Seismic Resistance Capacity of High-Rise Buildings subjected to Long-Period Ground Motions - E-Defense Shaking Table Test

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ABSTRACT :

A series of full-scale shaking table tests was conducted for a full-scale 21-story steel building frame subjected to a series of long-period, long-duration ground motions. The specimen consisted of a four-story, two-span by one-bay steel moment frame and an equivalent system. The frame portion was to present the lower four stories and the equivalent system was to simulate the remaining stories of the 21-story building. Major observations obtained from the test show that maximum drifts are smaller than 0.5% and 1.0% under minor and major earthquake loadings, respectively. Cyclic inelastic loading finally caused several fractures at the bottom flanges of beam ends. Shop weld connections show better cumulative deformation capacity than field weld ones. Story hysteresis behaviors remained stable and no severe strength deterioration occurred after connections fractured.

KEYWORDS: long-period ground motion, high-rise building, seismic performance, beam-to-column connections, shaking table test

1. INTRODUCTION

Occurrences of large earthquakes having a magnitude over eight along subduction zones in the southwestern part of Japan have been well documented in historical materials. The recurrence periods are remarkably regular and there is high probability that Japan is to be hit by large earthquakes in the middle of twenty-first century. Such subduction zone earthquakes are known to generate long-period ground motions in land, of which predominant periods range from several to ten seconds and the duration can be over five minutes (Mochizuki et al. 2003; Kamae et al. 2004; Nakashima et al. 2006). In this case, the energy input to a high-rise building can reach more than ten times of the design value, and the maximum and cumulative deformations may become excessive and the floor responses may intensify significantly (JSCE, AIJ. 2006). Especially for the early steel moment frames, large cumulative deformation tend to occur at the old types of beam-to-column connections, where much damage was observed in the Northridge and Hyogoken-Nanbu earthquakes. (Nakashima et al. 2000)

However, no actual data from either observations or physical tests are available regarding the behavior of high-rise buildings subjected to long-period ground motion. Therefore, this study presents shaking table tests applied to a high-rise building to acquire realistic data on the progress of damage to structural elements and the redundancy of the frame system in the deformation range beyond values considered in seismic design.

2. E-DEFENSE SHAKING TABLE AND TEST SYSTEM

The test was conducted using the E-Defense shaking table facilities in Japan, in 2008. The table is 20 meters by 15 meters in the plan dimension. Loading capacity is 1200 metric ton, and the largest space for specimen is 22 meters in high (Nakashima et al. 2006; Ogawa et al. 2001).

2.1 Reduction procedures of test structure

According to the statistical results from studies of existing buildings (Fukushima et al. 1999; BRI. 2005), neither strength nor stiffness of existing buildings changes with ages a lot in Japan. To evaluate the seismic performance of early high-rise buildings subjected to long-period ground motion, an 80 m high 21-story building was chosen as the prototype building by referring to statistical results. Considering the loading capacity of the shaking table, equivalent reduction methods were proposed to reduce the scale of the prototype building. Figure 1 shows the concepts of specimen and reduction methods. This test system consists of a lower part, represented by four-story, two-span by one-bay steel frame structure which has the same dimension as the prototype building in span length and story height, and an upper part simplified by an equivalent system made of concrete mass and rubber bearings to simulate the effects of the middle and upper part of a high-rise. Reduction methods are described below using multi-lumped-mass model (MLM model):

1. Design parameters of prototype building such as normalized yielding base shear coefficient, stiffness and strength distribution and so on, which were selected with reference to the statistical results, were present in a tri-linear spring of each story.
2. Story initial stiffness was arranged to achieve the first model period, 2.4 seconds which was calculated from the 0.03 times of building height. Stiffness distribution in the vertical direction was determined according to the Japanese Building Standard, which showed results similar to UBC codes (Nakashima et al. 2000).

Then, the total weight of the 21-story MLM model was reduced to below 1200t proportionally and given to the seven-story model. For the upper story of the 21-story MLM model, every five story's masses were combined and given to the upper three lumped masses of the seven-story model, respectively. Every five story's stiffness was combined into one, and the equivalent stiffness was decided by taking the same yielding story drift angle with the 21-story MLM model. In this way, the upper three masses of the specimen model represented the 9th, 14th, and 19th stories of the 21-story building.

