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COLLAPSE SIMULATION OF U.S. AND JAPANESE TYPE STEEL MOMENT-RESISTING FRAME STRUCTURES USING PRACTICAL MACRO MODELS

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Abstract

Building structures around the world have been designed using various framing methods. In Japan, the two-way moment-resisting frame structures, which is designed as a 3D seismic frame with beams connected to the columns, with moment connections in both directions, is traditionally constructed. In contrast, in the United States and many other countries in high seismic regions, the one-way moment-resisting frame structure, which is designed as separate seismic and gravity frame structure with only a few expensive moment connections in seismic frames, is typically constructed. Structures with these different framing systems are likely to exhibit different seismic response and collapse mechanism when subjected to large earthquake ground motions. However, due to the limitations of analysis program function and so on, the simulation up to complete collapse has almost not been conducted and safety margin to complete collapse of these different framing systems have not been sufficiently understood. In this study, precise seismic simulation up to complete collapse is attempted with general-purpose finite element analysis program to evaluate quantitatively seismic reliability of Japanese and U.S. type steel moment-resisting frame structures.

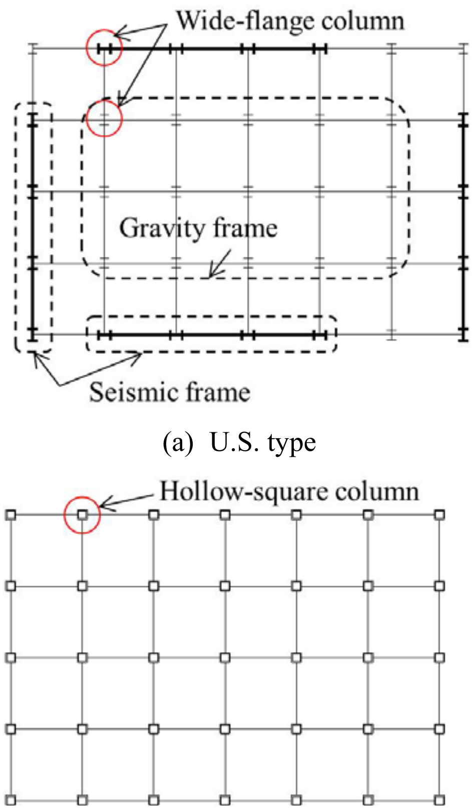
Practical macro models used for the simulation are based on structural elements such as beam and shell elements. In the modeling, steel columns and girders are modeled by beam element and concrete slabs are modeled by shell element. In order to consider the composite effects of concrete slabs on the increase in stiffness and strength, girders are placed under concrete slabs and the multiple-point constraint (MPC) conditions are utilized to connect nodes of girders and slabs assuming the plane remaining after deformation. Also, in order to consider local buckling of steel member, the regions where local buckling may occur are modeled by shell element. Hughes-Liu beam element with cross section integration and Belytschko-Lin-Tsay shell element are utilized for beam and shell elements, respectively. The combined isotropic and kinematic hardening model is utilized for steel constitutive law. Geometric nonlinearity is computed using the update Lagrangian method. Modeling approach is examined by conducting analyses on 1) cantilever column, 2) beam-column and beam-column-slab subassemblies and 3) a 4-story full-scale steel moment-frame structure tested at the world-largest shaking-table facility, E-Defense, in Japan.

In the simulation, two models of U.S. and Japanese type 3-story steel moment frame structures are placed on the virtual shaking-table and subjected to the same level of earthquake ground motion. The U.S. type steel moment frame structure analyzed is the one for the SAC steel project in the United States and Japanese type is the one designed by the BRI in Japan to compare seismic behavior of typical steel moment-frame structures in both countries following the 1994 Northridge and 1995 Kobe earthquakes. As the intensity of ground motion acceleration increases, U.S. type steel frame structure has larger story drift angle than the Japanese type. This is likely due to less redundancy resulting from fewer larger bays in the U.S. type structure. In future study, based on statistical data on the demand and capacity of the structures, seismic reliability of Japanese and U.S. type 3-story steel moment-resisting frame structures will be evaluated quantitatively using probabilistic approach.

Keywords: Steel moment-resisting frame structure; Seismic reliability; Collapse analysis with beam and shell

1. Introduction

Typical steel moment-resisting frame structure in Japan is designed so that almost all frames resist vertical and horizontal loading simultaneously connecting all column to beams with rigid connections as shown in Fig. 1(a). Since columns are subjected to biaxial bending, hollow-square section members are often used for columns. In contrast, typical steel moment-resisting frame structure in the United States and many other countries in seismic regions consists of seismic and gravity frames as shown in Fig. 1(b). Here, in seismic frames, beams are connected to columns with rigid connections, and in gravity frames, beams are connected to columns with bolts at the web, often modeled by pin connection in practical design.



(b) Japanese type

Fig. 1 – Steel moment-resisting frame structures (in plan)

MacRae and Mattheis [1], MacRae and Tagawa [2] conducted 3D frame analysis for U.S. and Japanese type steel frame structures. These studies suggested that these different framing systems may exhibit different collapse mechanism. Particularly, Japanese type steel structure may exhibit soft-story mechanism due to biaxial yielding of columns when subjected to severe earthquake [2]. Hasegawa et al. [3] and Kimura [4] conducted 2D frame analysis for Japanese and U.S. type steel frame structures. Tagawa, MacRae and Lowes [5] conducted reliability analysis on U.S. and Japanese type 3-story steel moment-frame structures utilizing simplified models of these structures, which were preferable for conducting reliability evaluation based on many numerical analyses. However, the models used for these previous analyses were relatively simple, utilizing fiber beam-column element and rotational spring to consider plastic hinge. Moreover, floor slab is not modeled. Also, since conventional structural analysis programs, which take the geometric nonlinearity, often referred to as P-Δ effect in structural engineering, assuming small deformation into account, were used, complete collapse behavior was not simulated.

Recently, due to significant advancement on computational capacity and efficiency, detailed analyses modeling almost all parts of a structure with solid elements to simulate damage and collapse behavior more accurately are conducted [6, 7, 8]. However, computational cost is still expensive modeling with solid elements and may not be suitable for reliability analysis based on many simulations at this moment. In this study, first, macro modeling approach using beam and shell elements to simulate damage and collapse behavior is evaluated for 1) cantilever column, 2) beam-column and beam-column-slab subassemblies and 3) four-story steel moment-resisting frame structure. Next, seismic simulation is conducted for three-story Japanese and U.S. type steel moment-resisting frame structures using general-purpose finite element analysis program. Finally, a method of the evaluation on seismic reliability of U.S. and Japanese type moment-resisting frame structures based on probabilistic approach in future study is described.

