THE UNIVERSITY of EDINBURGH

Edinburgh Research Explorer

Experimental study on progressive collapse resistance of steel frames under a sudden column removal scenario

Citation for published version:
Li, G, Li, L, Jiang, B & Lu, Y 2018, 'Experimental study on progressive collapse resistance of steel frames under a sudden column removal scenario', *Journal of Constructional Steel Research*, vol. 147, pp. 1-15. https://doi.org/10.1016/j.jcsr.2018.03.023

Digital Object Identifier (DOI):
10.1016/j.jcsr.2018.03.023

Link:
Link to publication record in Edinburgh Research Explorer

Document Version:
Peer reviewed version

Published In:
Journal of Constructional Steel Research

General rights
Copyright for the publications made accessible via the Edinburgh Research Explorer is retained by the author(s) and / or other copyright owners and it is a condition of accessing these publications that users recognise and abide by the legal requirements associated with these rights.

Take down policy
The University of Edinburgh has made every reasonable effort to ensure that Edinburgh Research Explorer content complies with UK legislation. If you believe that the public display of this file breaches copyright please contact openaccess@ed.ac.uk providing details, and we will remove access to the work immediately and investigate your claim.

OPEN ACCESS

Download date: 25. Apr. 2024

Experimental study on progressive collapse resistance of steel frames under a sudden column removal scenario

Guo-Qiang Li[1,2,*], Liu-Lian Li[1,3], Bin-hui Jiang[1], Yong Lu[4]

[1]*College of Civil Engineering, Tongji University, Shanghai 200092, PR China*
[2]*State Key Laboratory for Disaster Reduction in Civil Engineering, Tongji University, Shanghai 200092, PR China*
[3]*China State Construction Technical Center，Beijing 101300，PR China*
[4] *Institute for Infrastructure and Environment, School of Engineering, University of Edinburgh, Edinburgh EH9 3JL, UK*

Abstract: A comprehensive experimental apparatus for the progressive collapse testing of steel frames has been developed. The apparatus is suited for the testing of planar steel frames to study the load transfer process and the progressive collapse resistance of steel structures under a column removal scenario. In order to simulate a sudden removal of a middle column at the ground storey of the frames, a removable column unit has been designed to allow for an instantaneous knock-out by a pendulum hammer during the test. To avoid the out-of-plane instability of the planar steel frames, an out-of-plane restraining system has been designed and integrated into the test apparatus. Weights simulating the desired gravity loads were attached to the test frame through holding baskets, which were designed to minimize unwanted shaking and ensure that the suspended baskets moved together with the deformed steel frames during the tests. Experimental results showed that the column removal mechanism in the test apparatus was effective. Using this apparatus, the dynamic behaviour of three planer steel frames under a column removal scenario was investigated. Based on the measured deformations and strains, the dynamic response, collapse modes, load transfer path of the steel frames after the removal of the middle bottom column are studied.

Keywords: Steel frame; progressive collapse; experimental study; test apparatus; dynamic response; collapse modes

[*]Corresponding author, Tel: +86-21-65982975, Email: gqli@tongji.edu.cn

1. Introduction

Among major civil engineering incidents, progressive collapse triggered by a local structural failure is generally recognized as one of the most devastating types of structural failures. According to ASCE-7 [1], progressive collapse represents "the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it". Studies have shown that most of the past progressive collapse cases were attributable to external event including blast and impact [2, 3].

The process of progressive collapse of a structure subjected to blast and impact types of loadings may be divided into two different stages: a) abrupt failure of one or more load carrying members due to the direct loading effect, i.e. development of the local failure, and b) the structural response to the local failure, leading to either a rebalanced system or the collapse of the whole or a large part of the structure. Generally speaking, progressive collapse is complex as it involves dynamic response, inelastic behaviour, large deformations and contact-impact of structural members [4, 5]. Prevention of progressive collapse and improving the capacity of structures in withstanding local failure has become a key area of research in structural engineering.

In recent years, considerable amount of efforts has been devoted into studying the behaviour and load transfer mechanisms of frame structures during the progressive collapse. On the numerical simulation front, different approaches have been studied for modelling the structural behaviour in a progressive collapse scenario. For instance, Krauthammer [6] used the finite element code DYNA3D to investigate the influence of the structural concrete and steel connections on the robustness of blast resisting structures. Lee et al. [7] investigated two nonlinear methods for the analysis of the

resistance of welded steel moment frames, using the four-node quadrilateral shell elements in ABAQUS. Bao et al. [8] used a macro model-based approach, available in the nonlinear FE software DIANA, to numerically simulate the potential for progressive collapse of a typical reinforced concrete (RC) moment frame structure. Kripakov et al. [9] used finite-element ADINA models and studied a simplified approach to assess the structural stability of underground mine structures. Brunesi and Nascimbene [10] employed an open access procedure using a fiber-based model to simulate the dynamic response of RC buildings subjected to a sudden column loss.

On the experimental front, numerous studies have been performed using a quasi-static testing method. Yang and Tan [11] studied the performance of bolted steel beam-column joints under a central-column removal scenario. Lew et al. [12] investigated the performance of beam-column assemblies with two types of moment-resisting connections under a column removal scenario. Tsitos et al. [13] tested two 1/3 scale three-story, two-bay steel frames to evaluate the effectiveness of earthquake resistant design details in enhancing the progressive collapse resistance of steel frames. Yi et al. [14] conducted a 1/3 scaled progressive collapse test of a 4-bay and 3-storey plane reinforced concrete (RC) frame. A collapse resistance test of a 1/8 scaled 4-bay and 3-storey plane RC frame was performed by Sagiroglu [15]. Xie and Shu [16, 17] carried out a space steel frame experiment with 2 storeys to investigate the changes of the internal forces due to the loss of a column.

