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NUMERICAL SIMULATION OF E-DEFENCE-SPECIMEN SUBJECTED TO VARIOUS RECORDED GROUND MOTIONS

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ABSTRACT

A full-scale experiment to simulate collapse of a 4-story steel building was conducted using E-Defense shake table facility. This paper deals with the analytical simulation of the specimen based on the collaborative structural analysis (CSA) system. The reliability of the system was confirmed by comparing analytical results to test results. The additional numerical simulations for three other recorded ground motions were conducted. The scale was specified as the maximum ground velocity is 100 cm/s, which is twice the level 2 of earthquake in Japanese design practice. The specimen, which conforms Japanese Building Standard law, collapsed by Hachinohe, while it did not by El Centro and Taft.

Introduction

A full-scale experiment to simulate collapse of a 4-story steel building was conducted using E-Defense shake table facility. The specimen consists of beams of wide-flange sections, columns of rectangular hollow sections, concrete slabs with deck plates, and column bases with anchor bolts. The collapsing of the specimen was characterized by deteriorating behavior caused by local buckling of column at the first story. This paper deals with the analytical simulation of the specimen based on the collaborative structural analysis (CSA) system. Through the comparison of analytical and test results, the reliability of CSA system will be shown. The additional numerical simulations for three other recorded ground motions will be conducted. The seismic performance on the complete collapse for ordinary steel buildings in Japan will be discussed.

Full-Scale Building Specimen for Analysis

The specimen shown in Fig. 1 was so designed to conform to Japanese Building Standard

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law. The building has plan of 10 m in the longitudinal direction (Y) and 6 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 material properties obtained by coupon tests. Ribbed metal decks of 75 mm high are connected to the beams through studs that are welded to the beam top flanges on the second to fourth floor. Wire-meshes are placed above the metal deck sheets, and concrete of 100 mm thick is placed on site. Flat metal decks are used on the roof floor, and reinforced concrete slab of 150 mm thick is placed on site. Fully composite action is expected between the steel beams and concrete slab. The column bases are the exposed type with base plate of 50 mm thick and eight anchor bolts of M36-diameter. The anchor bolts are so designed that he bending strength of column base is higher than that of connecting column.

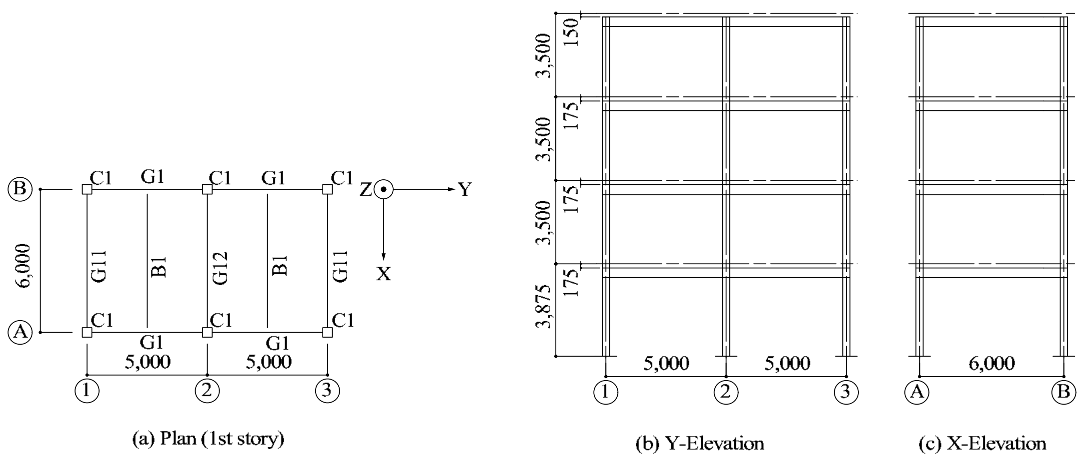


Figure 1. Framing plan and elevation of the specimen.

Table 1. Member schedule.

