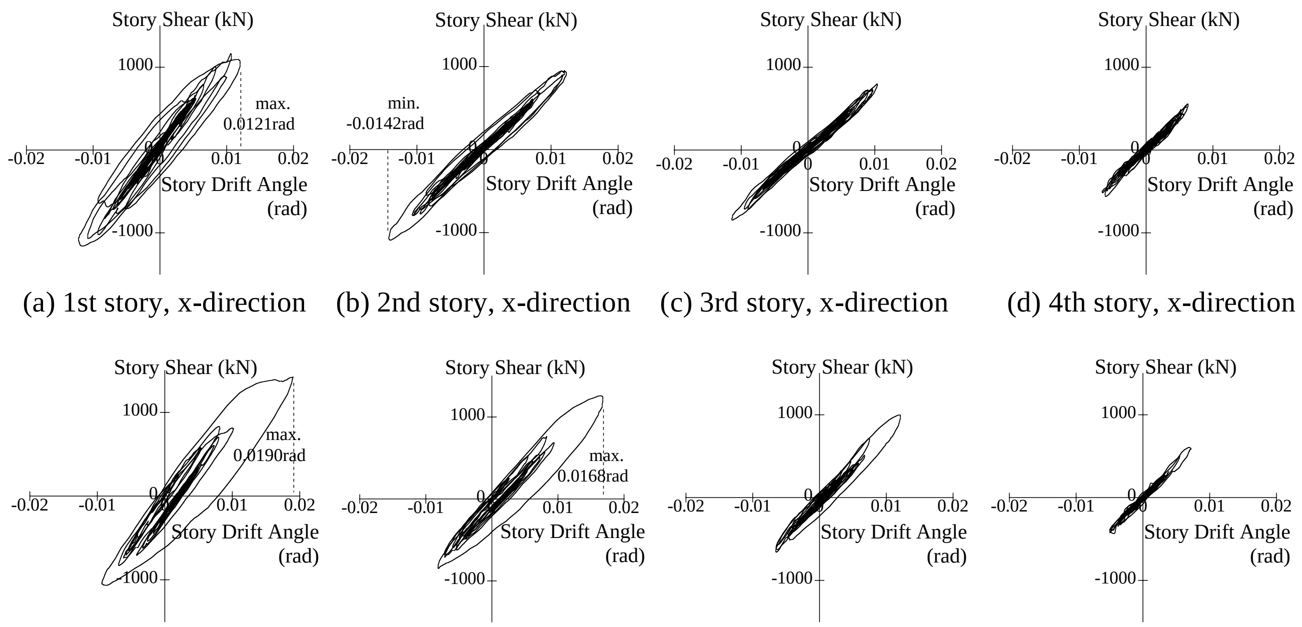


(a) Column at the base (b) Panel of center column (c) Panel of side column
Figure 5 Hysteresis behavior of column and panel by 0.4 Takatori records

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 (Figure 5b). At the side columns, panels are also yielded but panel moments are slightly less than the full plastic strength (Figure 5c). Both in X and Y direction, the frame behaved in a overall sway mechanism as intended in the design.

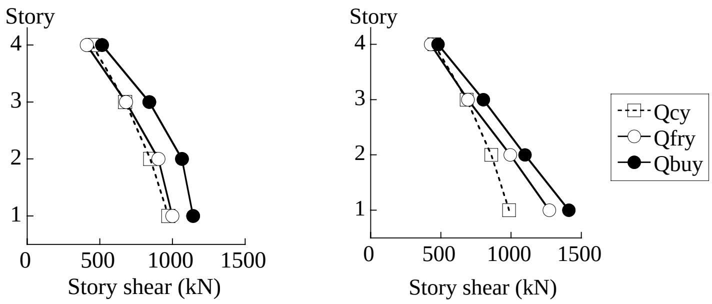
2.2. Inelastic Response by Over Level 2 Earthquake

Figure 6 shows the story shear force and inter-story drift relationships by 0.6 times the Takatori records. The peak ground velocity was 0.75m/s, 1.5 times larger than Level-2 earthquakes. The peak story drift angle increased to 0.019 at 1st story in Y-direction and inelastic hysteresis relationships were observed in 1st to 3rd stories, which means that the overall sway mechanism proceeded. Figure 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 as shown in Figure 3. 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 Figure 6(d). These results indicate that the frame attained its ultimate strength level by the over-all sway mechanism.



