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COLLAPSE EXPERIMENT ON 4-STORY STEEL MOMENT FRAME : PART 1 OUTLINE OF TEST RESULTS

S. Yamada[1], K. Suita[2], M. Tada[3], K. Kasai[4], Y. Matsuoka[5] and Y. Shimada[6]

[1] *Assoc. Prof., Structural Engineering Research Center, Tokyo Institute of Technology, Yokohama, Japan*
[2] *Assoc. Prof., Dept. of Architecture and Architectural Engineering, Kyoto University, Kyoto, Japan*
[3] *Prof., Division of Global Architecture, Osaka University, Osaka, Japan*
[4] *Prof., Structural Engineering Research Center, Tokyo Institute of Technology, Yokohama, Japan*
[5] *Researcher, National Research Institute for Earth Science and Disaster Prevention, Hyogo, Japan*
[6] *Graduate Student, Tokyo Institute of Technology, Yokohama, Japan*
Email: naniwa@serc.titech.ac.jp, suita@archi.kyoto-u.ac.jp, tada@arch.eng.osaka-u.ac.jp, kasai@serc.titech.ac.jp, pinehill@bosai.go.jp, shimada.y.ab@m.titech.ac.jp

ABSTRACT :

Shaking table test of full-scale 4-story moment frame was carried out at E-defense, one of the largest shaking tables in the world. The objectives of this experiment are to evaluate structural and functional performance of the steel moment frame under design-level ground motions, and to determine the safety margin against collapse under exceedingly large ground motions. The objectives of the full-scale test are to evaluate structural and functional performance of the building under design-level ground motions, and to determine the safety margin against collapse under exceedingly large ground motions. Major tasks involved in the experiments are summarized as follows:
1. To evaluate the structural performance of the building by subjecting a full-scale building specimen to design-level ground motions.
2. To evaluate the safety margin against complete collapse by subjecting the full-scale building specimen to ground motions significantly greater than design-level ground motions.
3. To evaluate the functional performance of buildings during and after a severe earthquake by observing the behavior of non-structural components. Data on the relationship between structural response and non-structural component response will be obtained to aid development of performance-based design methodologies.
4. To obtain data on real structural behavior up to collapse, which will then be used to calibrate and advance numerical simulation techniques.
The building specimen is designed according to the current Japanese specifications and practices (post 1995 Kobe earthquake).
In this part, protocols of the excitation and out line of the result are reported.

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

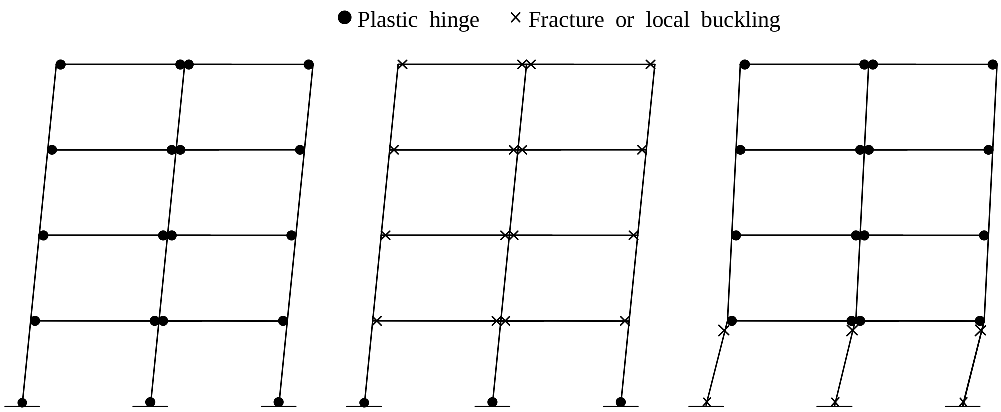
1.INTRODUCTION

1.1 General

In order to evaluate structural and functional performance of the steel moment frame under design-level ground motions, and to determine the safety margin against collapse under exceedingly large ground motions, full-scale shaking table test of 4-story steel building was carried out at E-defense. 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 current Japanese specifications and practices. In this test, not only seismic performance of structural system, but also seismic performances of non-structural components are examined. In this part, out line of the specimen, out line of the experimental method, and elastic behavior of the specimen are reported. The detail of elasto-plastic behaviors and collapse behaviors of the specimen building are shown in the companion paper by K. Suita et al. (2008), and the detail of seismic performance of non-structural components are also described in the companion paper by Matsuoka et. al. (2008).

