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FULL-SCALE TEST OF THREE-STORY STEEL MOMENT FRAMES FOR EXAMINATION OF EXTREMELY LARGE DEFORMATION AND COLLAPSE BEFAVIOR

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SUMMARY

This paper presents an overview of the full-scale test on a three-story, two-span by one-span steel moment frame. The test was conducted to characterize the cyclic behavior of steel moment frames beyond the deformation ranges considered in the contemporary seismic design. Stable behavior was observed up to an overall drift angle of 1/25. Pinching behavior was notable for cyclic loading with larger amplitudes primarily because of cyclic yielding and resulting slip-type hysteresis experienced at the column bases. Pushover analyses using numerical analysis codes commonly adopted in seismic design practices are very reasonable to predict the elastic stiffness and the strength. Adding strain hardening after yielding and composite action between the steel beams and RC floor slabs, numerical analyses are able to duplicate the experimental cyclic behavior very accurately. The generic frame model is also very accurate and effective in seismic design.

INTRODUCTION

Toward the advancement of "performance-based seismic design," *real data* about the performance, damage and collapse of structures under seismic action are indispensable. They are rather difficult to acquire, however, because of scarcity of larger earthquake events (difficulties associated with observations) and massiveness of our civil and building structures (difficulties associated with laboratory tests). Fortunately, the writers had an opportunity to run an experimental project in which a full-scale, three-story steel building frame was loaded quasi-statically to failure. The primary objectives of the project were: (1) to acquire realistic data about performance and progress of damage of the concerned frame in deformation ranges that are beyond those considered in contemporary seismic design; (2) to

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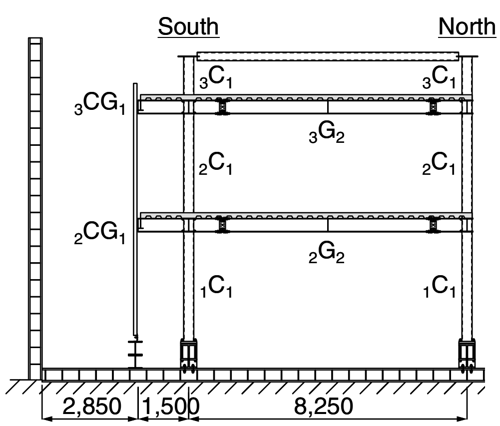
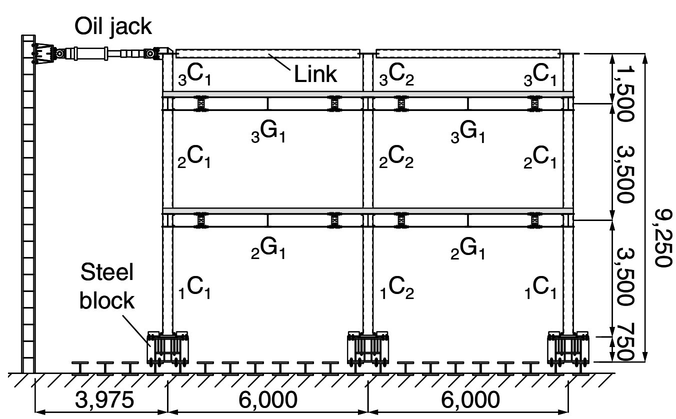
examine the interaction between the local damage induced into individual members and elements and the global damage sustained by the structural frame; (3) to observe effects of RC floor slabs on the behavior of steel moment frames; (4) to examine the interaction between the structural system and exterior finishes; and (5) to acquire data about the final collapse behavior of the structural frame. This paper reports on an overview of the test program and representative results obtained. The paper also examine how the numerical analyses commonly adopted in contemporary seismic design can trace the experimental behavior.