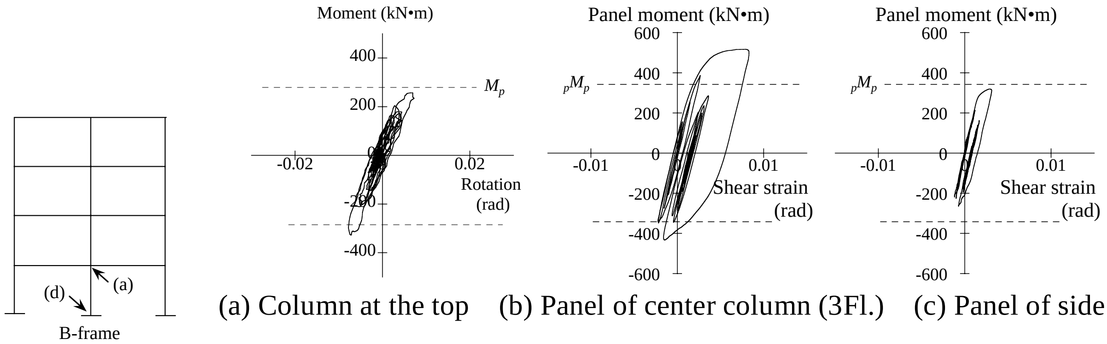
(a) 1st story, x-direction (b) 2nd story, x-direction (c) 3rd story, x-direction (d) 4th story, x-direction

(e) 1st story, y-direction (f) 2nd story, y-direction (g) 3rd story, y-direction (h) 4th story, y-direction
Figure 6 Story shear force and inter-story drift relationships by 0.6 Takatori records

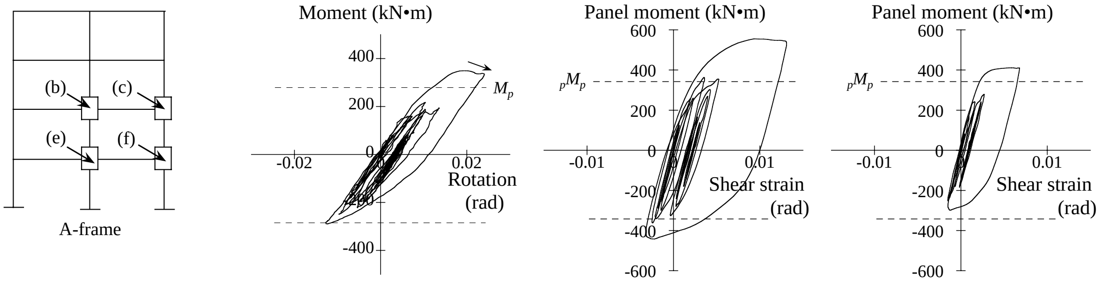


(a) X-direction (b) Y-direction

Figure 7 Maximum story shear force of each story by 0.6 Takatori records



(a) Column at the top (b) Panel of center column (3Fl.) (c) Panel of side column (3Fl.)



(d) Column at the base (e) Panel of center column (2Fl.) (f) Panel of side column (2Fl.)

Figure 8 Hysteresis behavior of column and panel by 0.6 Takatori records

Figure 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 Figure 8d. From visual observation after the test, residual out-of-plane deformation of the square tube 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 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 2nd and 3rd floor and the column bases at the 1st story is developed.

After the 0.6 times Takatori records shake test, several damage to non-structural components are also observed. The cracks at the corner were observed in some ALC panels of exterior walls. Damage to gypsum boards of partition walls and steel frame of doors are also observed. The details of behavior and damage of non-structurral components are presented in the companion paper by Y. Matsuoka et al. (2008).

3. COLLAPSE BEHAVIOR BY TAKATORI RECORD

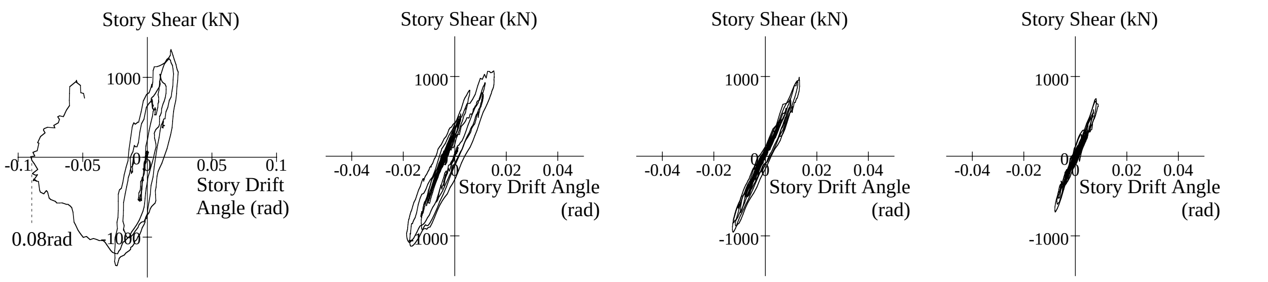
The collapse occurred by 1.0 times Takatori records, peak ground velocity is 1.28m/s, i.e., 2.5 times larger than the level-2 earthquake. The collapse mode was a side-sway with a mechanism in the first story as shown in