2. Modeling Approach using Beam and Shell Elements

In this study, a practical model utilizing beam and shell elements is used for collapse simulation. Shell elements are intended to consider 1) composite effect of beam and shell and 2) local buckling of steel member. General-purpose finite element analysis programs, NX-Nastran [9] and LS-DYNA [10], are used in this study. Modeling of the structures and eigenvalue analyses to check the modeling are conducted using NX-Nastran [9]. Nonlinear quasi-static analyses and dynamic time-history analyses are conducted using LS-DYNA [10] with implicit solver option. Geometric nonlinearity is computed using the updated Lagrangian method [10].

2.1 Beam and Shell Elements

The Hughes-Liu beam element with cross section integration model is used for beam element. The Hughes-Liu beam element is based on a degeneration of the isotropic 8-node solid element as shown in Fig. 2 [10, 11, 12]. The section divisions of the Hughes-Liu beam element for box-shape and I-shape are shown in Fig. 3. Here, the division number of *k* can be determined by a user. Belytschko-Lin-Tsay element [13], which is based on a combined co-rotational and velocity-strain formulation [10], is used for floor slab in this study.

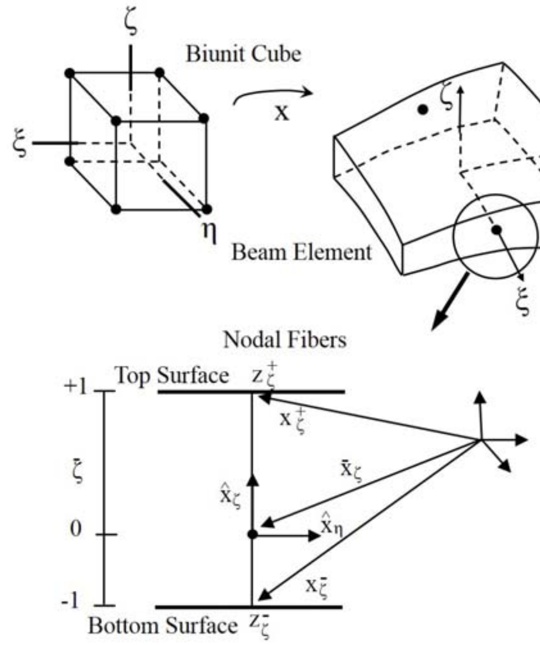
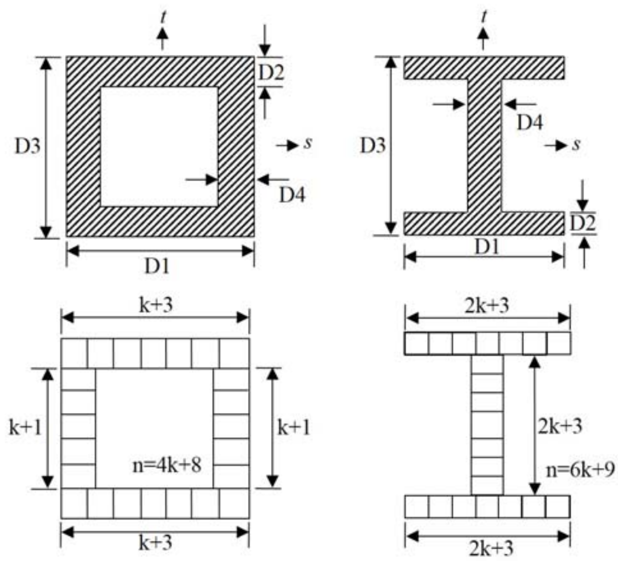


Fig. 2 – Hughes-Liu beam element [10]



(a) Box-shape (b) I-shape

Fig. 3 – Section division

2.2 Modeling of Composite Effect of Floor Slab

Axis line of a girder is located under the centerline of floor slab in actual steel moment-frame structures as shown in Fig. 4 and then strength and stiffness increase due to the presence of a slab. In this study, beam element for a girder is placed under shell element for floor slab and the multiple-point constraint (MPC) condition is applied to connect the nodes of the girder and slab as shown in Fig. 5. The MPC condition keeps the distance between two nodes always constant assuming the plane remaining after deformation. This modeling approach is intended to consider the composite effects by floor slab such as increase of stiffness and strength explicitly.

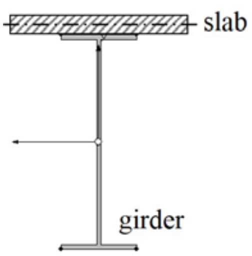


Fig. 4 – Girder under slab

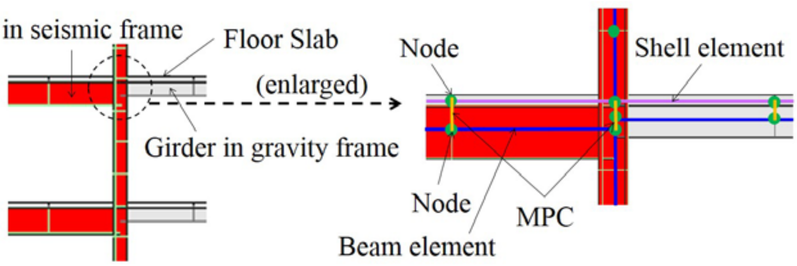
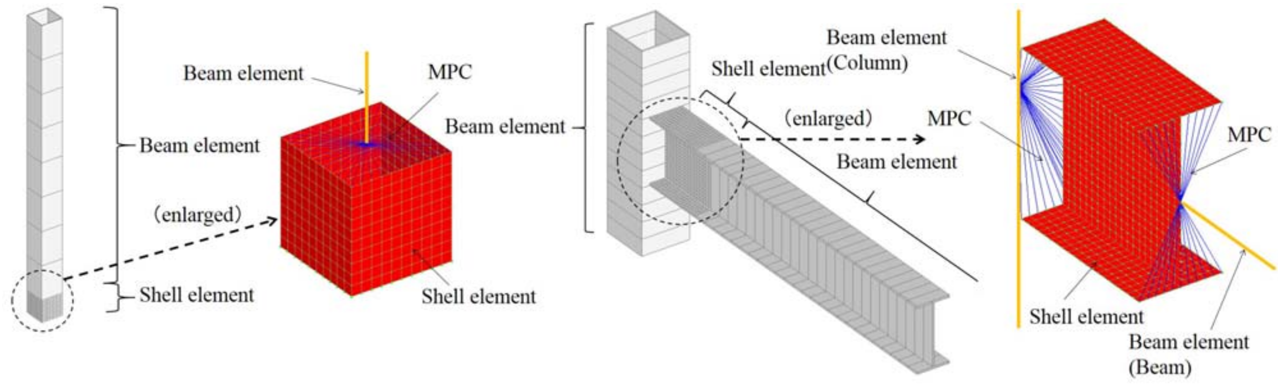