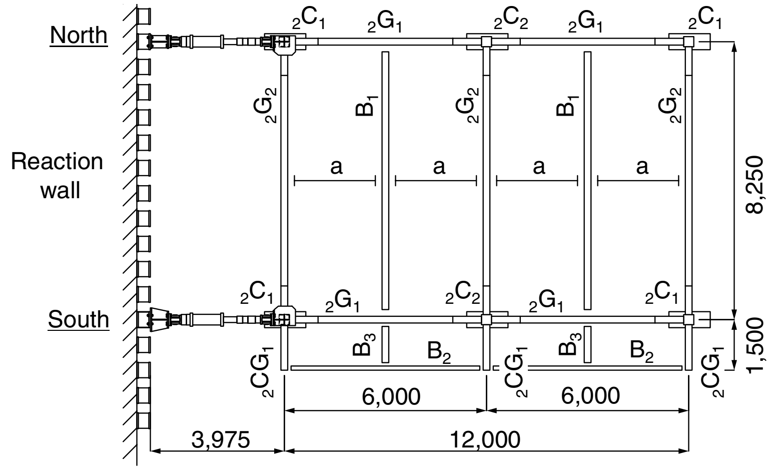
TEST STRUCTURE

The test structure was a three-story, two-bay by one-bay steel moment frame as shown in Figure 1, having a plan dimension of 12 m (in the longitudinal direction) by 8.25 m (in the transverse direction). The structure was designed following the most common design considerations exercised in Japan for post-Kobe steel moment frames. That is, the columns were made of cold-formed square-tubes, beams were made of hot-rolled wide-flanges, the through-diaphragm connection details were adopted, in which short brackets were shop-welded to the columns [Figure 2(a)]. The columns with short brackets were transported to the test site, and they were connected horizontally to beams by high-strength bolts. Metal deck sheets were placed on top of beams, with studs welded to the beam top flanges through the metal deck sheets. Wire-meshes were placed above the metal deck sheets, and concrete was placed on site. In the design of the test structure, yielding and plastic deformations were assigned for beam-ends, panel-zones, and column bases; hence the column-to-beam strength ratios ranged from 1.9 to 2.2. Fabrication and construction procedures adopted for the test structure faithfully followed those exercised in real practice [1]. Exception was the column bases. Instead of embedding anchor bolts in the foundation RC beams, anchor bolts were fastened in short, deep steel beams, which in turn were securely tied down to the strong floor [Fig.2(b)].

The columns were extended to the approximate mid-height in the third story, at which level steel braces were connected horizontally to the columns by high strength bolts through gusset plates. The braces served to achieve a rigid-diaphragm action in this plane, while the column rotations at the top were permitted by the out-of-plane flexibility of the gusset plates. Two quasi-static jacks, one in each longitudinal plane, were placed in this level, as shown in Fig.1. Exterior finishes (cladding) were installed during the test to explore the performance and damage of nonstructural elements and the interaction between these elements and structural frame. This part of the program is beyond the scope of this paper, and details on the performance of nonstructural elements can be found elsewhere [2, 3]

