ANALYTICAL SIMULATION UTILIZING COLLABORATIVE STRUCTURAL ANALYSIS SYSTEM

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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. 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. The CSA system is capable of performing highly sophisticated structural analyses utilizing the beneficial features of existing individual structural analysis programs. Several programs are being used to simulate various components of the whole frame. 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. Shell elements are used for modeling to consider the local buckling due to axial force and bending moment. It can simulate the deteriorating post-buckling behavior. 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. The analytical results are compared with test results of shake table. The effects of structural components to overall collapsing behavior of the specimen will be discussed.

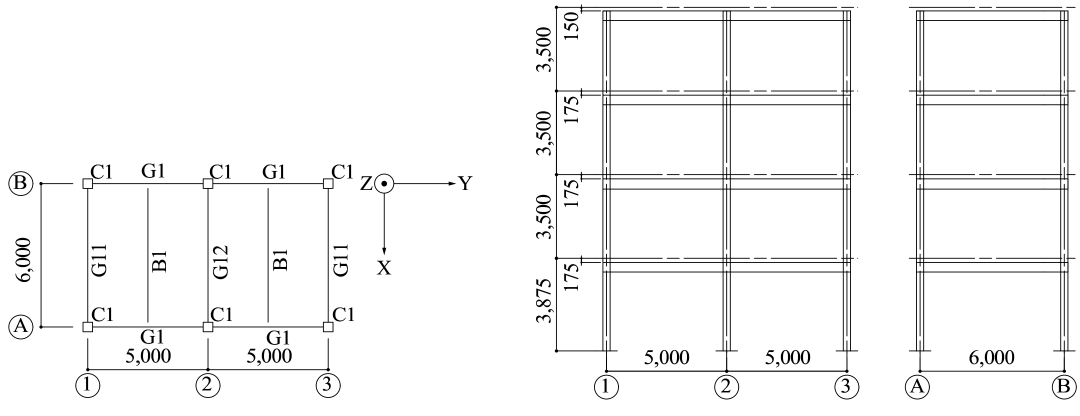
KEYWORDS: collaborative structural analysis, numerical analysis, local buckling, full-scale steel building, collapse

1. INTRODUCTION

The collaborative structural analysis (CSA) system [Tada et al 2004, 2007] is capable of performing highly sophisticated structural analyses utilizing the beneficial features of existing individual structural analysis programs. Several programs are being used to simulate various components of the whole frame. This paper deals with the analytical simulation of a 4-story steel building specimen, which is experimented using E-defense shake table facility, based on the collaborative structural analysis (CSA) system. The collapsing of the specimen caused by local buckling of column ends at the first story will be analyzed in detail, and compared with test results.

2. FULL-SCALE BUILDING SPECIMEN FOR ANALYSIS

The specimen building is shown in Figure 1. 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 which 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.



(a) Plan (1st story) (b) Y-Elevation (c) X-Elevation

Figure 1 Framing plan and elevation of the specimen

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	RHS-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	RHS-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	RHS-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	RHS-300×300×9

Table 1 Member schedule

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	

3. PSEUDO THREE-DEGREE ANALYSIS

Pseudo three-degree (3D) analyses are conducted by combining 2D frames under the consideration of geometrical conditions. 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 Figure 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, pseudo 3D analyses are conducted using 2D frame analysis program.

4. FORMATION OF STRUCTURAL ANALYSIS PROGRAMS

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.
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.