Fig. 5 – Modeling with shell and MPC to consider composite effect

2.3 Modeling of Local Buckling

Beam element cannot simulate the local buckling of steel member, which is out-of-plane behavior, accurately. To consider the local buckling, the regions where local buckling may to occur are modeled by shell element as shown in Fig. 6. Here, shell element installed at the column end is shown in Fig. 6(a) and shell element installed at the beam end is shown in Fig. 6(b). Between the nodes of shell element at the edge are connected to the end node of beam element with the multiple-point constraint (MPC) condition assuming the plane remaining after the deformation.



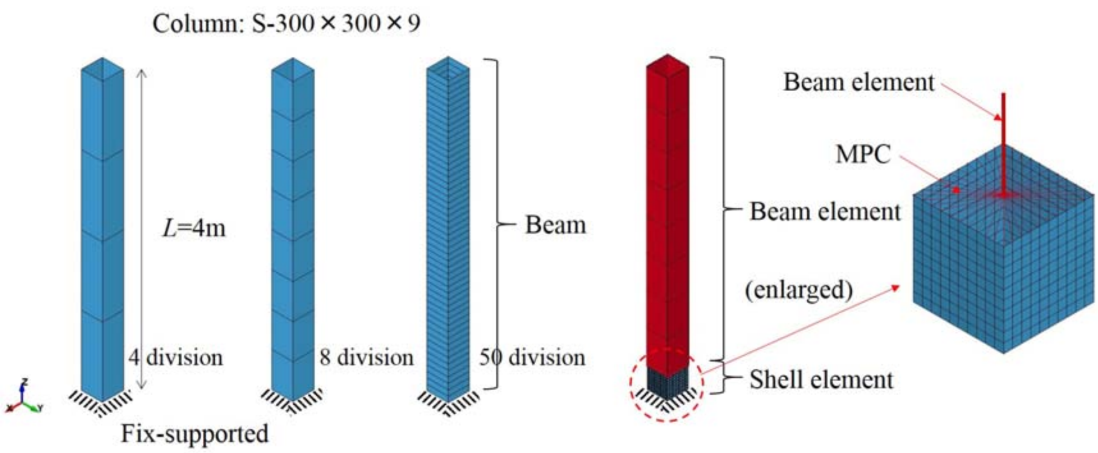
(a) Shell element at column end (b) Shell element at beam end

Fig. 6 – Modeling to consider local buckling using Shell elements

3. Verification of Modeling Approach

3.1 Cantilever Column

In order to determine an appropriate number of the divisions along the length and over the section of beam element, models for a cantilever column as shown in Fig. 7 are investigated. All models represent a column with a section of S-300×300×9 (box-shape) and the length, L, is 4 m. Three models shown in Fig. 7(a) consist of beam element only and the division number along the length is 4, 8 and 50. Also, a model shown in Fig. 7(b) consists of beam element and shell element located at the base to consider local buckling. Eigenvalue analyses are conducted for the models with various division numbers of beam element. The section division number, k, as shown in Fig. 3(a) for box-shape is varied from 0 to 2, 4, 6, 8 and 10. The values of the 1st and 2nd natural frequencies calculated by LS-DYNA are listed in Table 1. When the division number k is small, the 1st and 2nd natural frequencies are different, which results from different values of the moment of inertia due to different divisions over the section in X- and Y- directions. As the division number k increases, the 1st and 2nd natural frequencies converge.



(a) Models with beam element only (b) Model with beam and shell elements

Fig. 7 – Models for cantilever column

Table 1 – Natural fundamental frequencies (Hz)

		Division along length					
		4		8		50	
		1st	2nd	1st	2nd	1st	2nd
Section division k	0	18.29	20.47	18.52	20.72	18.59	20.81
	2	20.52	20.67	20.78	20.93	20.85	21.01
	4	20.69	20.72	20.95	20.98	21.02	21.07
	6	20.73	20.75	20.99	21.01	21.08	21.09
	8	20.75	20.76	21.01	21.02	21.09	21.09
	10	20.76	20.76	21.02	21.03	21.11	21.12

Theoretical value of the 1st natural frequency is given by Eq. 1.

$$f_{theory} = \frac{\lambda^2}{2\pi L^2} \times \sqrt{\frac{EI}{\rho A}} \tag{1}$$

Here, L is a column length, E is the elastic modulus, I is the moment of inertia, ρ is a density, A is an area, and λ is a constant value defined by boundary condition and 1.875 for a cantilever column. The theoretical value of the 1st natural frequency is 21.24 Hz. When the division number along the length is 8 and the section division number k is 10, the 1st and 2nd natural frequencies are 21.02 and 20.03 Hz, respectively, which are close to theoretical value above. Therefore, it is determined that when the model with beam element only is used for analyses, division number along the length is 8 and division number of section k is 10. The 1st and 2nd natural frequencies of the model with beam and shell elements as shown in Fig. 7(b) are 21.01 Hz, which is similar to that of the model with beam element only.

In order to evaluate the effects of local buckling, pushover analyses are conducted for a cantilever column model with beam element only as shown in Fig. 7(a) and a model with beam element and shell element at the bottom of a column as shown in Fig. 7(b). The relations of bending moment at the base normalized by the plastic moment, M/M_p, and a displacement at the top of the column normalized by column length, Δ/L, for the two models are compared in Fig. 8. The model with beam element only exhibits gradual increase in strength due to strain hardening after yielding, whereas the model with beam and shell elements exhibits strength degradation after yielding. This is due to local buckling of a column as shown in Fig. 9. These results suggest that, when local buckling occurs, modeling with beam element only is not appropriate and modeling with shell elements for the regions where local buckling may occur is necessary to simulate the damage and collapse behavior accurately.