Floor	G1 (SN400B)	G11 (SN400B)	G12 (SN400B)	B1 (SS400)	Story	C1,C2 (BCR295)
R	H-346×174×6×9	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	H-350×175×7×11	3	SHS-300×300×9
3	H-396×199×7×11	H-400×200×8×13	H-400×200×8×13	H-350×175×7×11	2	SHS-300×300×9
2	H-400×200×8×13	H-400×200×8×13	H-390×200×10×16	H-350×175×7×11	1	SHS-300×300×9

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

Table 2. Material properties by coupon tests.

		Yield stress (MPa)		Tensil strength (MPa)	
	Section	Flange	Web	Flange	Web
Beam	H-340×175×9×14	309	355	443	468
	H-346×174×6×9	333	382	461	483
	H-350×175×7×11	302	357	441	466
	H-390×200×10×16	297	317	451	458
	H-396×199×7×11	311	369	460	486
	H-400×200×8×13	326	373	454	482
Column	RHS-300×300×9	2~4 story	1,4 story	2~4 story	1,4 story
		332	330	419	426
Base plate (PL-50)		378		501	
Anchor bolt (M36)		336		507	

Collaborative Structural Analysis System

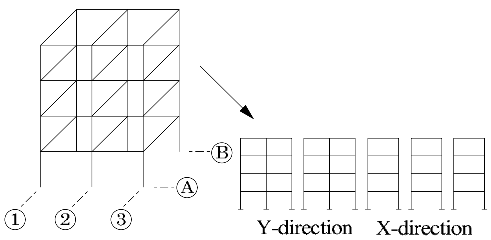
A collaborative structural analysis system is proposed by Tada et al (2004), which is capable of performing highly sophisticated structural analysis by utilizing the beneficial features of existing individual structural analysis programs developed by individual researchers. In the system, the simulated structure is divided into multiple substructures, and each substructure is analyzed by an individual program. Specifically, the host program formulates and solves the equations of motion for the entire structure, and sends boundary displacements to the corresponding station programs. The station programs run detailed analyses, and return forces associated with the degrees of freedom at the boundaries through a condensation procedure. The procedure is conducted step by step, and the data are exchanged through Internet.

The program, NETLYS, is used as a host program. It deals with the global equation of motion, to which numerical information of all sub-structures is involved. For the purpose of general combination of various programs, the explicit integration scheme of operator splitting (OS) method (Nakashima 1990) is used. It requires only restoring forces of sub-structures, and does not require tangential stiffness of sub-structures. Thus, general-purpose program, such as MARC, can be used as a station program. Here, a modified OS method (Tada and Pan 2007) is used for the numerical stability in geometrical high nonlinear analyses.

All frames of 1 to 3 in X-direction and A and B in Y-direction are modeled in the same plane as shown in Fig. 2. By postulating "rigid slab (in-plane) hypothesis", every horizontal displacement in the same floor is numerically related by the linear combination of horizontal displacements in X- and Y-directions and rotation at the center of the floor. Vertical displacements at the conjunction nodes of orthogonal frames are equalized. Columns are modeled and analyzed in 3D manner, bi-axial bending and longitudinal axial force are considered. While, beams and beam-to-column panels are modeled and analyzed in 2D manner. Thus, 3D analyses are conducted.

Figure 3 shows the formation of sub-structures and program names for each sub-structure. The program, COMPO, is used for analyzing composite beams that consist of steel beam and concrete slab. It can consider the composite action in plus bending, and the crack-opening of concrete slab in minus bending. It can simulate the complex cyclic behavior of composite beams. The general-purpose program, MARC, is used for analyzing columns of rectangular hollow sections in the first story. Shell elements are used to consider the local buckling due to axial force and bending moment. It can simulate the deteriorating post-buckling behavior in detail. As we know that the collapsing of the specimen is mainly characterized by deteriorating behavior at the first story-columns in the pre-analyses (Tada, Ohsaki et al 2007), the column ends at the second to fourth stories are simply modeled using rotational springs which demonstrate deteriorating behavior of local buckling and were used in the pre-analyses (Tada, Ohsaki et al 2007).

Beam-to-column panels are analyzed in detail using subroutines which are installed in NETLYS. Exposed column bases are modeled by elastic rotational springs whose stiffness corresponds to the state that the base plate perfectly touches to the foundation.