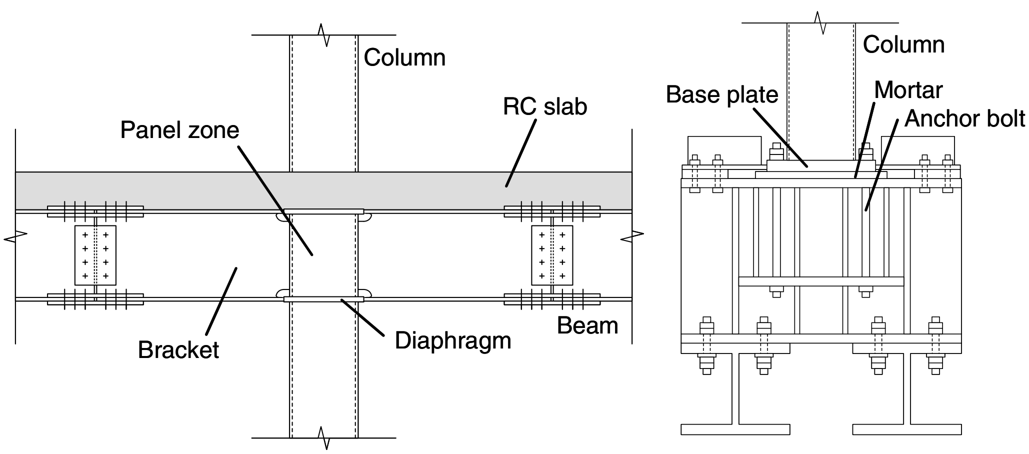






Member		Section	Member		Section (Stud bolt)
Column	$_1C_1$	□ - 300×9	Beam	$_2G_1$	H - 400×200×9×16 (19ϕ h=110 Single@200)
	$_1C_2$	□ - 300×12		$_2G_2$	H - 400×200×9×16 (19ϕ h=110 Single@200)
	$_2C_1$	□ - 300×9		$_3G_1$	H - 400×200×9×16 (19ϕ h=110 Single@200)
	$_2C_2$	□ - 300×9		$_3G_2$	H - 400×200×9×16 (19ϕ h=110 Single@200)
	$_3C_1$	□ - 300×12		$_2CG_1$	H - 400×200×8×13 (19ϕ h=110 Single@200)
	$_3C_2$	□ - 300×16		$_3CG_1$	H - 400×200×8×13 (19ϕ h=110 Single@200)
Anchor bolt	M33	length=740 (110)		B_1	H - 400×200×8×13 (16ϕ h=110 Single@300)
	M36			B_2	H - 300×150×6.5×9 (No)
Base plate		50×475×475		B_3	H - 200×100×5.5×8 (16ϕ h=110 Single@300)
Mesh		6ϕ 150×150		a	H - 200×100×5.5×8 (No)

Fig. 1 Plan and elevation of test structure (unit: mm)



(a) **(b)**

Fig. 2 Connection details: (a) beam-to-column connection; (b) column base connection

LOADING AND MEASUREMENT

As shown in Fig.1, two quasi-static jacks were arranged for horizontal loading. Each jack was placed at one end of the test structure and at the mid-height of the third story. An identical displacement was applied to both jacks. The two planes, taking the same displacement at the top, acted nearly independently; that is, no transfer of the force between the two planes was observed. This means that the load applied to each jack was the same as the force sustained by the concerned plane. Figure 3 shows the loading program used in the test. Quasi-static cyclic loading with increasing displacement amplitudes was adopted, and either two or three cycles were repeated for each amplitude. The displacement was expressed in terms of the overall drift angle, defined as the horizontal displacement at the loading point relative to the loading height (i.e., 8.5 m). Overall drift angles of 1/200, 1/100, 1/75, 1/50, 1/25, and 1/20 were adopted. After loading to the 1/20 amplitude, the jacks were dismounted once, and installed again with a 0.6 m long shim, and reloaded again to the maximum overall drift angle of 1/15 to examine the failure behavior. A computer controlled on-line test system was used for the test, the detail of which is found in [4, 5].