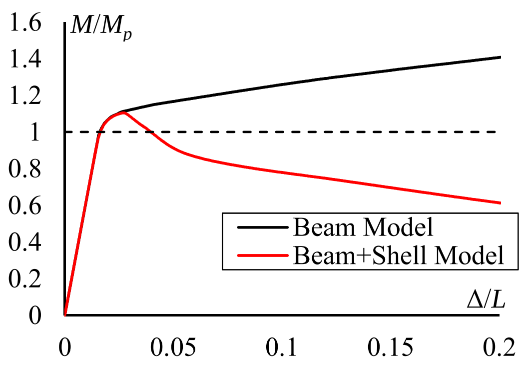


Fig. 8 – Pushover analysis results

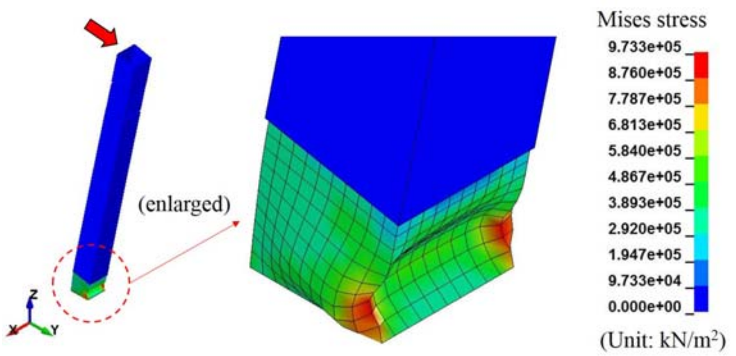
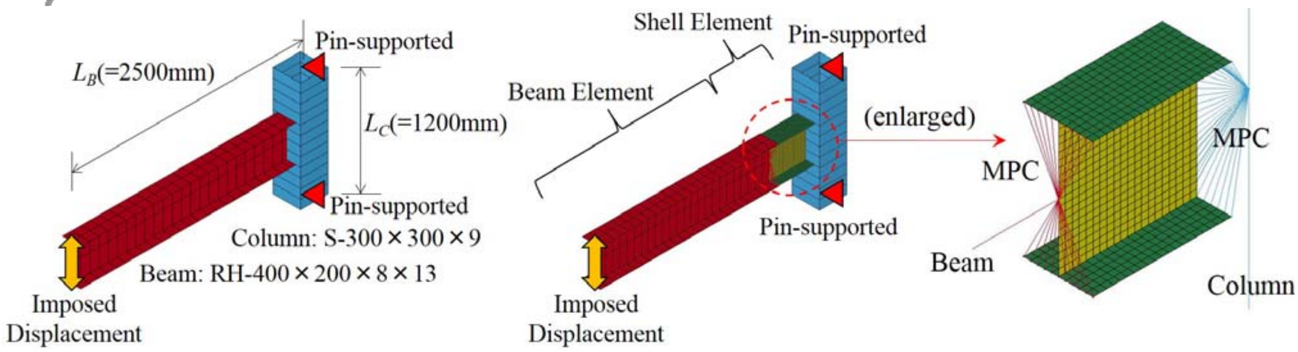


Fig. 9 – Local buckling observed in model with shell element

3.2 Beam-Column and Beam-Column-Slab Subassemblies

Models of a beam-column assembly as shown in Fig. 10 are analyzed for quasi-static loading. Imposed displacement is applied to the tip of a beam and both ends of a column are supported by pins.



(a) Model with beam element only (b) Model with beam and shell elements

Fig. 10 – Models for beam-column subassembly

The relation of bending moment at the end of a beam normalized by the plastic moment of a beam, M/M_p, and vertical displacement at the tip of a beam normalized by beam span, Δ/L, for a model with beam element only is shown in Fig. 11. Here, initial stiffness values obtained theoretically as follows are also shown as a reference. Vertical displacements at the tip of a beam when subjected to vertical force, P, due to beam and column flexure, and beam and column shear are calculated, respectively, as follows.

$$\delta_B^{flexure} = \frac{PL_B^3}{3EI_B} \qquad \delta_C^{flexure} = \frac{PL_B^2 L_C}{12EI_C} \qquad \delta_B^{shear} = \frac{PL_B}{GA_B^{shear}} \qquad \delta_C^{shear} = \frac{P}{GA_C^{shear}}\left(\frac{L_B^2}{L_C}\right) \tag{2a–2d}$$

Here, E is the elastic modulus, G is the shear modulus, L_B is beam span based on the centerline, L_C is column height between the top and bottom pin supports, I_B and I_C are the moment of inertia of beam and column, respectively, A_B^{shear} and A_C^{shear} are shear area of beam and column, respectively. Initial stiffness in the relation of vertical load, P, and vertical displacement at the tip of a beam, Δ, considering 1) only flexural deformation of beam, 2) flexural deformations of beam and column and 3) flexural and shear deformations of both beam and column are given as follows. As shown in Fig. 11, initial stiffness obtained by the analysis is almost identical to the value corresponding to K_3.

$$K_1 = \frac{P}{\delta_B^{flexure}}$$

$$K_2 = \frac{P}{\left(\delta_B^{flexure} + \delta_C^{flexure}\right)} \tag{3a–3c}$$

$$K_3 = \frac{P}{\left(\delta_B^{flexure} + \delta_C^{flexure} + \delta_B^{shear} + \delta_C^{shear}\right)}$$

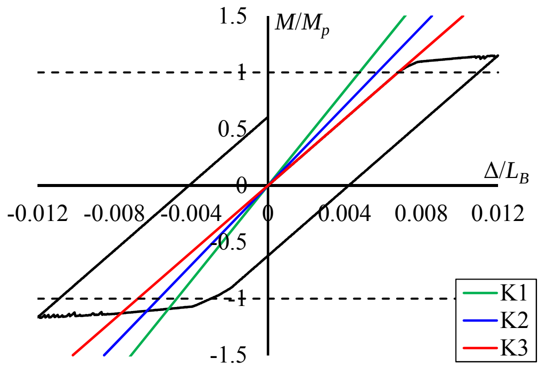


Fig. 11 – Push-pull analysis results for model with beam element only

In order to evaluate the local buckling effect at the end of a beam, quasi-static pushover analyses for the beam-column subassembly model with beam element only and the beam-column model with beam element and shell element located at the beam end. The relations between force and displacement for these models are compared in Fig. 12. The model with beam and shell elements exhibits strength degradation after peak strength due to local buckling as shown in Fig. 13. Initial stiffness of the model with beam and shell elements is larger than that of the model with beam model only since the former model considers the rigid zone at the beam-column joint with the MPC as shown in Fig. 10(b). In Fig. 12, the K_3 line assuming L_B=2.35m (=2.5m-0.3m/2) is plotted, which corresponds well to the model with beam and shell elements. These results suggest that the model with beam elements located at centerlines cannot consider the rigid zone at the beam-column joint.