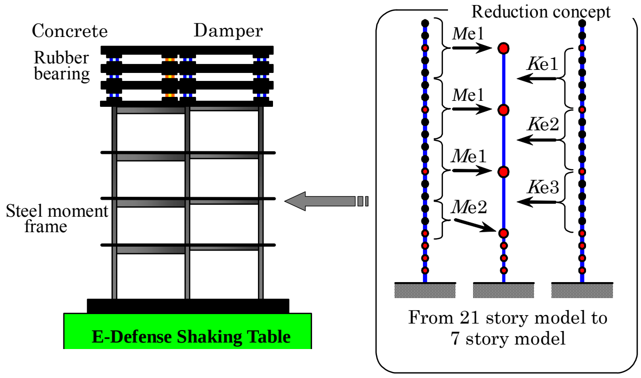


Figure 1 Specimen and reduction procedure

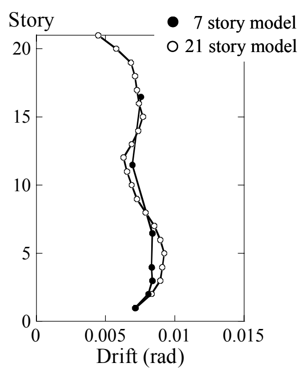


Figure 2 Max. story drift distribution

2.2 Numerical analysis results of lumped-mass model

Figure 2 shows the comparison of the two models subjected to the El Centro wave. By using an equivalent mass and stiffness system, the two models show the same maximum deformation and fundamental period, 2.4 seconds. The response representation of the prototype building under a seismic loading can be realized using the ideal reduction method.

2.3 Design of specimen

The specimen shown in Figs. 3 and 4 has a four-story steel frame portion and an equivalent system portion on the top. The specifications of the structural components are listed in Table 1.

2.3.1 Steel moment frame portion

The frame was designed following design policies and parameters with the prototype building. Member sizes such as depth and width-thickness ratio of web and flange were chosen in the design process by referring to the older member specifications. Figure 5 shows the old type of connection details used in the specimen. In the longitudinal direction, built-up H 600×200×9×19 and H 400×200×8×13 beams and shop weld connections with weld access holes of 1/4 circle which has a sudden change in geometry at the toe were adopted. In the transverse direction, honeycomb beams processed from a rolling-shape H 596×199×10×15 beam and field weld connections were adopted. The weld access hole at the slab side is the same as the one at the shop weld, and an ear-shaped one was adopted at the other side.

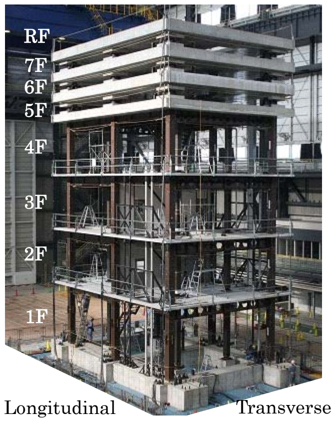


Figure 3 Overview of Specimen

Table 1 Specification of specimen

Column (SM490A)			□400x25
Beam	(SM490A)	G1	H600x200x9x19
		G2	H400x200x8x13
		G3	H500x200x9x16
		G4	H800x199x10x15
		G5	H650x199x9x14