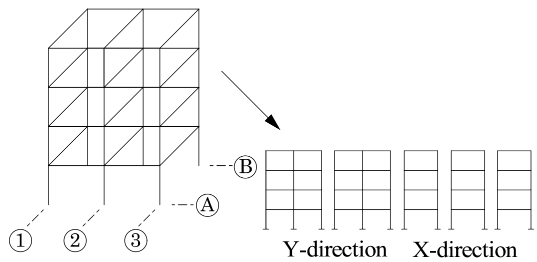


Figure 2 Pseudo 3D model

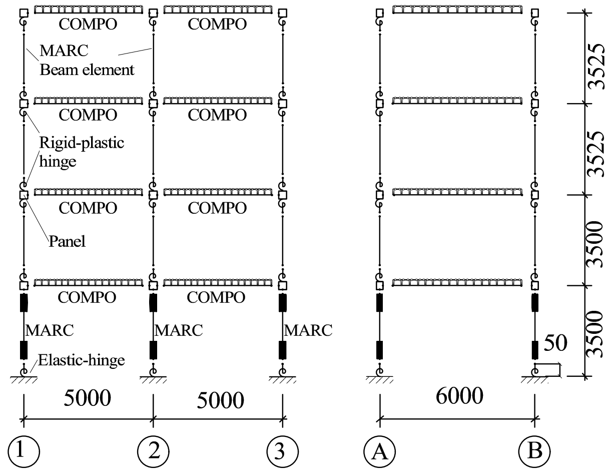


Figure 3 Formation of sub-structures

5. OUTLINE OF INDIVIDUAL PROGRAM

5.1 COMPO

Structural model of a composite beam analyzed in the program, COMPO, is shown in Figure 4. Steel beam is modeled by beam element which includes bending, axial and shear deformations, and whose plastic behavior is characterized by general yield hinges. Concrete slab is modeled by pin-ended strut which includes axial deformation. These line elements are located at the center of respective sections. As sufficient amount of studs are welded, slip between slab and top flange of steel beam is ignored. So, line elements, representing steel beams and concrete slabs, are discretely connected by rigid bars at some interval. The effective width of concrete slab is so determined after the standard of Architectural Institute of Japan [AIJ 1985]. Tri-linear stress-strain relation which has double yield surfaces is adopted for steel beams. These yield surfaces are defined by axial force and bending moment. The size of the first yield surface is determined by 0.7 of the actual yield stress obtained by coupon tests as shown in Table 2, and the size of the second yield surface is determined by the yield stress itself. The second stiffness is 0.2 of Young's modulus and the third stiffness is 0.02 of Young's modulus. Combined kinematic and isotropic hardening is adopted and the ratio of isotropic hardening is 20%.

Tetra-linear stress-strain relation as shown in Figure 5 is adopted for concrete slabs. The maximum stress is calculated from bearing strength of concrete slab at the column-faces. The bearing strength is calculated by [Inoue et al 1990] which utilizes Mohr-Coulomb's criterion. Standard concrete strength of 21 N/mm^2 is used. The deteriorating stiffness is determined by referring to [Tanaka and Tada 2007], in which deteriorating behavior is discussed using the results of push-out test of concrete slab.

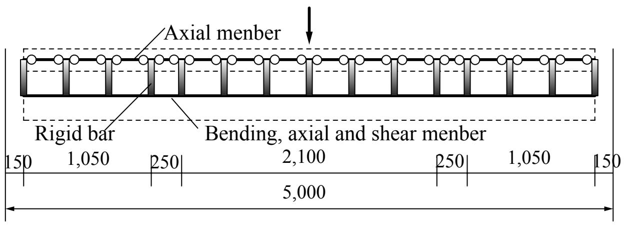


Figure 4 Structural model of composite beams

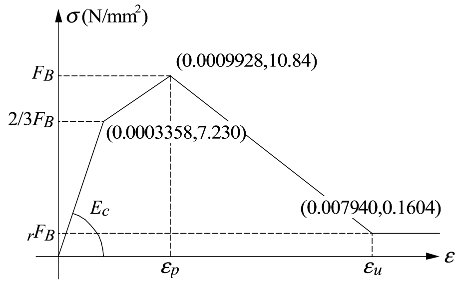


Figure 5 Stress-strain relation of concrete slab

5.2 MARC

Structural model of RHS-columns at the first story analyzed in the general purpose program, MARC, is shown in Figure 6. Both ends of the column are modeled by thin shell elements, and intermediate portion is modeled by elastic beam element which includes bending, axial and shear deformations. Kinematic hardening bi-linear stress-strain relation is adopted for steel beams. Yield stress of steel is determined by the results of coupon tests as shown in Table 2. The second stiffness is 0.002 of Young's modulus.