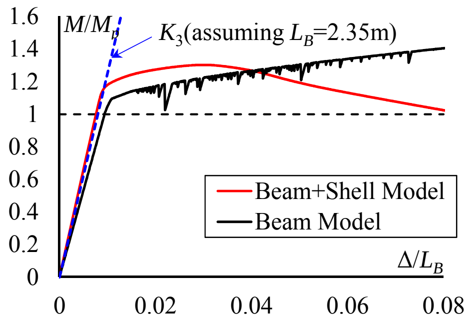


Fig. 12 – Pushover results for two models

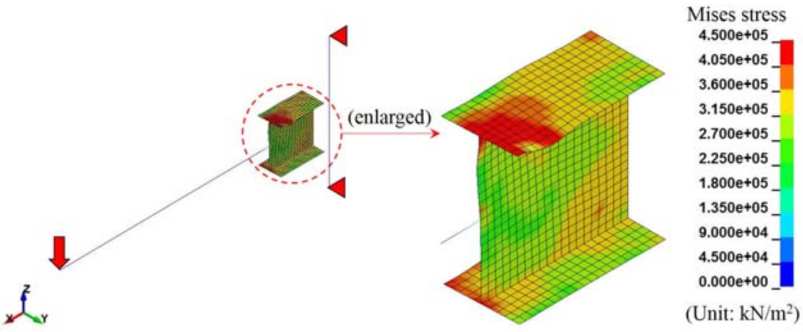


Fig. 13 – Local buckling observed in model with shell element

In order to evaluate the composite effect of steel beam and concrete slab, quasi-static pushover analysis is conducted for beam-column-slab subassembly as shown in Fig. 14. The relations of bending moment at the beam end and vertical displacement at the tip of the beam are compared for beam-column and beam-column-slab subassemblies. Beam-column-slab subassembly has larger initial stiffness and yield strength than beam-column subassembly as shown in Fig. 15. Here, constitutive law for concrete material is assumed to be elastic. Due to the presence of concrete slab, the increases in strength and stiffness are apparent. The degrees of the increase in positive and negative direction are similar due to the assumption of elastic material for concrete, which is not true in actual structure. Concrete damage should be included in the constitutive law in future study.

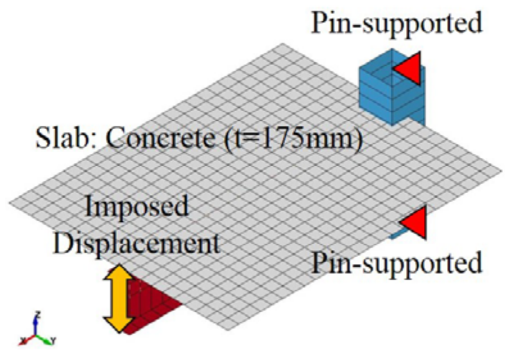


Fig. 14 – Beam-column-slab subassembly model

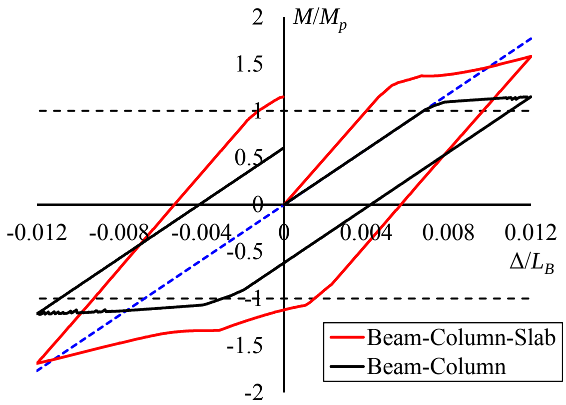
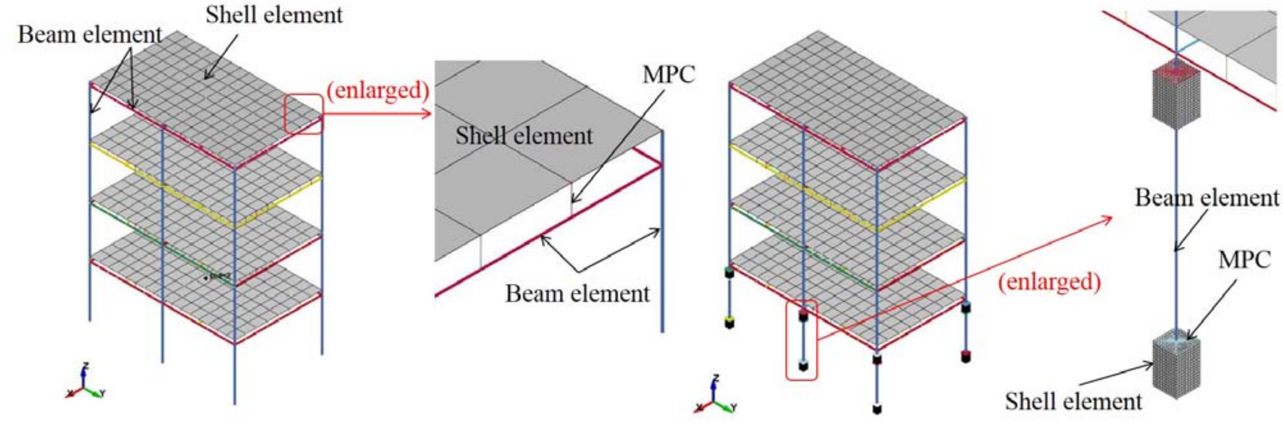