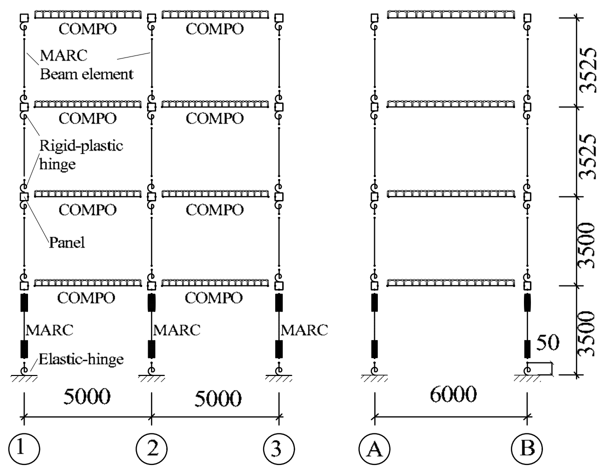


Figure 2. 3D model consists of plane frames. Figure 3. Formation of sub-structures.

Pushover Analysis

The result of pushover analysis is shown in Fig. 4. The constant vertical load and varying horizontal forces proportional to seismic design forces are applied. Story shear (Q) -drift angle (r) relation for each story is shown in Fig.4 (a). The distributions of plasticized sections are shown in Figs. 4 (b) and (c). Figure 4 (b) represents the case where horizontal forces reach the maximum value, and Fig. 4 (c) represents the case where the analysis reaches the end. Solid circle represents the plastic hinges at beam and column, and solid square represents shear yield at beam-to-column panel. The plasticized sections spread all over the structure as the horizontal forces increase until they reach the maximum value. The horizontal forces deteriorate after the local buckling occurs at the columns of the first story. As the horizontal strength deteriorates the plasticized sections are localized at the first story. The final collapsing mechanism is column-yield type with local buckling at both ends of the first story-columns.

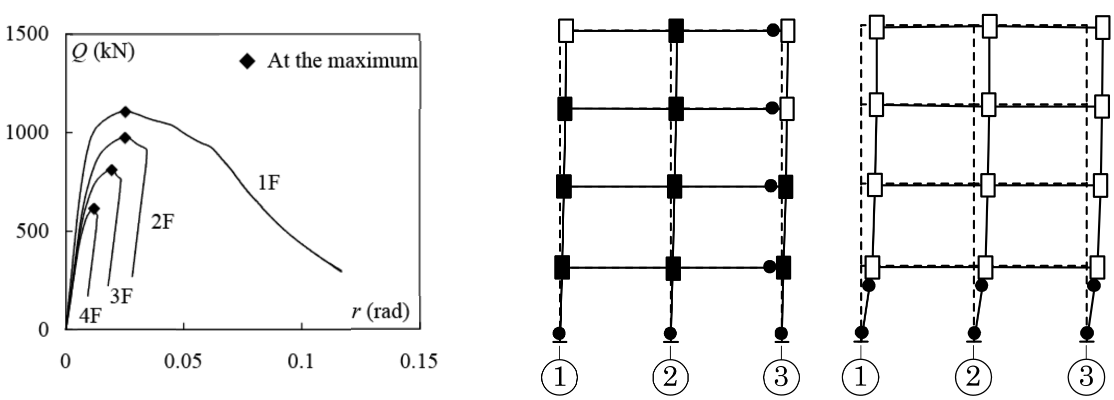
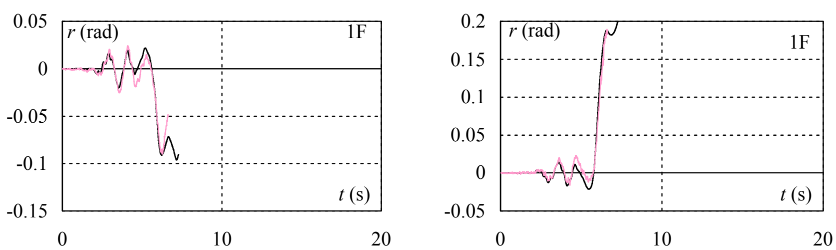


Figure 4. (a) Story shear-drift angle relation. (b) At the maximum. (c) At the end.