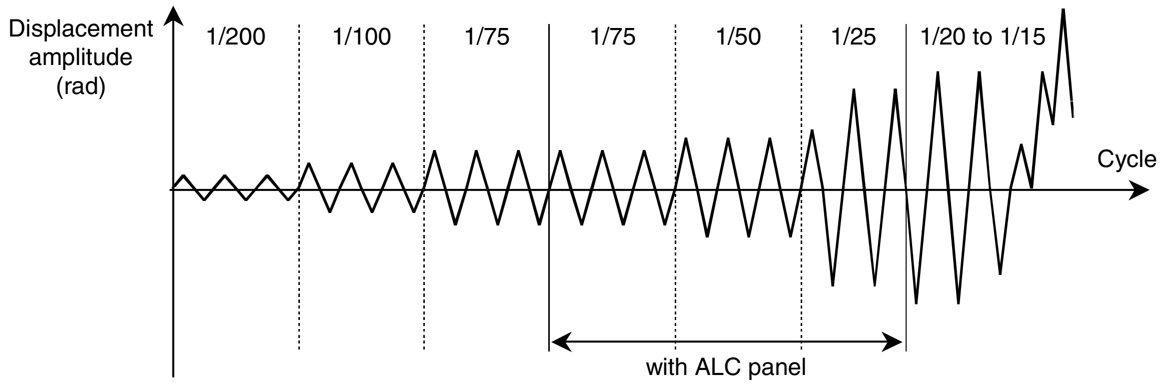


Fig. 3 Loading program

A load cell attached to the head of each jack measured the horizontal load applied by the jack. A digital displacement transducer that had a resolution of 0.01 mm was used to measure the displacement of the jack. Four strain gauges were glued on the column surface at two cross-sections, each located at a distance of 1 m inward either from the column top or bottom. The cross-sections remained elastic; thus the bending moments applied at the cross-sections were estimated from the corresponding curvatures. The shear force applied to the column was estimated as the sum of the two bending moments divided by the distance between the measured cross-sections. According to the measured results, the shear force thus estimated was found very reasonable. The column axial force was estimated from the average of the strains measured by the column strain gauges. The beam shear force was estimated from the difference between the axial forces exerted into the two columns, one located on the top of and the other located underneath the concerned beam. Shear deformations of the panel zones, deformations of the floors in the direction orthogonal to the loading direction, rotations and lateral displacements of the column bases, and out-of-plane rotations and displacements of the beams were also measured by displacement transducers having a variety of gauge lengths. Furthermore, many strain gauges were glued on the beam flanges and webs in the vicinity of beam-to-column connections as well as on the anchor bolts at the column bases. These gauges were used to obtain information on local strains and deformations. Summing all the displacement transducers and strain gauges, a total of 283 data channels were connected to the data logger, which in turn was connected on-line to PC for Operation.

TEST RESULTS AND ANALYSES

Global Behavior

Figure 4 shows the relationship between the total forces versus the overall drift angle, plotting the curves for loading from the amplitudes of 1/200 to 1/20. Here, the total force was the sum of the loads applied by the two jacks. Figure 5 shows the results of pushover analyses conducted in the course of the design of the test structure. The program code named "CLAP" [6] was used. The code is based on the direct stiffness method with member-by-member representation. Plastic hinges inserted at member ends represent plastification, with the relationship between the moment and plastic rotation taken to be bilinear. In Japan, such program codes are commonly adopted in daily seismic design practices. Since the analyses were carried out prior to the test, nominal strength values were adopted for the material strengths. Table 1 shows the nominal elastic stiffness and strength values adopted in the analyses. Note that the elastic stiffness values of the column bases were estimated in consideration of elastic elongation of anchor bolts. The four cases shown in Table 2 were analyzed and plotted in Fig.5. In some cases of analyses, composite action with RC floor slabs was taken into account, and both the stiffness and strength of composite beams were adjusted using the concept of "effective width." Using the effective width stipulated in the Japan's composite slab guideline [7], the elastic stiffness of the beams was enlarged by 1.8 times, and the positive moment strength was enlarged by 1.5 times, respectively. In some cases of analyses, panel-zone behavior, i.e., the size, flexibility, and yielding of panel-zones were also considered. The panel-zone strength was enlarged by 1.3 times the values calculated using the design equations. This is also a common practice in Japan to allow for rather significant hardening sustained by panel-zones. In all cases, no strain hardening after reaching the respective strength was considered. This is again a common seismic design practice in Japan.

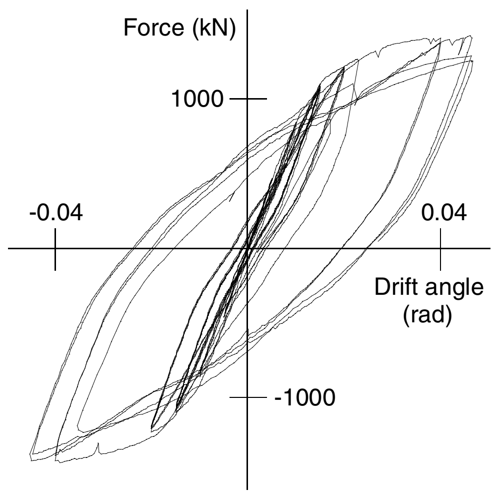


Fig. 4 Total load versus overall drift angle relationship

Table 1 Strength and stiffness values adopted pushover analyses (nominal values)

Column base			
Location	C_1	C_2	C_3
K_{cb} (kN·m/rad)	83400	99600	83400
${}_{cb}M_y$ (kN·m)	212	335	303

Beam			
Story	${}_{b}M_y$ (kN·m)	${}_{b}M_p$ (kN·m)	K_b (kN·m/rad)
1	353	359	60264
2	353	359	60264

Column				
Story	${}_{c}M_p$ (kN·m)		K_c (kN·m/rad)	
	Exterior	Interior	Exterior	Interior
1	357	461	53170	67773
2	357	357	56600	56600
3	357	357	134970	134970

Panel				
Story	${}_{p}M_y$ (kN·m)		${}_{p}M_p$ (kN·m)	
	Exterior	Interior	Exterior	Interior
1	381	495	472	613
2	381	381	472	472

Table 2 Analysis cases in pushover analysis

Analysis case	Composite action	Panel-zone effect
Case 1	Not considered	Not considered
Case 2	Considered	Not considered
Case 3	Not considered	Considered
Case 4	Considered	Considered