Fig. 15 – Push-pull analysis results

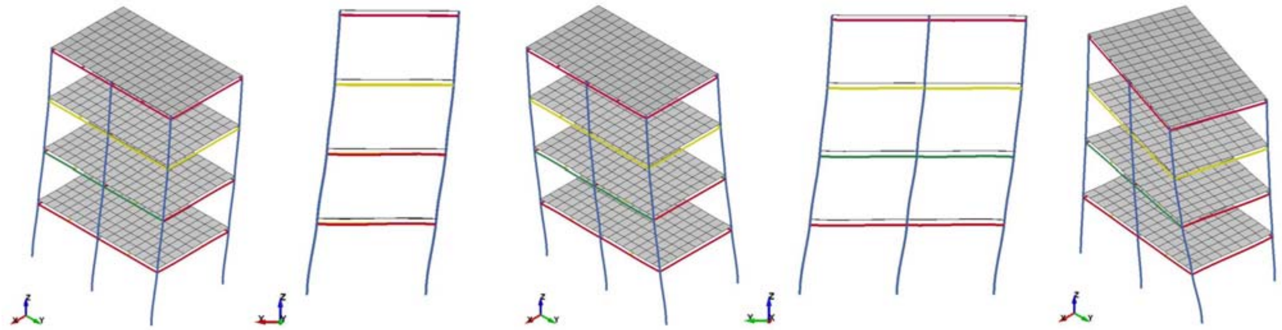
3.3 Four-story Steel Moment-resisting Frame Structure

Models for a four-story steel moment-resisting frame structure, which was tested at the world-largest shaking-table facility, E-Defense, [14, 15], are analyzed for the JR-Takatori records in the 1995 Kobe Earthquake. Two types of models are investigated; the first one is the model with beam element and shell element for floor slab only as shown in Fig. 16(a) and the second one is the model with beam element and shell element for floor slab and the top and bottom ends of the 1st-story columns [16] as shown in Fig. 16(b). Concrete density is increased from 2.4 to 4.74 so that the weight of the entire structure of the model is similar to that of the specimen. Ground accelerations are imposed to the nodes at the base of all 1st-story columns as the translational acceleration and rotational degrees-of-freedom are constrained at these nodes. The 1st, 2nd and 3rd-mode shapes are shown in Fig. 17, which are translational deformations in X-, Y-directions and torsional deformation. The 1st and 2nd natural periods obtained by the models are 0.81 sec and 0.79 sec, which are similar to those of the specimen obtained by the experiment, 0.80 sec and 0.76 sec [14]. Rayleigh damping is applied to the entire model so that damping ratio at the 1st natural period and 0.2 sec is 0.02. Deformations at t=7.5 sec and t=8.3 sec are shown in Fig. 18. Complete collapse resulted by local buckling of the 1st-story columns, which were observed in the shaking-table test [15], is simulated successfully using the model with shell element located at the top and bottom of the 1st-story columns. The 1st-story drift angle history for two models is compared in Fig. 19. The model with shell elements at the top and bottom of the 1st-story columns exhibits significant increase in the 1st-story drift in Y-direction after around t=6.5 sec due to local buckling, resulting in complete collapse. However, the model with beam element and shell element for slab does not collapse.



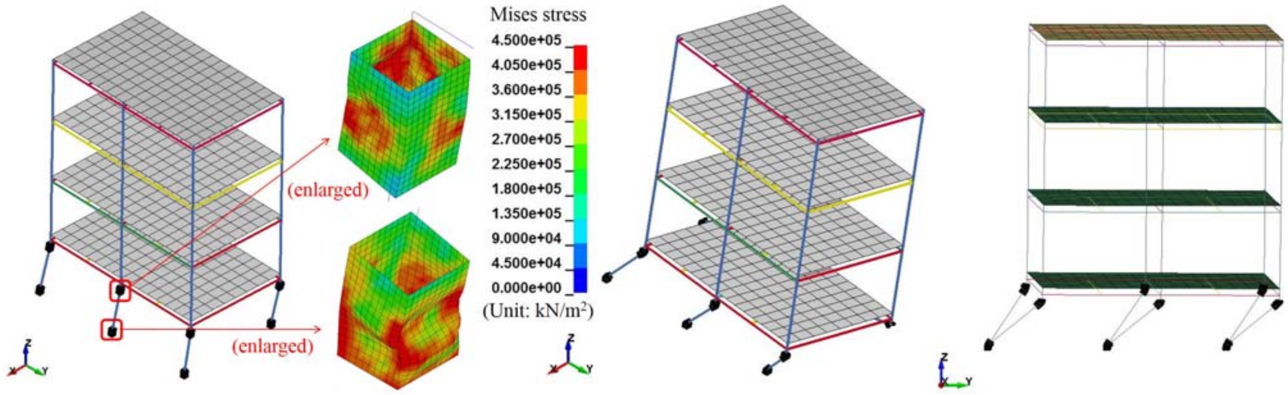
(a) Model with beam and shell (slab) elements (b) Model with beam and shell (slab and column end) elements

Fig. 16– Model for 4-story steel moment-resisting frame structure



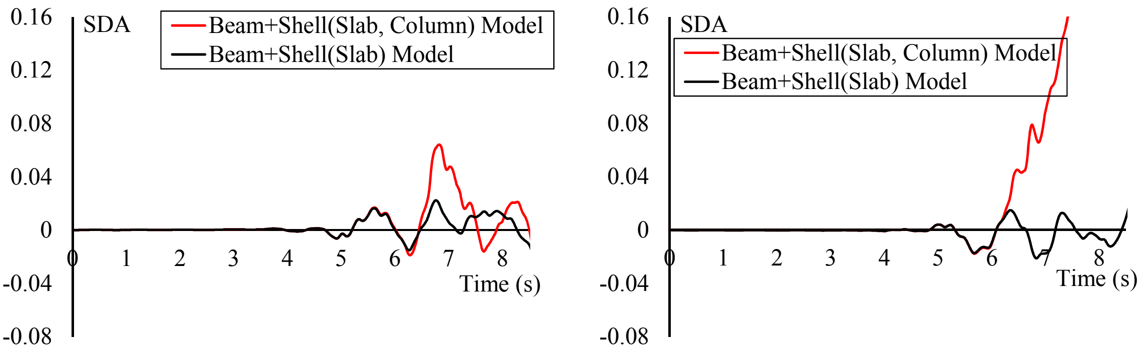
(a) 1st-mode shape (X-translational) (b) 2nd-mode shape (Y-translational) (c) 3rd-mode shape (torsional)

Fig. 17 – Fundamental mode shapes



(a) Deformation at t=7.5 sec (b) Deformation at t=8.3 sec

Fig. 18 – Complete collapse simulation resulted by local buckling by model with shell element at top and bottom of 1st-story columns