Comparison of Results by Shaking Table Test and Numerical Analysis

Time history analyses are conducted to compare with the results of shake table tests. The earthquake ground motion recorded in Hyogoken-Nanbu earthquake, Takatori (Nakamura et al 1995, FD Serial No. T065), was adopted in the tests, and the table motion recorded in the tests are used for the analyses. EW, NS, and UD components are used for the X-, Y-, and Z-directions, respectively. Rayleigh damping is considered, and damping ratio of 2% is assigned to the first and second vibration modes in X-direction. Fundamental and the second periods are 0.81 and 0.26 s in X-direction, 0.78 and 0.25 s in Y-direction.

The analytical and test results are shown below. Here, solid lines show the analytical results, and gray lines show the test results reported by the Building Collapse Simulation Working Group. Figure 5 shows the time history of story drift angle (r) at the first story, where Figs. (a) and (b) show the cases in X- and Y- directions, respectively. Figure 6 shows the orbit of story drift angles at the first story, where ordinate and abscissa represent drift angles in X- and Y- directions, respectively. Both analytical and test results show that the specimen collapses in minus X-direction and plus Y-direction. Figure 7 shows the story shear (Q) - drift angle (r) relation at the first story. The deterioration of story shear can be observed. Figure 8 shows the distribution of plasticized sections at the end of analysis. The plasticized sections are localized at the column-ends at the first story as observed in the pushover analysis. In Figs. 5 to 8, the analytical results can express well the test results obtained by the shake table test.



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

Figure 5. Time history of story drift angle at the first story.

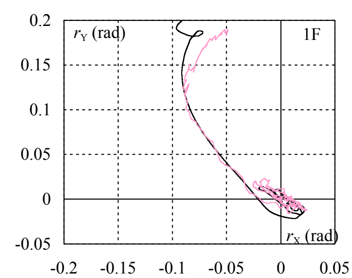
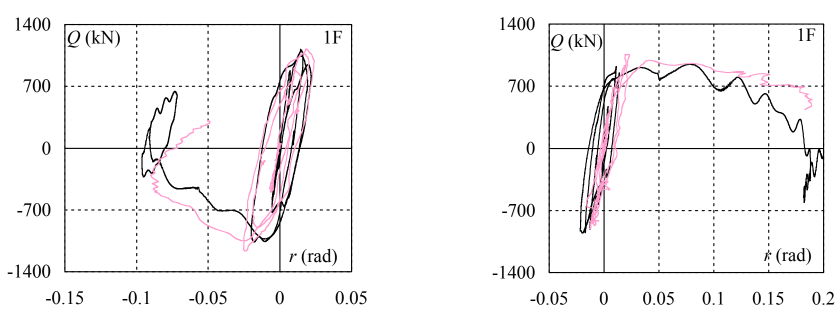


Figure 6. Orbit of drift angle.



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

Figure 7. Story shear-drift angle relation at the first story.

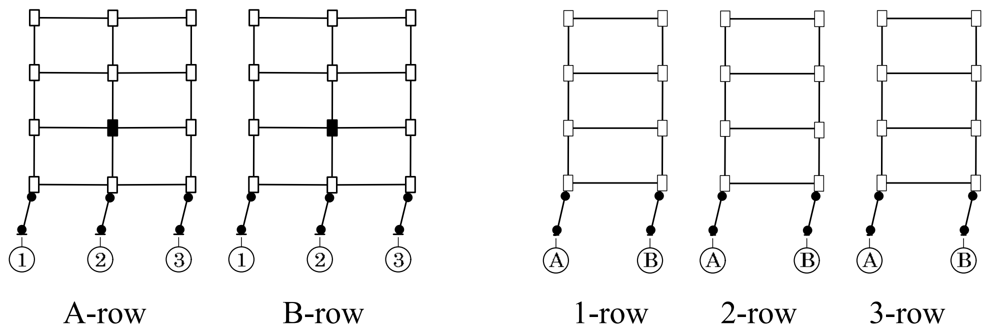


Figure 8. Final collapse mechanism.