5.3 MECHANICAL MODEL OF BEAM-TO-COLUMN PANEL

Structural model of beam-to-column panels proposed in [Kuwahara et al 1995] as shown in Figure 7 is adopted. Panels, flanges and diaphragms are divided into flange and web elements. By considering not only shear deformation but also longitudinal deformation, stress re-distribution after shear yielding of web can be demonstrated, and significant hardening after shear yielding can be obtained. Combined kinematic and isotropic hardening tri-linear stress-strain relation is adopted. The second and third stiffness are determined by [Kuwahara et al 1995], where, yield stress in Table 2 and tensile strength of 426 N/mm^2 is used.

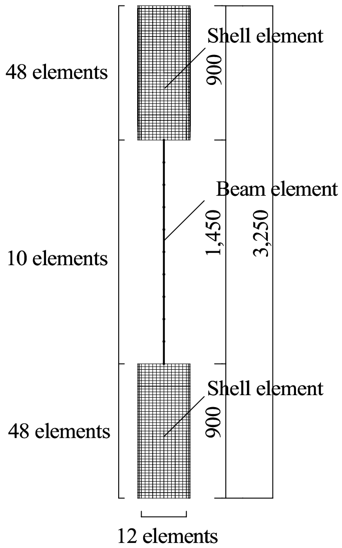


Figure 6 Structural model of RHS-column

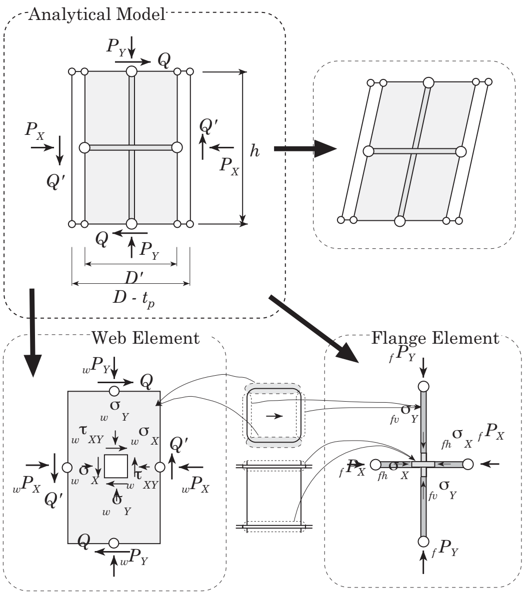


Figure 7 Structural model of beam-to-column panel

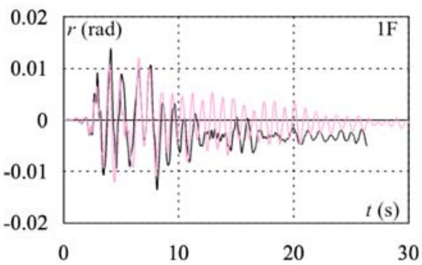
6. SEISMIC RESPONSE BY CSA-SYSTEM

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. The cases where ground motions are scaled 0.6 times and non-scaled are analyzed. Rayleigh damping is considered, and damping ratio of 2% is assigned to the first and second vibration modes in X-direction. Fundamental periods obtained by pseudo 3D model are listed in Table 3.

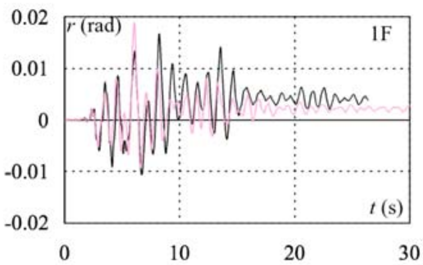
Table 3 Fundamental periods (s)