Figure 9(a,b). Plastic hinging and local buckling occurred at both the top and base of the columns as shown in Figure 9(c). There was yielding in other members (columns above 1st story, beams, panel zones), but these did not govern the collapse.

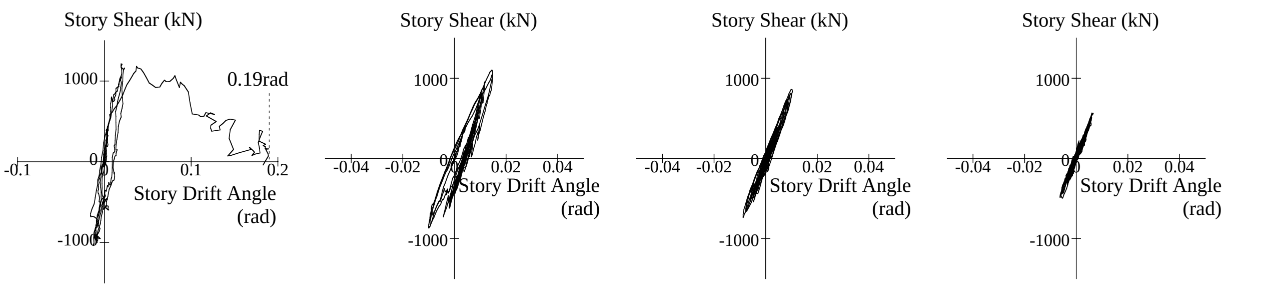
Figure 10 shows the story shear force and inter-story drift relationship during 1.0 times Takatori records. The peak story drift angle at 1st story were 0.08 and 0.19 in X and Y direction respectively. On the other hand, The peak story drift angle above 2nd stories were about 0.01 to 0.02 rad and increase of drift angle from previous test were small. The increase of story drift is concentrated at the 1st story. The change of peak story drift during all tests are shown in Figure 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 shift of controlling mechanism of the specimen frame occurred during 0.6 to 1.0 Takatori records.

(a) Overall of A-frame (b) Collapse of 1st story (c) Center column of 1st story

Figure 9 Collapse of specimen after test by 1.0 Takatori records

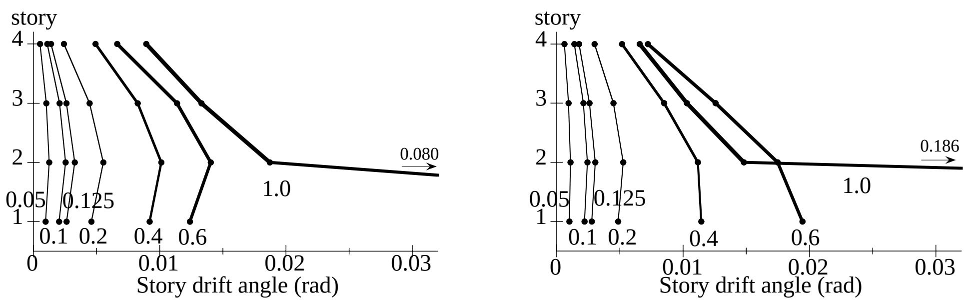


(a) 1st story, x-direction (b) 2nd story, x-direction (c) 3rd story, x-direction (d) 4th story, x-direction



(e) 1st story, y-direction (f) 2nd story, y-direction (g) 3rd story, y-direction (h) 4th story, y-direction