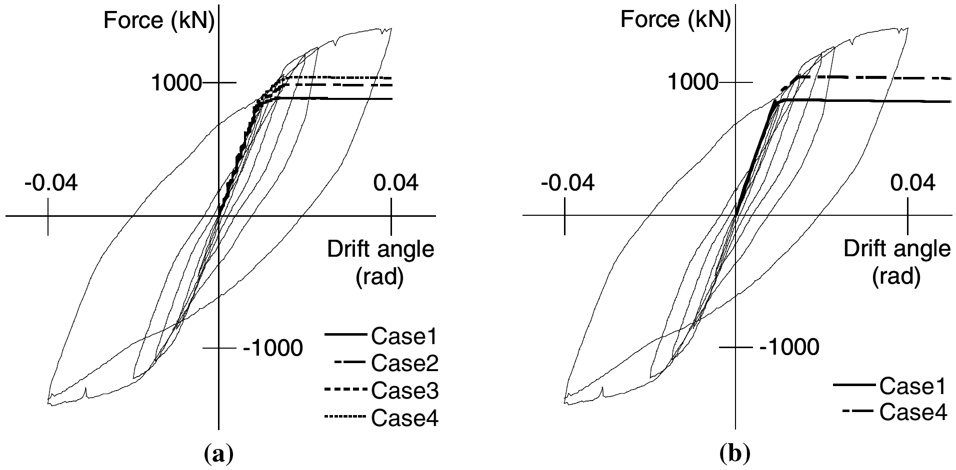


Fig. 5 Prediction by pushover analyses: (a) frame model using CLAP; (b) generic frame model

As for the elastic stiffness, the stiffness obtained for Cases 1 and 2 (without panel-zone effect) is 7.5% smaller than the experimental stiffness, while the stiffness obtained for Cases 3 and 4 is 3.9% smaller than the experimental stiffness. Correlation between the estimated and real is very reasonable. As evidenced from Fig.5, the maximum strength is about 40-60% greater in the test than in analysis cases. It is natural, because no strain hardening was taken into account in the analysis cases, and it is reasonable, too, if conservatism inherent to seismic design is reminded. Among the four analysis cases, Cases 1 and 3, both not considering composite action, provide the smallest strength. The two cases give nearly the same strength. Cases 4 provides the largest strength and is greater than 18% than the smallest strength given in Case 1.

Fig.5(b) shows the corresponding pushover analysis results obtained using the generic frame model proposed in [8]. In this model, all columns belonging to one story are represented by one representative column and all beams (and panel-zones) are represented by one rotational spring. Two cases corresponding to Cases 1 and 4 of the frame analyses are shown in Fig.5(b). The differences between the frame analyses and generic model analyses are 2.2% (maximum strength) and 0.7% (stiffness) for Case 1 and 0.4% (maximum strength) and 3.1% (stiffness) for Case 4, respectively. This clearly indicates the effectiveness of the generic frame model.

Cyclic Behavior

As shown in Fig.6, the test frame exhibited very stable behavior to the end of 1/25 amplitude. Small but visible cracks started during the cycles of the 1/25 amplitude and grew either from the toe of the weld access hole or from the edge of the runoff tab at a few beam ends. These cracks had no visible effects on the global behavior. During the first cycle in the positive loading of the 1/20 amplitude, the "North" plane's second floor beam was fractured from the beam bottom flange at the connection to the exterior column located on the loading jack's side. The fracture caused a sudden drop of the "North" plane's resistance by about 15% but the incremental stiffness for the succeeding loading was positive again. Details of the fracture behavior are found in [2].