X-direction		Y-direction	
1st	2nd	1st	2nd
0.81	0.26	0.78	0.25

The results when the ground motion is scaled 0.6 times 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 8 shows the time history of story drift angle (*r*) in the first story, where figure (a) and (b) show the cases in X- and Y-directions, respectively. Figure 9 shows the orbit of story drift angles in the first story, where ordinate and abscissa represent drift angles in X- and Y-directions, respectively. The maximum story drift angles are +0.014 and -0.014 rad in X-direction, while +0.012 and -0.012 rad by tests, and +0.017 and -0.011 rad in Y-direction, while +0.019 and -0.009 rad by tests. Thus, the analytical results show good correspondence to test results. Figure 10 shows the story shear (*Q*) - drift angle (*r*) relation in the first story, and figure 11 shows the moment ($_cM$) - rotation ($_c\theta$) relation of the column at the A-2 row in the first story. The story shear calculated by column-strain is adopted in the gray line of test results. Elastic stiffness and the degree of plasticity in *Q*-*r* and M-θ relations are corresponding well between the analytical and test results, while analytical results of the maximum plus values in Y-direction are little smaller than test results. Figure 12 shows the energy dissipation mechanism, where solid circles and squares represent plasticity in steel elements and beam-to-column panels, respectively. The plasticized sections are distributed all over the structure, thus, the design concept of beam-yield mechanism is successfully achieved. Figure 13 shows the moment ($_bM$) - rotation ($_b\theta$) relation of the composite beam at the 1-row-end of A-row in the second floor. The rotation in the analytical results is defined by the end-angle measured from the line connecting both ends of beam, while, the rotation of test results is defined by the angle between the end-section and the mid-section, 1850 mm apart from the end. We can find that the plasticity is not severe, and the stiffness in plus bending is higher than that of minus bending because of the composite action. Figure 14 shows the moment ($_pM$) - shear angle ($_p\theta$) relation of the beam-to-column panel in Y-direction at the A-2 row in the second floor. The maximum strength in the analytical results is smaller than the test results.



(a) X-direction



(b) Y-direction

Figure 8 Time history of story drift angle in the first story (0.6 x Takatori)

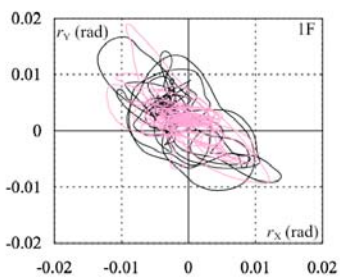
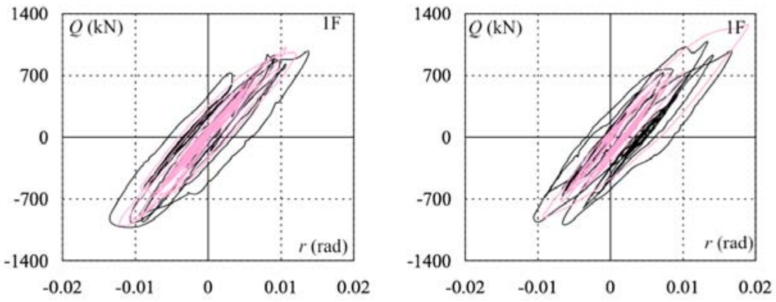


Figure 9 Orbit of story drift angle



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

Figure 10 Story shear - drift angle relation in the first story (0.6 x Takatori)

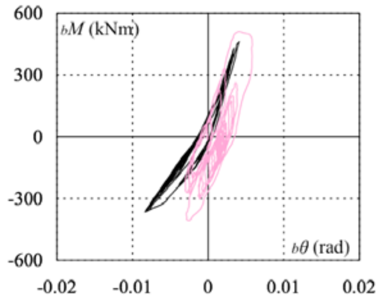
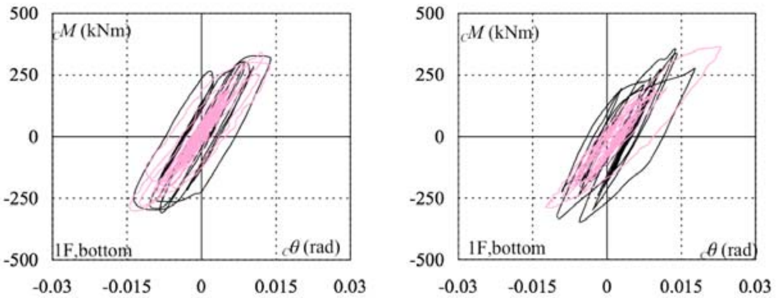


Figure 13 Moment-rotation relation of composite beam at 1-row-end of A-row in the second floor (0.6 x Takatori)



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

Figure 11 Moment - rotation relation of column at the A-2 row in the first story (0.6 x Takatori)