1.2 Scenario for Complete Collapse

A seismic-resistant steel moment frame aims to contain plastic deformation in beams (and other energy dissipating elements), to limit yielding of the columns, and thereby, to maintain the gravity sustaining capacity and avoid collapse. The current seismic regulations ensure ductile deformation capacity of beams by specifying width-to-thickness ratio limits for the beam cross-section. As long as the story shear strength is sufficient to resist the P-Δ moment produced by gravity loads, the moment frame will not collapse [Figure 1(a)]. On the other hand, as observed in the Northridge and Kobe earthquakes, fracture of welded moment connections causes deterioration of beam strength, and hence reduces story shear strength [Figure 1(b)], thereby making the building more vulnerable to collapse. There is another possible scenario for collapse. 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 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 [Figure 1(a), (b)] to a weak story collapse mechanism [Figure 1(c)]. Based on detailed examination, 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 code.



(a) Mechanism envisioned in design (b) Overall collapse mechanism (c) Weak story mechanism
Figure 1 Collapse mechanism of steel moment frame

2. SPECIMEN

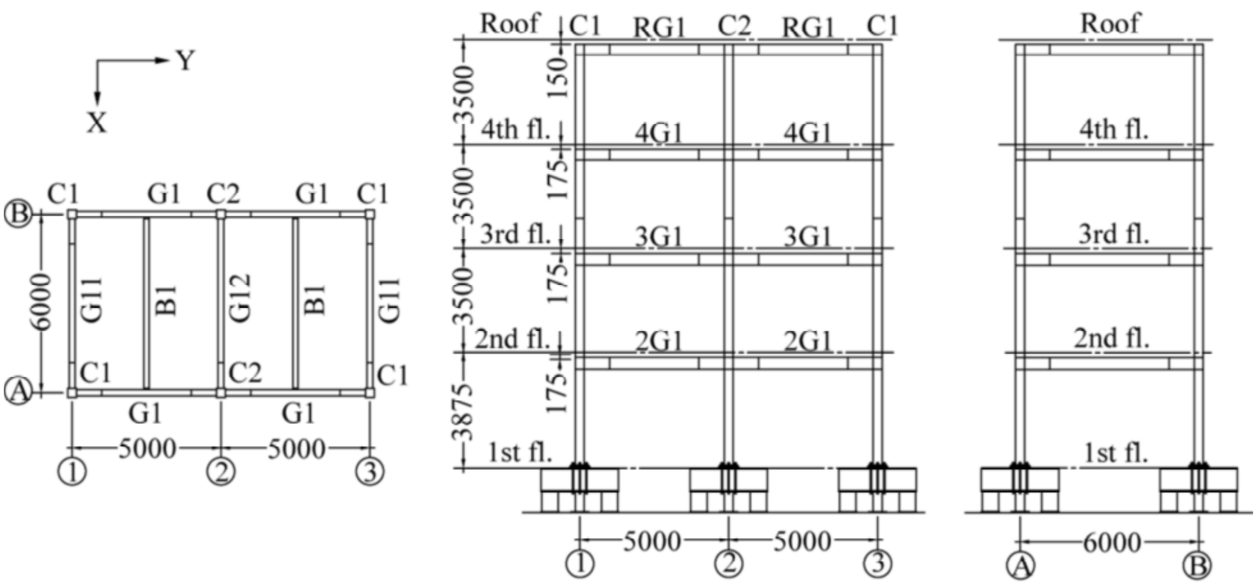
***2.1** Design Consideration*

The building specimen was designed within the following conditions and constraints.