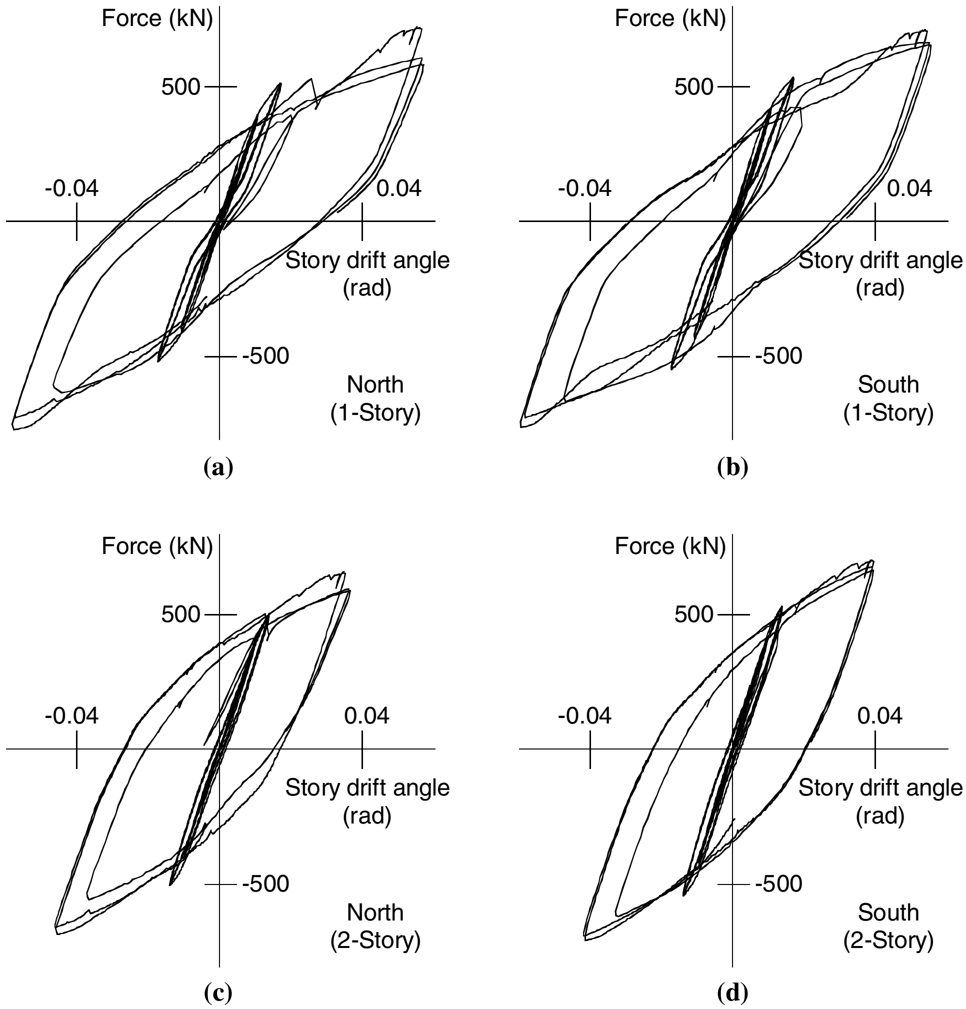


Fig. 6 Story shear versus story drift angle relationships:
(a) first story in "North" plane; (b) first story in "South" plane;
(c) second story in "North" plane; (d) second story in "South" plane

To examine how the analysis codes commonly used in seismic design and analysis are able to trace the experimental cyclic behavior, the analysis code adopted for the pushover analyses (conducted prior to the test) was used again. The analyses this time were different from the previous analyses in the following aspects. For one, yield strength values obtained from the associated coupon tests were used instead of the nomial strength values, resulting in a 31% increase for beams, a 32-35% increase for columns, and a 5-8% increase for column bases. For two, strain hardening after yielding was included, with the modulus of strain hardening (relative to the elastic stiffness) determined by trial and errors. For three, increase of moment capacity by composite action was adjusted based on the exeprimental results. For the last, a slip model was incorporated to represent the hystertic behavior of the column bases.

Figure 7 shows the results (for the cycles of 1/25 amplitude) thus obtained [Fig.7(a), (c), (e) for the frame model and Fig.7(b), (d) and (f) for the generic model]. In the development of the analytical curves, 2, 5, and 10% of strain hardening were adopted for the columns, column bases, and beams and panel-zones, respectively. The positive moment capacity was increased by 19% to allow for composite action. The thin and bold lines are the experimental and analytical curves. Correlation between the experimental curves and analytical curves obtained for the frame model is excellent, with the difference in the maximum strength not greater than 2.6% (positive) and 4.7% (negative), and the difference in the dissipated energy (areas of enlosed loops) not greater than 4.0%. Pinching behavior notable particularly in the first story is also reproduced very reasonably. In reference to Fig.7(b), (d) and (f), correlation between the test and generic model is also excellent.