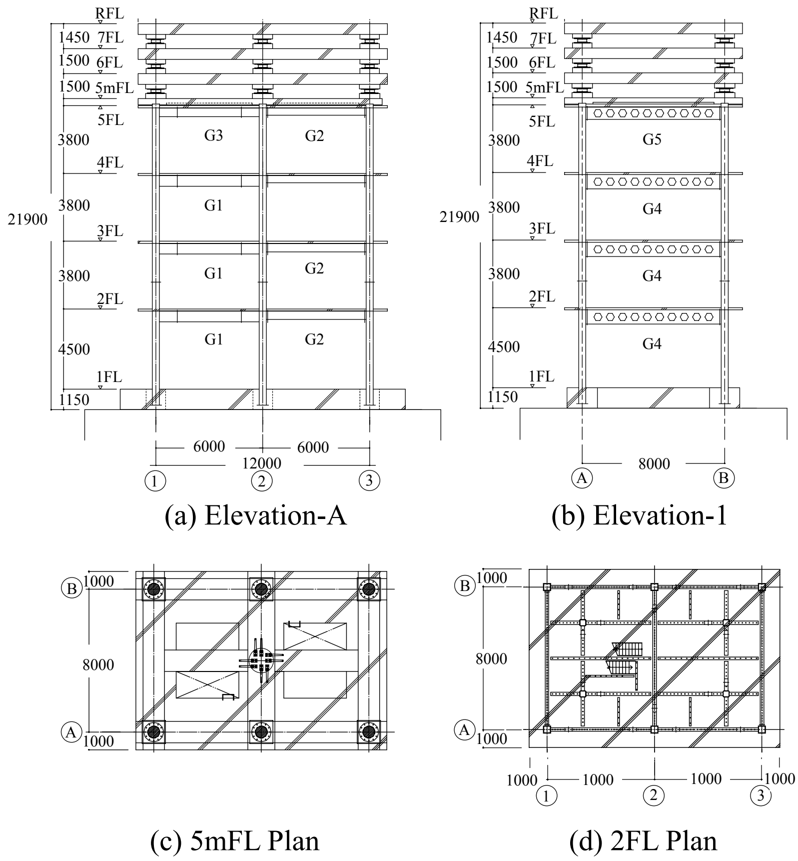
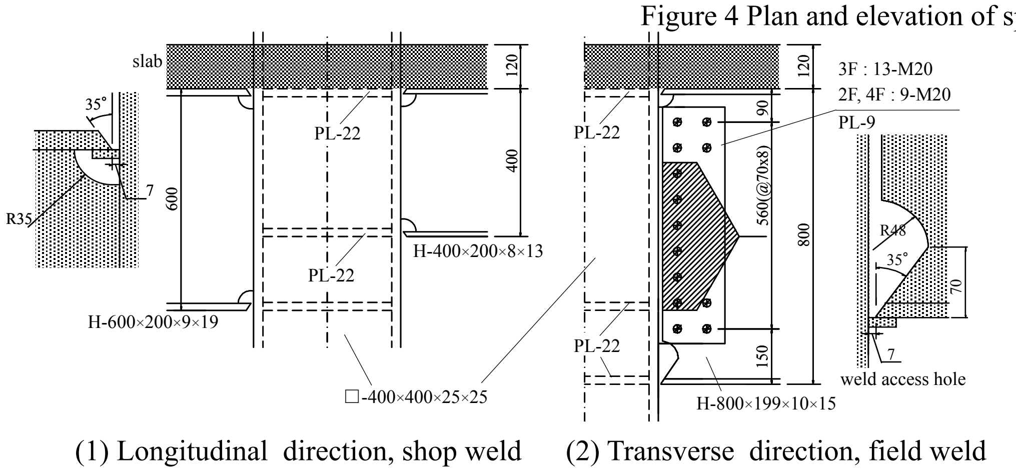


Figure 4 Plan and elevation of specimen

Table 2 Material Property

		σ_y (N/mm^2)	σ_u (N/mm^2)
	□400	343	515
G1	H600	357	532
G2	H400	426	576
G3	H500	366	532
G4	H800	373	526
G5	H650	344	514



(1) Longitudinal direction, shop weld (2) Transverse direction, field weld

Figure 5 Beam-to-column connection details

To ensure that plastic hinges form at beam ends, relatively strong cross-section columns made of cold-formed square tubes of 400×400×25, with inner-diaphragm at connections were adopted. The thickness of the reinforced concrete slab was 120mm at every floor (compressive strength of concrete was 30/mm^2). Specified and measured material properties are listed in Table 2. The reported measured properties represent the mean of coupon test of structure members. Charpy V-notch (CVN) impact tests were also conducted for the specimen and the deposited metals of welds. The measured CVN values of shop weld and field weld connections were 105 J and 88 J at 0^{o}C, respectively. The CVN value at 20^{o}C, which was the interior temperature when the test was conducted, was over 155 J.

2.3.2 Equivalent system portion

Figure 6 shows the design concept of the equivalent system. The objective skeleton curve mentioned in the MLM model was represented by six rubber bearings and one damper system arranged in the plane parallelly. By connecting steel damper and rubber bearing in series, the damper system is able to simulate the energy absorption and stiffness deterioration behaviors of the steel moment frame.
The strength and stiffness were adjusted by the combination of damper stiffness (K_d), yielding strength (Q_{dy}), and rubber bearing stiffness (K_{rs}). Story secondary stiffness was 0.6 times the initial stiffness for each story.