1. Due to the loading capacity of the shaking table and safety considerations, a four-story, two-bay by one-bay steel moment frame was deemed the optimal configuration.
2. The structure was designed following the most common design considerations exercised in Japan for post-Kobe steel moment frames.
3. Welding details with no weld access hole, which are recommended after the Kobe earthquake [JASS6 1996], are adopted for the beam-to-column connections. This is a premium detail to ensure ductile deformation capacity and strain hardening of the beam. And sections with relatively large width-to-thickness ratio were selected for columns.. Those increase the applied forces in columns, and increase the likelihood that local buckling and the consequent strength degradation occur in the columns.
4. The frame should remain elastic and the story drift should be less than 1/200 for Level-1 design earthquake load (computed with a base shear coefficient of 0.2). At the ultimate state (plastic collapse of the frame), the base shear coefficient should be greater than 0.3 (to meet the requirements of Level-2 design).
5. The strong column-weak beam philosophy is employed. Specifically, at each story, the sum of design

column strength is more than 1.5 times greater than the sum of the design strength of the beams and 1.3 times greater than the sum of the design strength of the panel zone.
6. Exposed base plate connection with enough rigidity is adopted for the column bases. The design strength of the column base connection, which is controlled by the yield strength of anchor bolts, is between 1.3 and 1.8 times greater than the design strength of columns. The rigid column base connection is expected to increase the likelihood that local buckling and the consequent strength degradation occur at the column base.

2.2 Framing System and Conformance to Code Requirements
The building has plan dimensions of 10 m in the longitudinal direction (Y) and 6.0 m in the transverse direction (X). Each story is 3.5 m high, making the overall story height equal to 14 m. Table 1 shows a list of sections and Table 2 shows the specified and measured material properties of steel members. The wide flange beams ranged from 340 to 400 mm deep, and columns were of square hollow sections of 300 mm wide. 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 of this type of materials. On the other hand, the yield strength of wide-flanges for beams are fairly larger than specified values. Therefore, the actual strength ratio of columns to beams of the specimen frame are lower than expected in design. Metal decks are connected to the beams through studs which are welded to the beam top flanges through the metal decks. Wire-meshes are placed above the metal decksheets, and concrete is placed on site. Fully composite action is expected between the steel beams and concrete slab.
The story shear versus story drift relationships at each story, obtained from pushover analyses of the test specimen are shown in Figure 3. In these analyses, the earthquake load distribution specified in the code provisions was used. Solid lines are analytical results based on the measured yield strength of material, and the broken lines are analytical results based on the nominal strength of material. The story drift at the Level-1 design earthquake load is indicated by open circles. At this load level, the frame was elastic, and the story drift at each story was less than 1/200. The story drift when the maximum story drift was equal to 1/50 is indicated by filled circles. At this stage, the base shear coefficient was 0.46 in the Y-direction and 0.47 in the X-direction. Therefore, the analysis shows that the structure satisfies essential code requirements. The natural periods of the specimen in Y-direction calculated from these analysis models are 0.90s in 1st mode and 0.29s in 2nd mode, respectively. The total weight of specimen is 2113kN.



(a) Plan (1st Floor) (b) Y-Elevation (Longitudinal) (c) X-Elevation (Transverse)
Figure 2 Specimen (4-Story Steel Moment Frame)