Numerical Simulation for Various Recorded Ground Motions

The merit of numerical simulations, compared to shake table tests, is the capability of parametric simulations in many cases. Here, time history analyses for El Centro (California 1940), Taft (California 1952), and Hachinohe (Aomori, Japan 1968) are conducted. The amplitudes of the ground motions are specified as the maximum velocity is 100 cm/s. According to the design practice of high-rise buildings in Japan, the ground motion of level 2 is specified by 50 cm/s. Thus, the ground motions considered here are the twice the level 2 of earthquake.

Velocity response spectrums of all ground motions are shown in Fig. 9. Figure (a) corresponds to X-direction of the specimen, and (b) to Y-direction. The ground motion of Takatori is non scaled, and other three motions are scaled as the maximum velocity be 100 cm/s.

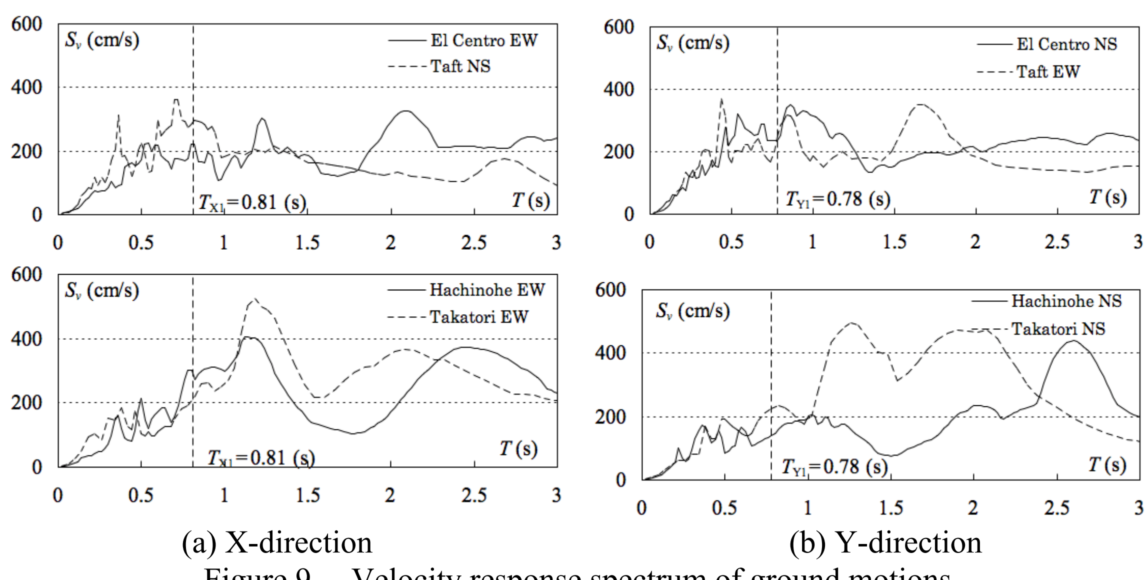
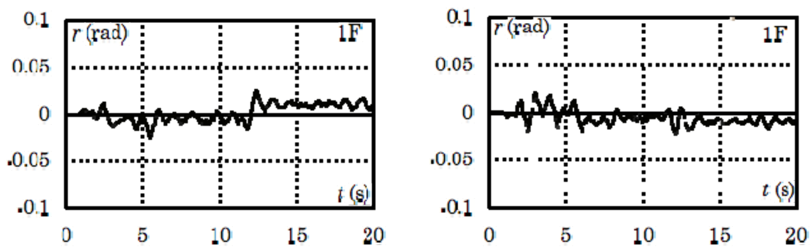
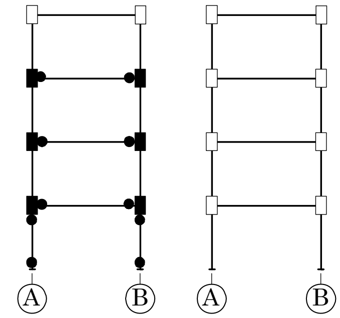


Figure 9. Velocity response spectrum of ground motions.