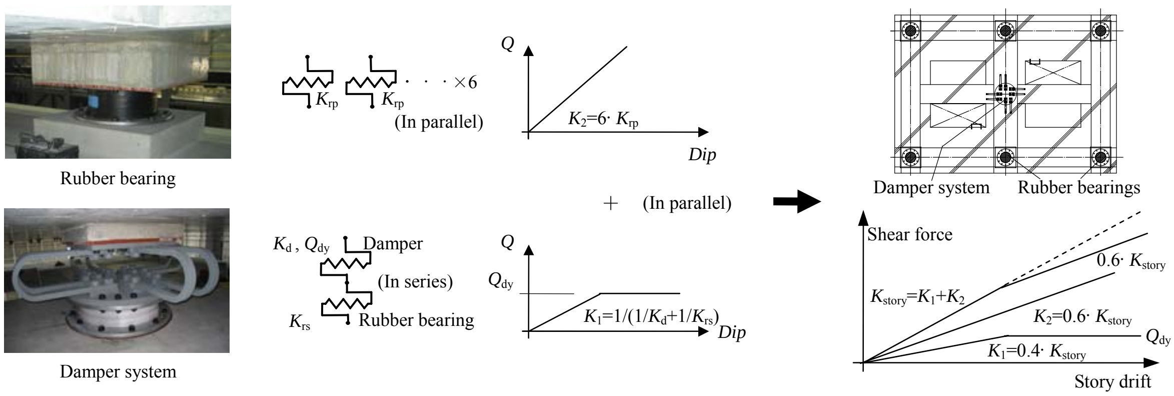
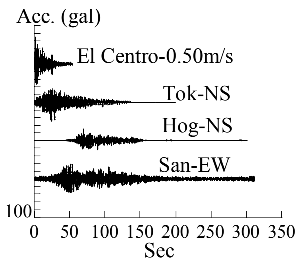


Figure 6 Design procedure of equivalent layers

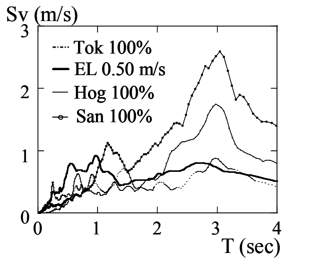
3. LOADING PROGRAM

Bi-direction shaking table tests were conducted at three levels with increasing magnitudes of earthquake excitation. A stronger directional wave was input in the longitudinal direction of the specimen because of the better deformation capacity of the shop weld connections (Yamada et al 2008). Figure 7 shows the time history and velocity response spectrum of input waves. Along with the1994 El Centro earthquake, which is considered as a standard design wave in the Japanese seismic design, three simulated waves were adopted.

Hog and San waves are long-period ground motions from Tokyo and Nagoya sites, generated by subduction zone source models in the southwest of Japan. Tok wave is an epicentral earthquake of Tokyo site. The loading program is shown in Fig. 8. Three levels of loadings were equal to the design earthquake force presented in Japan's seismic design code (Nakashima et al 1998). At Level 1, elastic loading is equal to small to medium earthquakes with maximum ground accelerations ranging between 80 and 100 gal (PGV is about 0.25 m/s). Structure systems are required to remain elastic and the inter-story drift angle should smaller than 0.5% during such earthquakes. At Level 2, plastic loading is equal to large earthquakes with maximum ground accelerations ranging approximately from 300 to 400 gal (PGV is about 0.50 m/s). For such earthquakes, some degree of damage to structures is permitted but no collapse, and the drift angle should be smaller than 1%. For the larger level, the San wave was input cyclically until beam-to-column connections fractured.