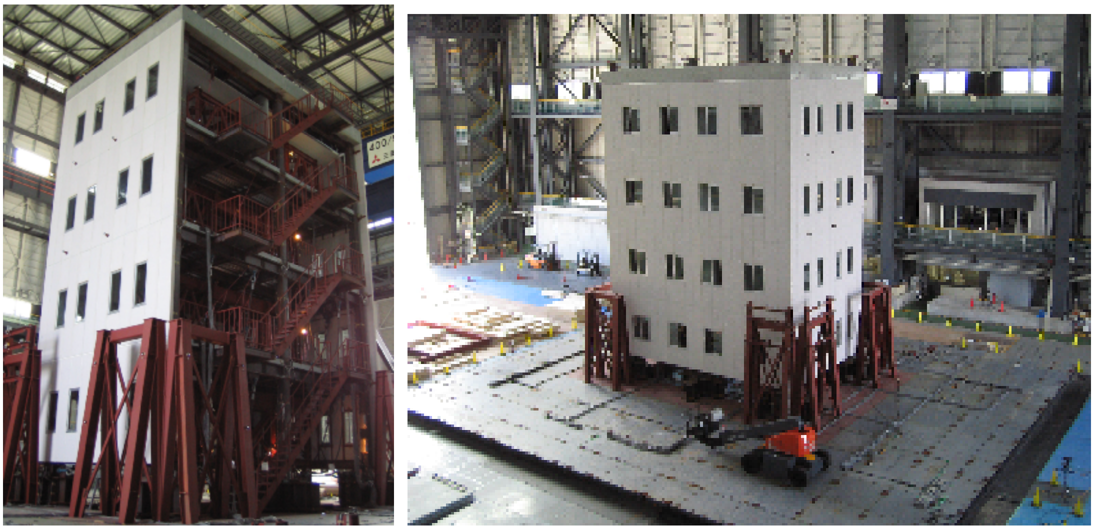


Photo 1 Specimen Building

Table 1 List of member sections

Floor	Beam * G1	Beam * G11	Beam * G12	Story	Column ** C1, C2
R	H-346×174×6×9	H-346×174×6×9	H-346×174×6×9	4	SHS-300×300×9
4	H-350×175×7×11	H-350×175×7×11	H-340×175×9×14	3	SHS-300×300×9
3	H-396×199×7×11	H-400×200×8×13	H-400×200×8×13	2	SHS-300×300×9
2	H-400×200×8×13	H-400×200×8×13	H-390×200×10×16	1	SHS-300×300×9

* wide flange: height × width × web thickness × flange thickness, ** square hollow section: height × width × thickness

Table 2 Material properties

Member	Steel	Section	Element	Specified Properties* Yield stress	Specified Properties* Tensile strength	Measured Properties* Yield stress	Measured Properties* Tensile strength
Beam	SN400B	H-340×175×9×14	flange	235	400	309	443
			web	–	–	355	468
		H-346×174×6×9	flange	235	400	333	461
			web	–	–	382	483
		H-350×175×7×11	flange	235	400	302	441
			web	–	–	357	466
		H-390×200×10×16	flange	235	400	297	451
			web	–	–	317	458
		H-396×199×7×11	flange	235	400	311	460
			web	–	–	369	486
		H-400×200×8×13	flange	235	400	326	454
			web	–	–	373	482
Column	BCR295	SHS-300×300×9	1st fl.	295	400	330	426
			2nd-4th fl.	295	400	332	419

* unit: N/mm^2

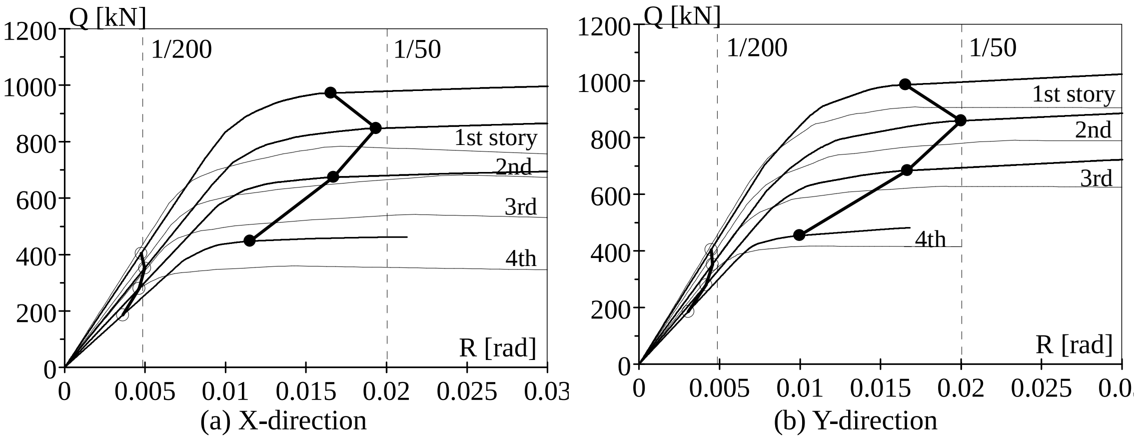


Figure 3 Story shear versus story drift relationships obtained from pushover analyses

2.3 Non-structural Components

Not only structural system, but also non-structural components were tested in this shaking table test. Non-structural components, i.e. external wall cladding panels and interior dry partition walls were installed to the specimen building. Detail of non-structural components are described in the companion paper. (Matsuoka et.al. 2008)

3. Experimental Method

3.1 Set-up

Specimen building was set on the center of the shaking table. Each column base is connected to steel foundation beams which are tied down to the shaking table. A safeguard system consists of the three safety features to protect the shaking table from damage during the collapse test was designed as shown in Figure 4. (1) Diagonal wires attached to the first and second stories and each span of the perimeter frame to prevent inter-story drift beyond 1/3.5 rad. (2) Tables placed on the floor slab of each story to arrest the floor slab dropping from the upper story. (3) A fence surrounding the specimen which prevents exceedingly large story drift at the first story.