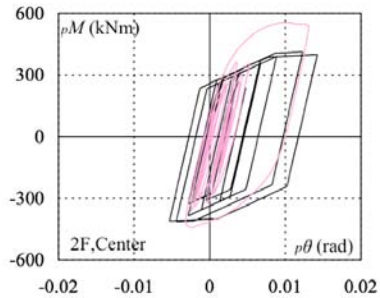
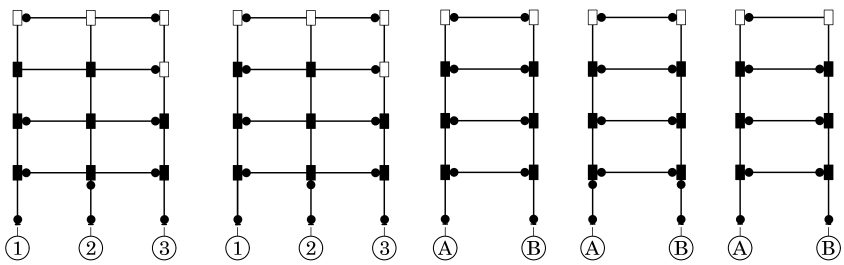


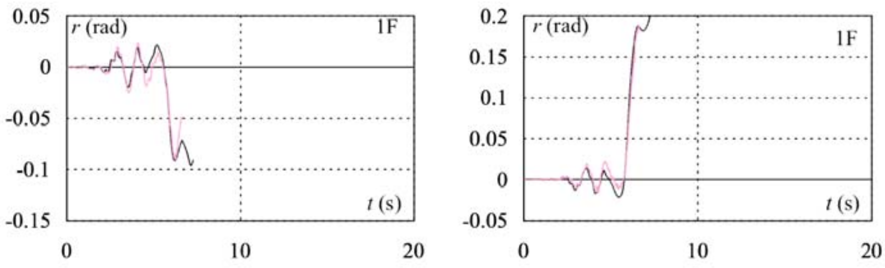
Figure 14 Moment-rotation relation of beam-to-column panel in Y-direction at A-2 row in the second floor (0.6 x Takatori)

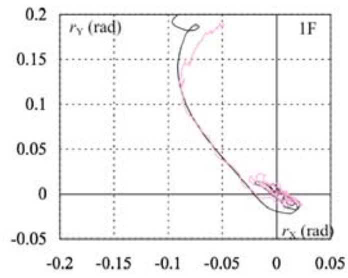


A-row B-row 1-row 2-row 3-row

Figure 12 Energy dissipation mechanism (0.6 x Takatori)

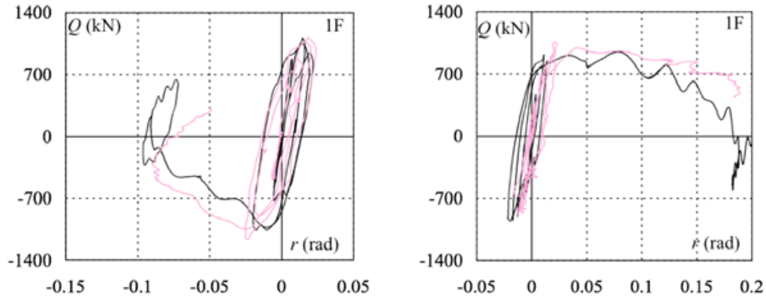
The results when the ground motion is non-scaled are shown below. Figure 15 shows the time history of story drift angle (r), and figure 16 shows the orbit of story drift angles in the first story. Both analytical and test results show that the specimen collapses in minus X-direction and plus Y-direction. Figure 17 shows the story shear (Q) - drift angle (r) relation in the first story, and figure 18 shows the moment (${}_cM$) - rotation (${}_c\theta$) relation of the column at the A-2 row in the first story. We can see that the bending strength of columns in the first story deteriorates due to local buckling, and it leads the specimen to collapse. Figure 19 shows the distribution of plasticized sections at the end of analysis. Plastic hinges are localized to the column-ends at the first story, while plastic hinges at the upper floors are unloaded. The mechanism developed during collapse is the soft first story type as observed in the test. Figure 20 shows the moment (${}_bM$) - rotation (${}_b\theta$) relation of the composite beam at the 1-row-end of A-row in the second floor, and figure 21 shows the moment (${}_pM$) - shear angle (${}_p\theta$) relation of the beam-to-column panel in Y-direction at the A-2 row in the second floor. The degree of plasticity in these relations is almost the same with those in case of 0.6 times (Figure 13 and 14). Thus, seismic damage is localized to the columns of the first story by the severe ground motion.