Analysis parameters (degree of strain hardening and increase in the positive bending moment) were chosen in reference to the experimental results; hence the analyses, typical post-analyses, are not fair in terms of "prediction." The writers' contention is that the analyses commonly used in daily design and analysis practices are reasonable enough to duplicate the inelastic behavior up to the drift angle of 1/25, which is significantly larger than the range of deformations considered in contemporary seismic design, just by considering strain hardening and composite action. How much hardening and composite action to consider is a subject of further exploration.

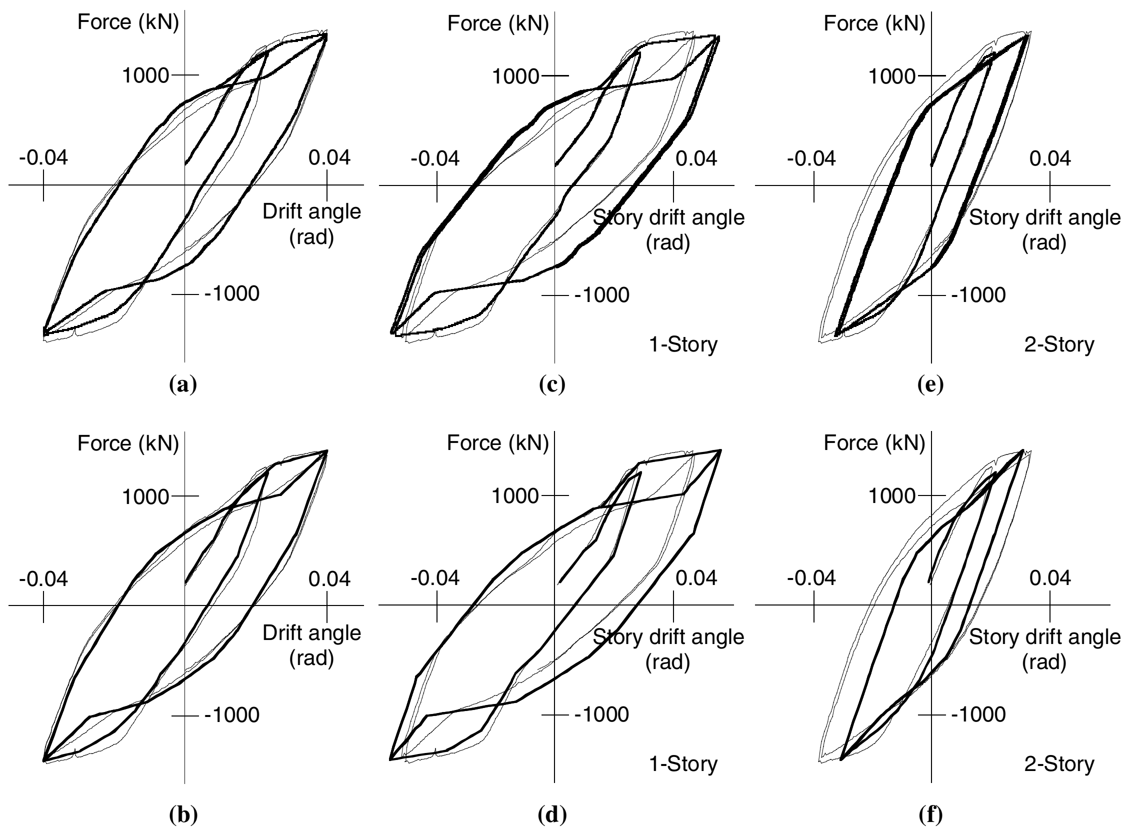
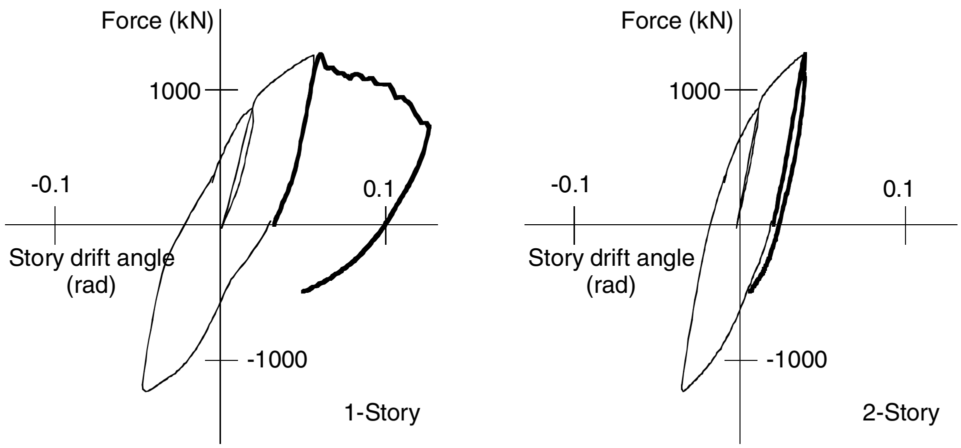