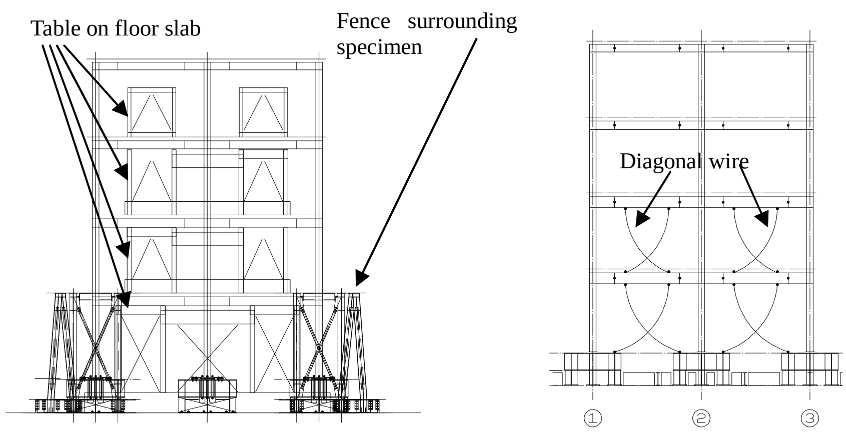


Figure 4 Safe guard system protect shaking table

3.2 Excitation

At first, free vibration test was carried out to examine the fundamental natural period and damping coefficient of the specimen. Then, several seismic excitations with different input level were applied to the specimen. In the seismic excitation, JR Takatori station Record [Nakamura et al. 1995] was used as input wave. NS, EW, and UD components were considered for the Y, X, and Z directions. The positive direction of each coordinate corresponded to South, West, and Up direction, respectively. Response velocity spectrum is shown in Figure 5. Input wave has strong peak at slightly longer period than the predicted fundamental natural period of the specimen. So, large amount of energy input can be expected along with the yielding of the specimen. List of the main excitation is shown in Table 3. The excitation level is expressed as the amplification factor which is multiplied to the original record.

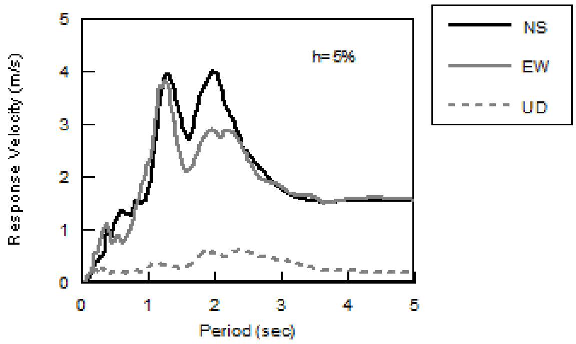


Figure 5 Response velocity spectrum of the JR Takatori station record

Table 3 List of the excitation

Number of the excitation	Amplification factor	State of the specimen	
1	0.05	elastic	
2	0.1	elastic	
3	0.125	elastic	
4	0.2	elastic	corresponding to the design earthquake level-1
5	0.4	elasto-plastic	corresponding to the design earthquake level-2
6	0.6	elasto-plastic	
7	1.0	collapse	

3.3 Measurement

To measure 3-dimensional dynamic behavior of the specimen building, a total of 945 instrumentations are used.

Main measurement items are followings; In order to measure acceleration, three dimensional servo type accelerometers were set up in the center and the four corners of the upper surface of the shaking table and each floor as shown in Figure 6. Inertial force in each direction was obtained by multiplying acceleration to mass of the story. Both laser-type and potentiometer-type displacement transducer were set to directly measure story drift and rotation as shown in Figure 6. These displacement sensors are accurate but not able to measure large displacement because of those capacities. So, in the collapse excitation, three dimension behavior of the 1st and 2nd story were measured by other potentiometer type displacement transducers with large capacity. Rotation angle of beams and columns and deformation of panel zones are also measured by displacement transducers as shown in Figure 7. Restoring forces were measured by strain gauges located at the elastic portion of beams and columns as shown in Figure 8. Besides these, displacement transducers to measure the behavior of non-structural components were installed.