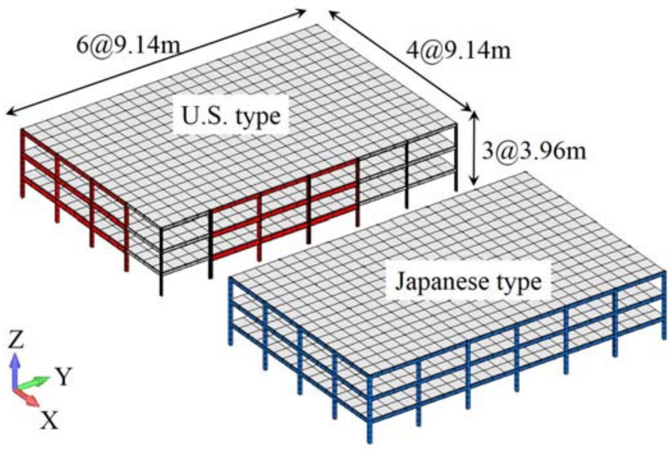
(a) X-direction (b) Y-direction

Fig. 19 – 1st-story drift angle history for two models

4. Seismic Simulation of U.S. and Japanese Type Steel Moment-Frame Structures

4.1 Modeling

Analysis models for Japanese and U.S. type steel moment-frame structures [17] are shown in Fig. 20. These are set on the same plane, referred to as virtual shaking-table in this study, and subjected to the same level of ground motion in seismic simulation. List of member size is given in Table 2. In Japanese type, hollow-square (box-shape) section columns are used. The elastic modulus is 205 kN/mm^2 is for steel and 11.25 kN/mm^2 for concrete. The density is 7.85 ton/m^3 is for steel and 2.4 ton/m^3 for concrete. The thickness of floor slab is 150 mm. Member and section sizes are referred to 3-story steel moment-frame structure designed in the SAC steel project [18] for the U.S. type and to 3-story steel moment-frame structure designed according to Japanese standards by Hasegawa et al. [3] for Japanese type. Each beam element for columns and girders is divided in 4 and each edge of floor slab is divided in 4 according to the division of adjacent girders. In gravity frames in the U.S. type structure, beams are modeled to connect columns with pins. The Mises yield surface and combined isotropic and kinematic hardening model is applied to steel constitutive law.



[U.S. Type]
(Seismic frame)

	Column		Beam
	exterior	interior	
3/roof	W14x257	W14x311	W24x68
2/3			W30x116
1/2			W33x118

(Gravity frame)
Column: W14x68
Beam: W18x35 (except roof), W16x26(roof)

[Japanese Type]

	Column	Beam
3/roof	□-450x16	H-550x200x9x19
2/3	□-450x19	
1/2	□-450x22	H-550x200x12x22

Fig. 20 – Analysis models on virtual shaking table and member list

4.2 Eigenvalue analysis

Eigenvalue analysis is conducted for Japanese and U.S. type steel moment-frame structures using NX-Nastran [9]. For reference, the analysis is conducted for the models in which beam element for a girder is placed at the same level to shell elements for floor slab. The 1st, 2nd and 3rd natural periods are shown in Table 3. The 1st-mode shape for the U.S. type is shown in Fig. 21. This is a translational mode. The 3rd-mode shape is shown in Fig. 22. This is a torsional mode. As shown in Table 3, for the U.S. type structure, the 1st natural period is shortened from

0.769 sec to 0.678 sec when shifting beam element for a girder under shell element for floor slab in modeling. This means that the 1st mode stiffness increases by $(0.769/0.678)^2=1.286$ times due to the composite effect by floor slab, considering both models have the same amount of mass. Also, the 1st and 2nd natural periods, which correspond to translational mode, of the U.S. type are larger than those of Japanese type. This is likely due to a smaller number of seismic frames in the U.S. type structure. However, the 3rd natural period, which corresponds to torsional mode, is smaller than that of Japanese type. This is likely due to the location of stiff seismic frames, at the perimeters of the structure, which is most effective for torsion.

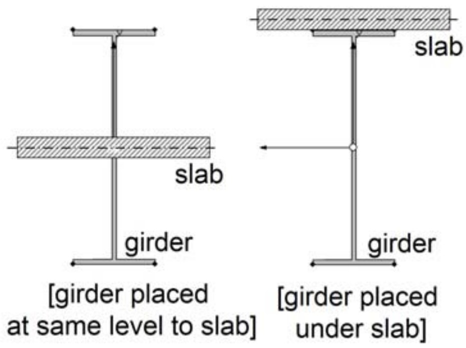
Table 3 – Natural fundamental periods

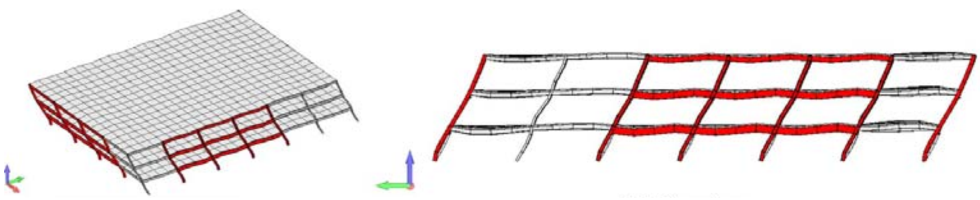
(a) U.S. type

	girder placed at same level to slab	girder placed under slab
1st mode	0.769 sec	0.678 sec
2nd mode	0.748 sec	0.654 sec
3rd mode	0.496 sec	0.443 sec

(b) Japanese type

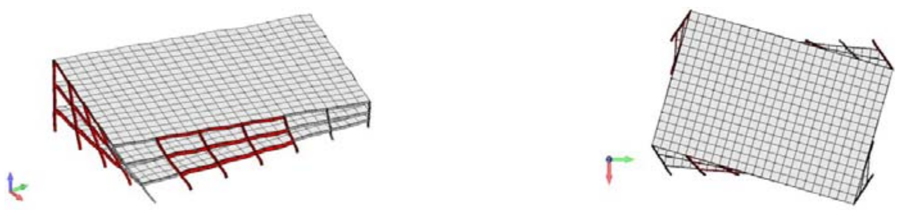
	girder placed at same level to slab	girder placed under slab
1st mode	0.621 sec	0.524 sec
2nd mode	0.608 sec	0.516 sec
3rd mode	0.533 sec	0.457 sec





(a) Bird's eye view (b) Elevation

Fig. 21 – 1st-mode shape (translational)