(a) Time history



(b) Velocity response spectrum

Figure 7 Longitudinal dir. input waves

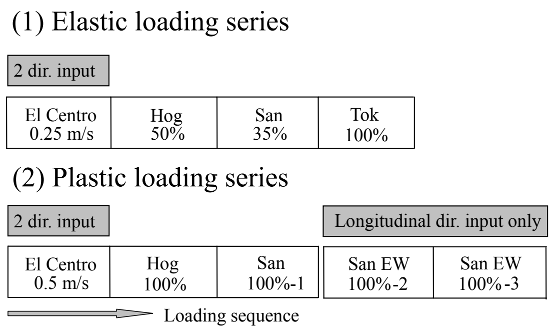


Figure 8 Loading program

4. TEST RESULTS

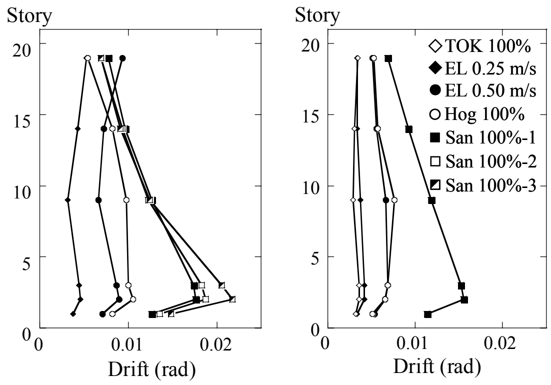
A 60gal frequency ranged from 0.2 to 20 H_z white-noise loading was conducted before the test. Drift angle of the frame portion was 0.4~0.5 %, and the maximum deformation of rubber bearings reached about 70% of the shearing strain. With the large-scale elastic white-noise loading, the first mode periods of the longitudinal and transverse directions were 2.13 and 2.24 seconds, respectively

4.1 Maximum deformation and global behaviors

Figure 9 summarizes the maximum inter-story drift responses of the test system under various levels of seismic loading. For Level 1 loading, the maximum deformation occurred at the 2nd story and the drift angle was small than 0.5 %. The maximum strain at the bottom flange of beam-to-column connections, which was 40 mm far from the column face, was 0.11%, and no structure damage was observed. For Level 2 El Centro wave loading, the maximum drift angle at the 2nd story was smaller than 1% and the maximum strains of the longitudinal, transverse direction connections were 0.23% and 0.56%, respectively. Connections yielded, and some steel mil scale (rust covering) of the bottom flanges dropped and yielding lines at the bottom of the connections were confirmed, though no obvious structure damage like local buckling or residual deformation was observed. For Level 3, San wave loading, the maximum drift angles of the 2nd story were larger than 1.7 % and 1.5 % in the longitudinal and transverse directions, respectively. The maximum deformation was concentrated significantly in the frame portion. Field weld connection beams fractured at three places on the bottom flange, but no cracks or obvious local buckling were observed in the shop weld connections under this loading. Longitudinal direction only input of the San wave was conducted twice until the fracture of the shop weld connections occurred. Maximum drift angles grow to 2.2 %, and fractures of bottom flanges occurred at the 3rd and 4th story beams, eventually.

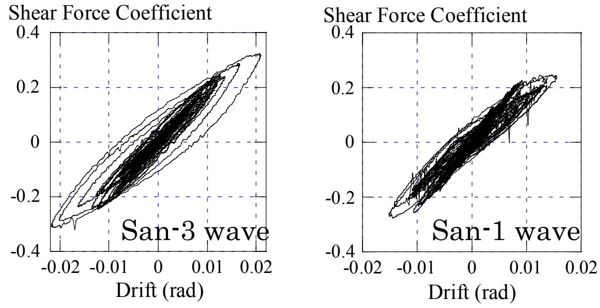
Figures 10 (a) and (b) show the normalized shear force coefficient versus drift angle relationship of the 2nd story for longitudinal and transverse directions of specimen subjected to San-3 and San-1 wave excitations, respectively. After fractures of beam-to-column connections were developed, story stiffness decreased about 20 %. Story resistance and stiffness didn't decrease rapidly after beam-to-column connections fractured in two directions for the specimen.

Figures 11(a) and (b) show the story ductility ratio (μ) and normalized story cumulative plastic deformation (η) of the 2nd and 6th stories generated by three seismic loadings. Story ductility ratio was defined as the maximum deformation divided by yielding deformation. Normalized story cumulative plastic deformation was defined as the story energy absorption generated by cyclic loading divided by elastic energy absorption as shown in Fig. 12 (Chusilp et al 2004). 0.5% drift angle was considered as the yielding deformation under these calculations. In Fig. 11(a), maximum deformation generated by two similar level loadings, El Centro wave and a long-period ground motion Hog wave, were close to each other but the η value of the Hog wave can be four times El Centro wave's, as shown in Fig. 11(b). Comparing the η value with San wave's, the largest long-period ground motion can generate 15 times the one generated by the design wave.