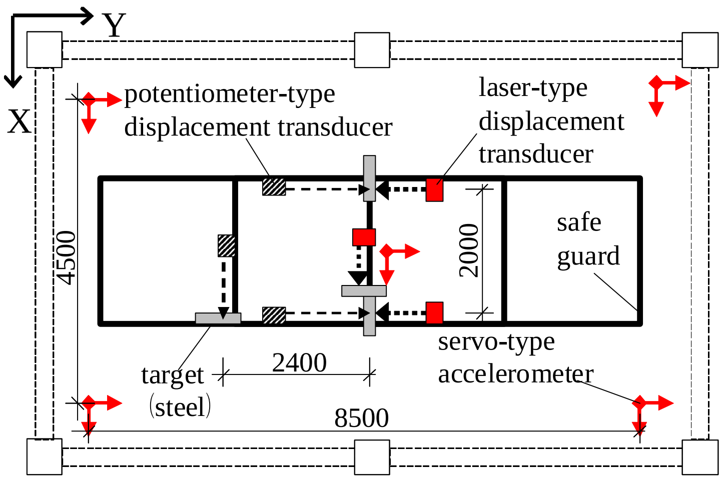


Figure 6 Accelerometers and displacement transducers in each story

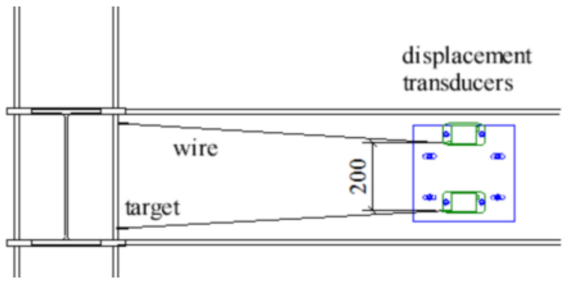


Figure 7 Measurement of rotation angle of beam

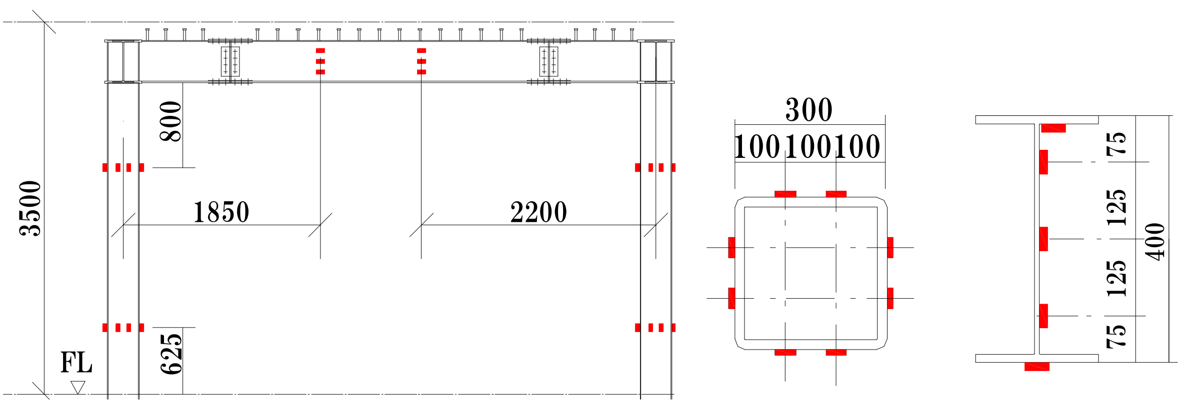
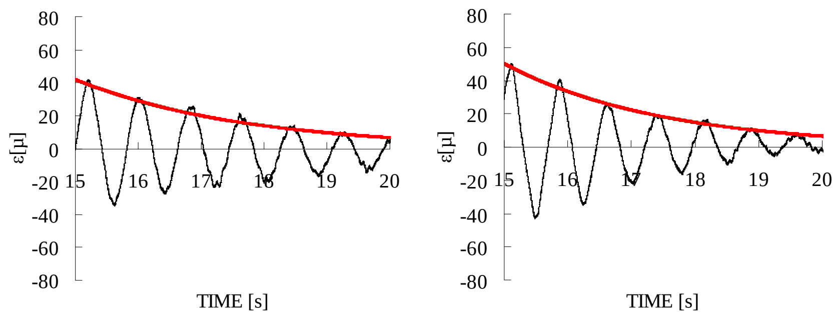


Figure 8 Strain gauges

4. EXPERIMENTAL RESULTS IN ELASTIC RANGE

4.1 Fundamental Natural Period and Damping Factor of the Specimen Building

Before main excitation, free vibration test was carried out to examine the fundamental natural period and damping factor of the specimen building. Time history of strain gauges are shown in Figure 9 as test results.. Fundamental natural period of X and Y direction are 0.80 sec and 0.76 sec, and damping factor of X and Y direction are 2.1% and 2.3% respectively.