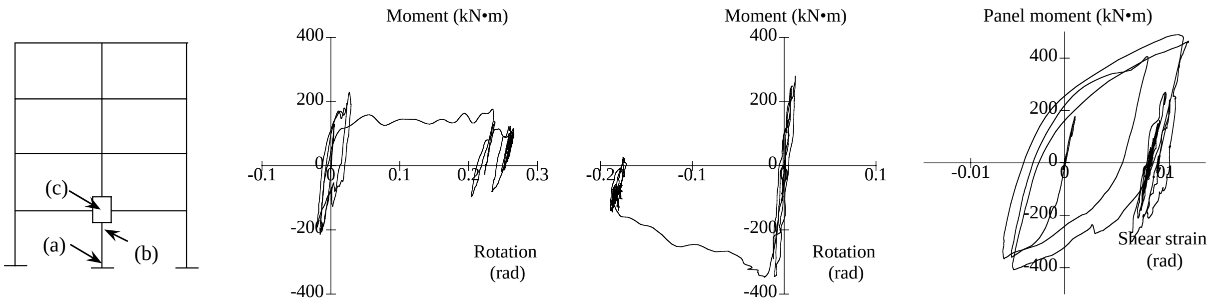
Figure 10 Story shear force and inter-story drift relationships by 1.0 Takatori records



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

Figure 11 Profile of maximum story drift angle at each excitation level by Takatori records

The shift of the mechanism of the frame from a overall sway mechanism to a weak story mechanism is verified from the behavior of structural members. Figure 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 Figure 12(a) is less than the results of the previous test by 0.6 Takatori records shown in Figure 8(d) due to deterioration. At the top side of the column, the bending moment remarkably deteriorated as shown in Figure 12(b). On the other hand, the panel shown stable hysteresis behavior, but soon after the deterioration of the column strength, unloading of the panel moment occurred and the base shear of the frame decreased fatally. From these hysteresis relationships, the instance at the failure of the primary members are detected. Figure 13 shows the orbit of story drift at the 1st story by 1.0 Takatori records. On the line of the orbit, the instance of the degradation of the column strength and the unload of the panel are merked by circles.



(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

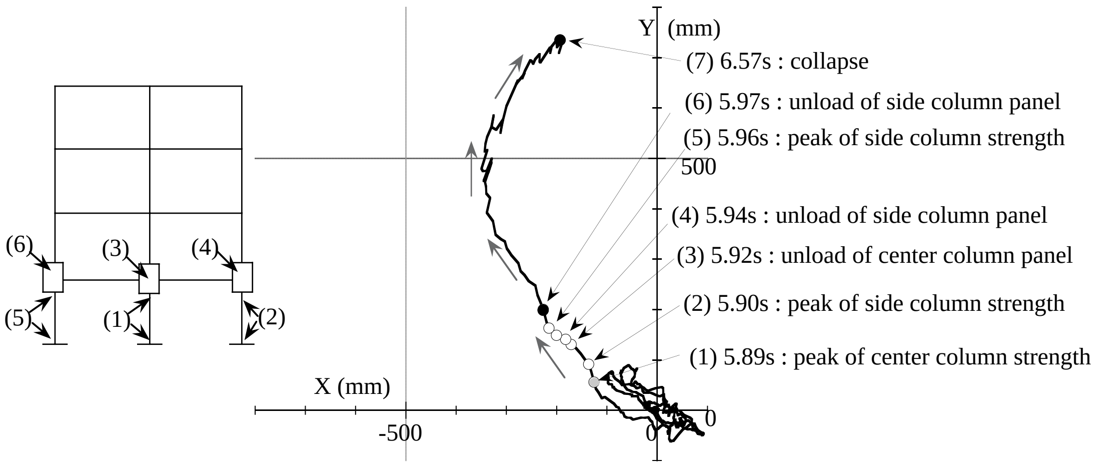


Figure 13 Story drift orbit of 1st story and sequence of collapse by 1.0 Takatori records

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 Figure 15, the shift of mechanism occurred in mere 0.08 seconds and at 6.57s elapsed from the start of Takatori records, the specimen completely collapsed and settled on the safe guard frame in the 1st story. 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 level-2 design level.

4. 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. 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. 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 cycles of large plastic deformation caused the change of the mechanism to a weak story collapse mechanism. This is a possible scenario of collapse for steel frames designed by current design codes under over design level earthquakes.

ACKNOWLEDGEMENT

This study is a part of "NEES/E-Defense collaborative research program on steel structures", and was pursued by the sub-working group under the Building Collapse Simulation Working Group (WG). The Japan team leader for the overall program is Kazuhiko Kasai, Tokyo Institute of Technology, and the WG leader is Keiichiro Suita, Kyoto University. The writers acknowledge Analysis Method and Verification Working Group (leader is Motohide Tada, Osaka University) for their collaboration . And the writers acknowledge financial support provided by the Ministry of Education, Culture, Sports, Sciences and Technology of Japan under Grant No. 18360266, and by the National Research Institute for Earth Science and Disaster Prevention.

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