Time history of story drift angle (r) at the first story and distribution of plasticized sections for three ground motions are shown in Figs. 10 to 15. Figures 10 and 11 correspond to El Centro, Figs. 12 and 13 correspond to Taft, and Figs. 14 and 15 correspond to Hachinohe. The specimen does not collapse for El Centro and Taft, though large story drift angle, as much as 1/40 rad for El Centro and 1/28 rad for Taft, experiences. On the other hand, the specimen collapses for Hachinohe. The collapse mechanism is the column-yield type of the first story as same as Takatori.



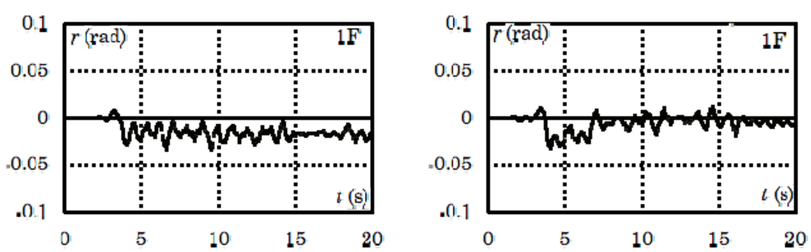


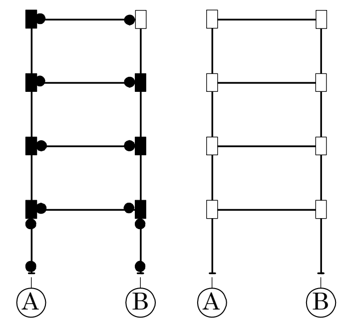
(a) X-direction (b) Y-direction

Figure 10. Time history of story drift angle (El Centro).

experienced final state

Figure 11. plasticized sections.



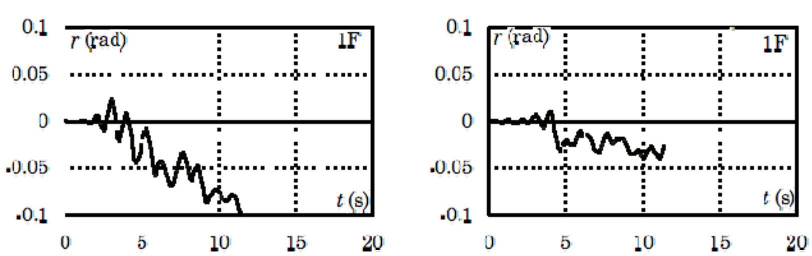


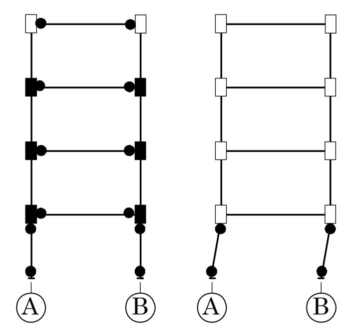
(a) X-direction (b) Y-direction

Figure 12. Time history of story drift angle (Taft).

experienced final state

Figure 13. plasticized sections.





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

Figure 14. Time history of story drift angle (Hachinohe).

experienced final state

Figure 15. plasticized sections.

Conclusions

This paper has discussed analytical simulation of collapsing behavior of 4-story steel building which was tested using E-Defense shake table facility. The specimen conforms to the Japanese Building Standard law. Analytical simulations have been conducted using Collaborative Structural Analysis (CSA) system. In case of non-scaled Takatori, the numerical simulation well express the test results. The plasticized sections spread all over the structure, and then the damage was localized at the first story. As the localization proceeded, the bending strength of columns at the first story heavily deteriorated due to local buckling, and finally it lead the specimen to collapse.

The numerical simulations for other ground motions have also been conducted. The scale was specified as the maximum ground velocity is 100 cm/s, which is twice the level 2 of earthquake in Japanese design practice. The specimen collapsed for Hachinohe, while it did not for El Centro and Taft.

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