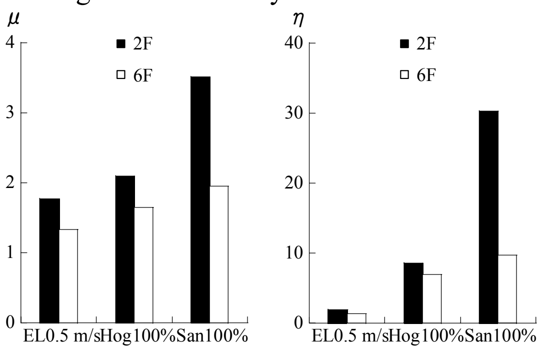
(a) Longitudinal (b) Transverse

Figure 9 Max. story drift distribution



(a) 2F- Longitudinal (b) 2F- Transverse

Figure 10 Story shear force – deformation



(a) Ductility ratio (b) Cumulative plastic deformation ratio

Figure 11 Longitudinal direction deformation

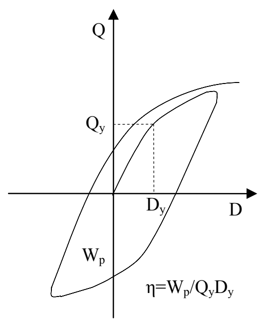
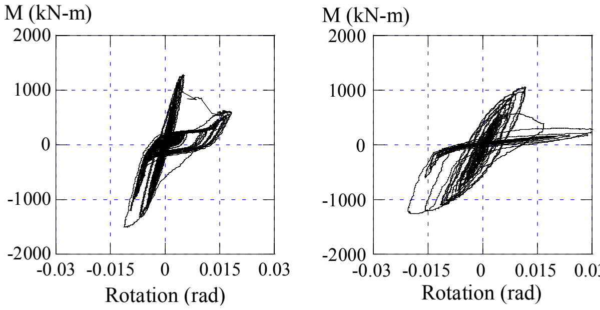


Figure 12 Definition of cumulative deformation

4.2 Fracture behavior of beam-to-column connections

Figures 13 and 14 show bending moment versus rotation relationships for fractured beam-to-column connections and the fracture pictures. In Fig. 13(a), the bottom flange of field weld connection fractured without large plastic deformations, and resistance decreased rapidly in positive bending. Residual resistance recovered when the rotation exceeded 0.01 rad because of the contact of slid bolts with web metal. Resistance in the negative bending remained and shaped a slip type behavior because of compression due to the fractured beam flange compacting with the column face. In Fig. 14 (a), fracture of this connection occurred along the weld boundary where the toe of the weld access hole is located. Also, several cracks were observed at the same location of other unfractured field weld connections. It's believed that stress was extremely concentrated on the weld access hole end, as initial cracks were formed from here and grew to the fracture of flange eventually.

In Fig. 13 (b), fracture was developed after three times of San wave input, and shows similar slip type behavior to that of field weld connections. Very limited resistance remained, however, in the positive bending after the flange and web fractured as shown in Fig. 14 (b). Initial cracks developed from base metal at toe of the 1/4 circle weld access hole. Also, several cracks were observed in the base metal of the beam flange of other unfractured shop weld connections. After fractures of bottom flanges were formed, cracks of web extended with the growing rotation.



(a) 3F Field weld (b) 3F Shop weld

Figure 13 Moment-rotation relationship of beam ends

(a) 3F Field weld (b) 3F Shop weld

Figure 14 Fracture of beam ends

5. CONCLUSIONS

An equivalent reduction method was proposed and used to reduce an 80 m high building to a test system with a full-scale steel moment frame and an equivalent system on the top. The test system was designed according to the statistical results and used older types of structure member details to represent the characteristics of early high-rise buildings. At various levels of seismic loading using E-Defense shaking table, the seismic performance of this system was confirmed. Major observations obtained from the study are summarized as follows:

1. Maximum drift angles of substructure specimens are smaller than 0.5% and 1.0% under medium and large earthquakes, respectively. Beam-to-column connections yielded, and the strain value was up to 0.56%, but no cracks or obvious residual deformation observed. Columns remained elastic.
2. Drift angles of longitudinal and transverse directions of the specimen grew to 1.76 % and 1.55%, respectively, when it was subjected to a long-period ground motion larger than large earthquakes. Field weld connections fractured at three places in the bottom flanges but no serious damages, such as the cracks observed in the shop weld connections, was observed. Global deformation was concentrated in the steel frame portion significantly after the specimen became plastic. By inputting the same wave longitudinal direction only cyclically, the shop weld connections finally fractured during the second time input and drift angle grew to 2.18% eventually. Shop weld connections show better cumulative deformation capacity than field weld ones.

Story stiffness decreased about 20 % in the two directions, but the story hysteresis behavior remained stable, with no severe strength deterioration after connections fractured. Story normalized cumulative plastic deformation generated by a long-period ground motion is 30 which is 15 times of the one generated by design wave.

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