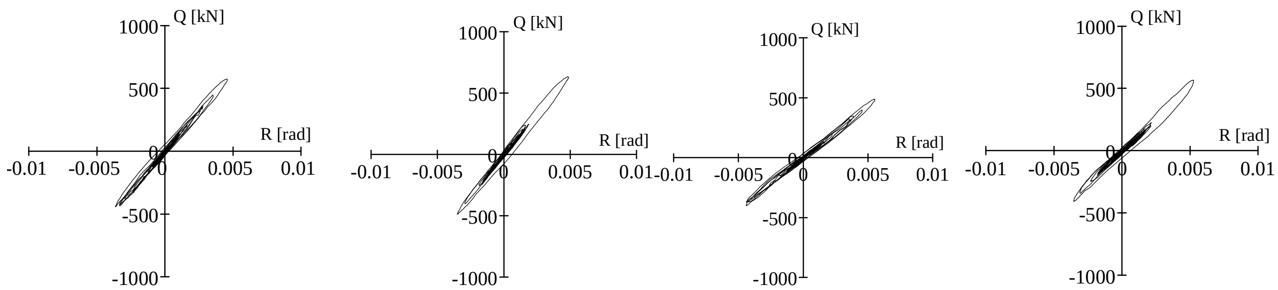


(a) X-Direction (b) Y-Direction

Figure 9 Results of the free vibration tests

4.2 Response under Level 1 excitation

Story shear and story drift angle relationship in X and Y direction at the 1st and 2nd story are shown in Figure 10. Under Level-1 excitation, maximum story drift angle at each story reached around 1/200 of the design criteria. Slight non-linearity is observed in the Figure10. But each member kept in elastic range under this excitation. That is because story shear in those relationships are calculated by the inertial force obtained from the acceleration record on each floor times mass of the story. It is sum of restoring forces and damping forces related to the whole structural and non-structural components of the specimen building. So, non-linearity in the story shear and story drift angle relationships are thought to be caused by damping.



(a)X-direction of 1st Story (b) Y-direction of 1st Story (c) X-direction of 2nd Story (d) Y-direction of 2ndt Story

Figure10 Story shear and story drift angle relationship under Level 1 excitation

4.3 Elastic stiffness

Elastic stiffness of each story obtained from story shear and story drift relationship by the least square method are shown in Table 4. In the table, K_{bu} is the elastic stiffness of whole building including non-structural components. K_{bu} is calculated from story shear which is obtained by inertial force. K_{fr} is the elastic stiffness of framing system. K_{fr} is calculated from story shear which is obtained by restoring force of column measured by strain gauges. , K_a' is elasticity stiffness of the framing system obtained from push-over analysis. Comparing K_{bu} with K_{fr}, K_{bu} is larger than K_{fr} because of the contribution of the stiffness of non-structure components. Comparing Kfr and Ka', Kfr are larger than Ka' in 1st and 4-th story. In 4-th story, stiffness of the parapet is thought to be affected to the elastic stiffness of the story. And in the 1st story, rigidity of the column base is thought to be affected to the elastic stiffness of the story.

Table 4 Elastic stiffness

Story	X-direction			Y-Direction		
	Kbu (MN/m)	Kfr (MN/m)	Ka' (MN/m)	Kbu (MN/m)	Kfr (MN/m)	Ka' (MN/m)
4	25.2	18.9	14.7	25.6	20.7	17.4
3	22.7	18.2	17.3	27.4	20.9	19.2
2	24.5	20.1	20.2	28.7	22.3	22.1
1	32.6	27.8	21.9	33.4	29.6	22.3

5. CONCLUSION

In this paper, outline of the specimen building, outline of the experimental method and experimental results of elastic excitation of the full-scale shaking table test of 4 story steel building were reported. Under level-1 excitation, specimen building remain elastic and generated maximum story drift almost corresponds to the design criteria.

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