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

Figure 15 Time history of story drift angle in the first story (1.0 x Takatori) Figure 16 Orbit of story drift angle



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

Figure 17 Story shear - drift angle relation in the first story (1.0 x Takatori)

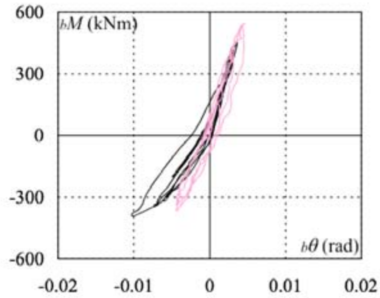
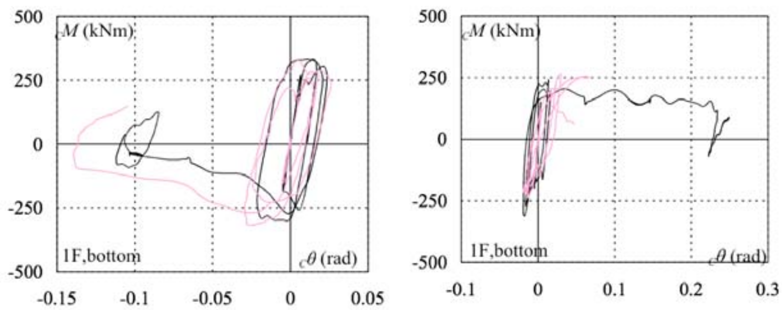


Figure 20 Moment-rotation relation of composite beam at 1-row-end of A-row in the second floor (1.0 x Takatori)



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

Figure 18 Moment - rotation relation of column at the A-2 row in the first story (1.0 x Takatori)

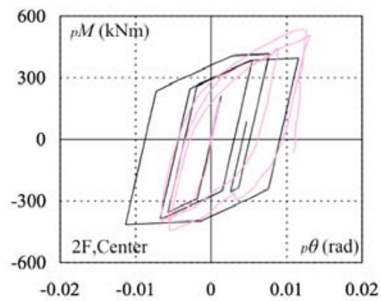
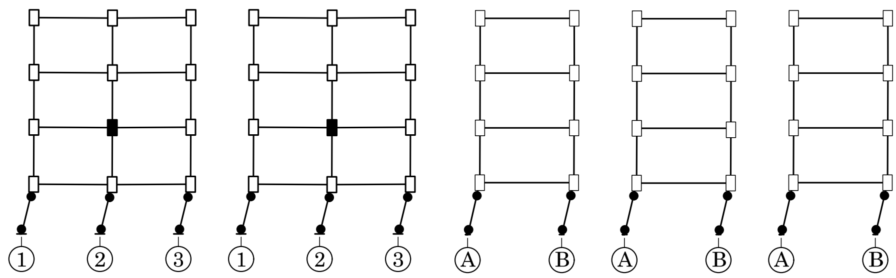


Figure 21 Moment-rotation relation of beam-to-column panel in Y-direction at A-2 row in the second floor (1.0 x Takatori)



A-row B-row 1-row 2-row 3-row

Figure 19 Fainal collapse mechanism (1.0 x Takatori)

7. CONCLUSIONS

This paper has discussed analytical simulation of collapsing behavior of a 4-story steel building which was tested using E-Defense shake table facility. Pseudo 3D analyses have been conducted using Collaborative Structural Analysis (CSA) system. The results are summarized as follows;
1) Analytical results show good correspondence to test results of shake table; in case of 0.6 times Takatori, the specimen was plasticized, and in case of non-scaled Takatori, the specimen collapsed by the soft first story type.
2) In case of 0.6 times Takatori, the plasticized sections are distributed all over the structure in the analysis, thus, the design concept of beam-yield mechanism is successfully achieved.
3) In case of non-scaled Takatori, the damage was localized to the first story, and the bending strength of columns in the first story heavily deteriorated due to local buckling, and finally it lead the specimen to collapse.

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