Fig. 7 Comparison between test and post-analysis:
(a) overall (frame); (b) first story (frame); (c) second story (frame);
(d) overall (generic); (e) first story (generic); (f) second story(generic)

Collapse Behavior

Figure 8 shows the story shear versus story drift angle relationship for the last portion of loading with large drift angles. The first story shear decreased significantly with the increase in story drift angle from 1/20 to 1/8 [Fig.8(a)], whereas the second story was unloaded [Fig.8(b)]. Formation of a first-story collapse mechanism was the primary reason for the drop in resistance. The moment resistance of the column bases decreased seriously during the last-stretch of loading, because of the combined effect of plastic elongation of anchor bolts and the crash of concrete placed underneath the column base plates.

This decrease moved the column's inflection point lower and increased the bending moment at the column top, which eventually reached yielding. The first story's unstable behavior was accelerated because of local buckling at the column top (Fig.9). The width-to-thickness ratios of the first story columns were 25 (interior columns) and 33 (exterior columns), which were not compact in the classification of AISC 2000 Seismic Provisions. Unless the effects of concrete crashing at the column bases and local buckling and succeeding strength degradation of column ends are reflected into the analysis model, it is to duplicate the collapse behavior observed experimentally. A separate study is ongoing to refine the analysis codes for both the frame analysis and generic frame analysis.



(a) **(b)**

Fig. 8 Unstable behavior in formation of first-story collapse mechanism: (a) first story; (b) second story

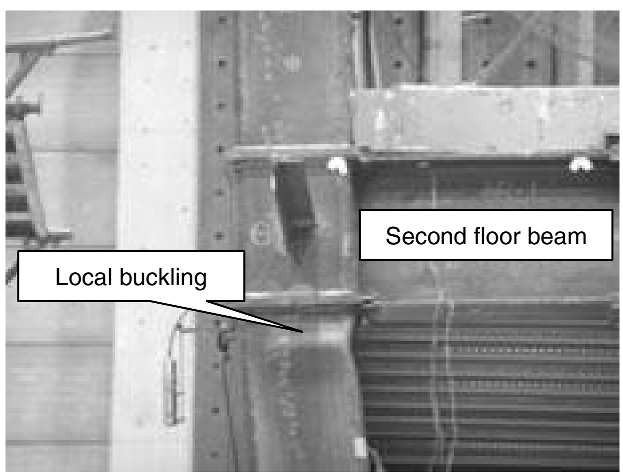


Fig. 9 Local buckling at first story column top

CONCLUSIONS

This paper presented an overview of the test program in which a three-story steel moment frame was loaded cyclically to failure and discussed on the representative results, including the ability of the numerical analyses commonly adopted in daily design practices to duplicate the experimental behavior. Major findings obtained in the study are as follows.

(1) Up to the overall drift angle of 1/25, the test frame exhibit very stable behavior, with balanced deformations between the beams, panel-zones, and column bases (primarily due to yielding of the anchor bolts). Pinching behavior was notable for cyclic loading with larger amplitudes (up to 1/25 in the overall drift angle) primarily because of cyclic yielding and resulting slip-type hysteresis experienced at the column bases.
(2) Pushover analyses conducted prior to the tests predicted the elastic stiffness very reasonably and the strength with a good amount of conservatism. This indicates that present numerical analyses commonly adopted in daily design practices are adequate as design tools.
(3) Including strain hardening after yielding and composite action, numerical analyses were able to duplicate the cyclic behavior of the test structure with great accuracy, although a reasonable procedure to determine the degrees of hardening and composite action is yet to be explored.
(4) The generic frame model traced the experimental behavior very accurately, indicating the effectiveness of this model in design practice.

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