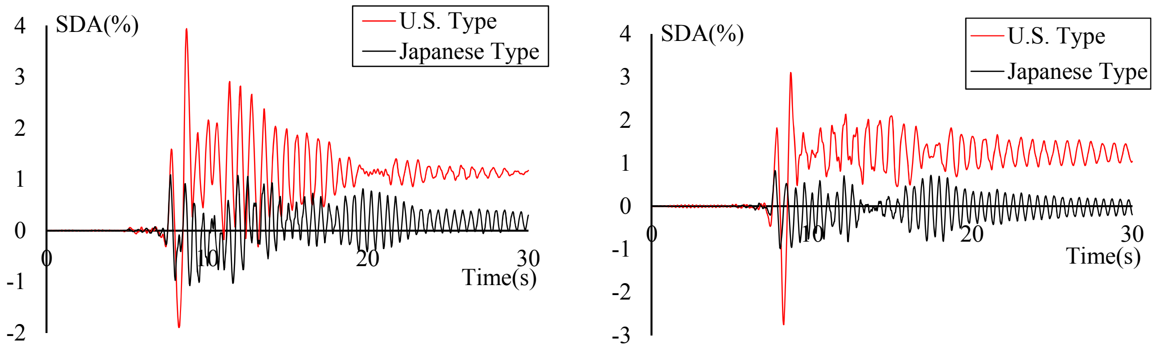


(a) Bird's eye view (b) Plan

Fig. 22 – 3rd-mode shape (torsional)

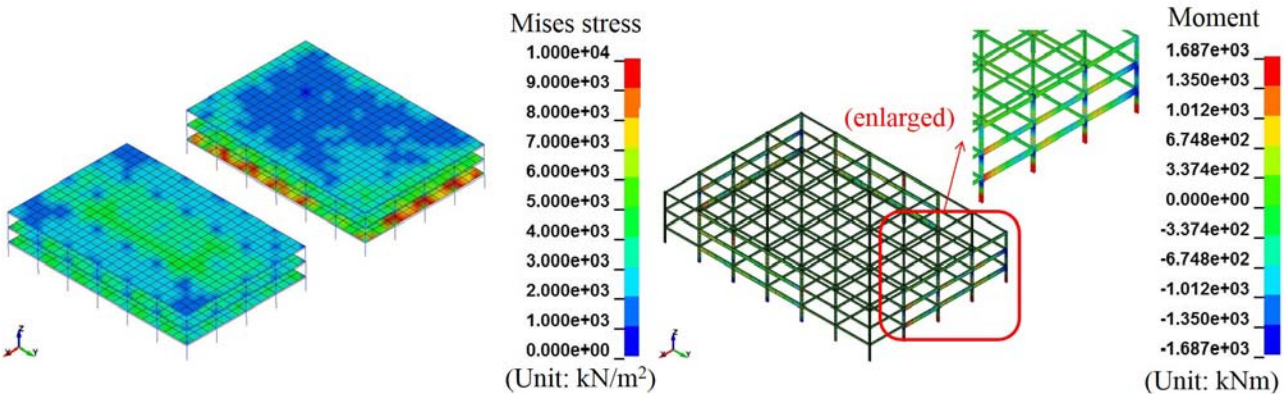
4.3 Seismic Simulation

Seismic simulation is conducted for the U.S. and Japanese type steel moment-frame structures. The ground motion input is the JMA-Kobe records in the 1995 Kobe Earthquake. The 1st-story drift angles (SDA) of U.S. and Japanese type steel moment-frame structures are compared in Fig. 23. The maximum of the 1st SDA of the U.S. type structure is 3.94% in X-direction whereas the 1st SDA of the Japanese type is 1.09%. Also, the U.S. type has large residual displacement more than 1% SDA. The deformation plot with the counter of the Mises equivalent stress at the occurrence of the maximum 1st-SDA is plotted in Fig. 24(a). Bending moments of columns and beams in the U.S. type structure at that time are plotted in Fig. 24(b). The yield moment of the 1st-story beam (W33×118) is 1687 kN· m. It is found that yielding occurs at many column and beam ends. The response of two types of framing systems needs to be investigated for many ground motion accelerations in future study.



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

Fig. 23 – 1st-story drift angle response for 1995 Kobe JMA-Kobe records



(a) Mises equivalent stress on floor slab (b) Bending moment of column and beam in U.S. type

Fig. 24 – Results of seismic simulation for 1995 Kobe JMA-Kobe records

4.4 Evaluation on Seismic Reliability in Future Study

In future study, reliability analysis will be conducted for the U.S. and Japanese type steel moment-resisting frame structures based on probabilistic approach represented by the following equation (e.g. [19]).

$$v(DV) = \iiint G < DV \mid DM > dG < DM \mid EDP > dG < EDP \mid IM > d\lambda(IM) \tag{5}$$

1) The regression analysis is conducted for analysis results of response spectra of many ground accelerations. The function of the annual exceedance, $\lambda(IM)=\lambda(S_a)$, is computed.

2) The Incremental Dynamic Analysis is conducted for many ground motions. The probability of exceedance $G<EDP|S_a>$ of the Engineering Demand Parameter (EDP) for the ground motion intensity S_a is computed. The intensity of ground motion is increased to the level of complete collapse. Collapse mechanism will be identified.

3) Using the probability of the annual exceedance of the ground motion intensity and the IDA results, the probability of the annual exceedance for various EDPs will be computed using Eq. 6.

$$v(EDP) = \int G < EDP \mid IM > d\lambda(IM) = \int G < SDA \mid S_a > d\lambda(S_a) \approx \sum G < SDA \mid S_a > \cdot \lambda(S_a) \cdot \Delta S_a \tag{6}$$

4) Using the probabilistic data on the capacity of members, the annual probability of failure will be computed by Eq. 7.

$$APE(fail) = \sum \left\{ P[S_a - \frac{\Delta}{2} < X_i < S_a + \frac{\Delta}{2}] \cdot p_f(S_a) \right\} \cong \sum \left\{ \frac{dH(S_a)}{dS_a} \cdot \Delta] \cdot \int_0^{\infty} \left[1 - F_s(r)|_{S_a} \right] f_R(r) dr \right\} \tag{7}$$

5. Conclusions

Practical macro modeling using beam element and shell element to consider composite effect and local buckling is investigated and used for seismic simulation of 3-story U.S. and Japanese type steel moment-frame structures. The U.S. type analyzed exhibits larger 1st-story drift angle and also residual displacement than the Japanese type for ground acceleration investigated.

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