A number of experimental studies have also been carried out in attempt to simulate the dynamic effects in the progressive collapse process. Xie [17] employed a cylinder to simulate the failure of the middle column and conducted tests of three planer frames with a sudden failure of a middle column to study the progressive collapse resistance

of steel frames. Chen et al. [18] reported an experimental test on a full-scale two-story steel frame, where a perimeter column was suddenly pulled down by a chain block, to evaluate the influence of the concrete slabs on the progressive collapse resistance of steel moment-frame buildings. Xiao et al. [19, 20] used a hydrogen gun to dislodge concrete blocks inserted in the mid-section of a column to simulate a sudden removal of the column. Using this method, they investigated the dynamic response, failure mechanism and changes in the load transfer paths of a 3-bay and 3-story, 1/2 scaled space RC frame structure subjected to a series of sudden column removals. Sasani and Sagiroglu [21] studied the progressive collapse resistance of an actual six-story RC frame structure subjected to a blast scenario in which one corner column and one adjacent column along the short span direction were removed by blast. Sasani et.al [22] studied the progressive collapse resistance of an actual 11-story RC structure subjected to a sudden removal of four first-floor neighbouring columns and two second-floor perimeter deep-beam segments by explosion.

For most of the numerical studies reported in the literature on the progressive collapse of steel structures, there has been a general lack of verification by experimental data. On the other hand, the sudden removal of a column due to blast or explosion will cause dynamic effect on the progressive collapse of frame-type structures, and the dynamic effects tend to amplify the gravity load in the progressive collapse and such effects are dependent on the column removal time. Generally the smaller the column removal time, the larger the dynamic effects. However, few previous experiments on frame-type structures under a column removal scenario have simulated this critical factor in a realistic manner. As mentioned above, most of previous experiments used the pseudo-static loading method [11-17], which do not represent the dynamic effect of the damaged structure in the progressive collapse. Some experiments involved the

process of removing the column, but the column removal time was not short enough [17-20], thus the dynamic effect of the damaged structure on the progressive collapse was under-represented. In addition, the structure responded almost in the elastic region after the removal of the columns in some experiments [18-22] , consequently the change in the load transfer paths and the collapse modes of the damaged structure could not be identified.

As a conservative treatment, two most recent guidelines [23, 24] recommended an instantaneous column removal as the principal design scenario for progressive collapse mitigation [25]. For these reasons, a more reliable experimental approach that is capable of resembling a sudden local failure (removal of a column) and testing the dynamic effects into the inelastic region is needed.

In this paper, a comprehensive experimental apparatus for the progressive collapse testing of steel frames with a rapid removal of a column has been developed so that the dynamic effect of the structure in the progressive collapse can be reliably reflected. Three planar steel frames were designed to represent different structural conditions and they were tested to study the load transfer process and progressive collapse resistance mechanisms under an instantaneous removal of a column scenario. The effectiveness of the testing system concerning the dynamic effects induced by the removal of a column is evaluated based on measured dynamic response. Detailed measurements of the time histories of deformations and strains of the steel frame specimens are presented and examined. The load transfer mechanisms and collapse modes due to different design considerations are observed and discussed. The experimental results also provide benchmark data for verifying numerical models for the analysis of the progressive col1lapses resistance of steel structures under a sudden column removal.

2. Test steel frame structures and overall test setup

Three two-storey, four-bay planar steel frames were designed and fabricated to evaluate their resistance against progressive collapse under a typical scenario with a sudden removal of a middle column at the first storey, as schematically illustrated in Fig. 1. The temporary replacement support for the middle column will be explained later.

As full-scale tests for progressive collapse studies would have been extremely costly and time-consuming, scale-down specimens of steel frames were chosen for the tests. The scale ratio was about 1:3, which actually belonged to a medium-scale range and was considered reasonable for the present study with a focus on the load transfer process and the progressive collapse resistance mechanisms of steel structures.

It should be noted that at this reduced scale it was already difficult to fully preserve a realistic damage process within the joints. For this reason, the joints in the test frames were intentionally strengthened to eliminate joint failure, and instead observations were focused on the nonlinear response and damage process in beam-column members and due to column instability.

The dimensions of the test frames, including the storey height, span length, and the cross-section sizes of beams and columns are listed in Table 1. The members were made of H-section with dimensions as shown in the table for different members, for example H80×50×3×4 stands for H-section with overall depth of 80mm, flange width of 50mm, flange thickness of 3mm and web thickness of 4mm, respectively. The columns and beams were arranged to be subject to bending about the strong axis.

Table 1. Summary of the dimensions of steel frame specimens

Specimen	Beam section	Column section	Story height		Span length
			h_1	h_2	l
	mm	mm	mm	mm	mm
FRAME1	Middle bay: H54×50×4×4 Side bay: H80×50×3×4	H100×100×6×8	1227	1054	2100
FRAME2	H54×50×4×4	H54×50×4×4	1227	1054	2054
FRAME3	H54×50×4×4	H100×100×6×8	1227	1054	2100

The first test frame, FRAME1, was focused on the behaviour of the beam-column critical regions in a middle column removal scenario, with enhanced fully welded connections (Fig. 2a). FRAME2 and FRAME3 were focused on the influence of the column performance on the resistance of the frames against progressive collapse, and to avoid connection failure the beam-to-column connections were further strengthened by stiffeners (Fig. 2b and Fig. 2c). In all the specimens, the beams and columns were strengthened by distributed stiffeners to limit the influence of the local buckling on the global behaviour of the steel frames.

To simulate a fixed support at each column base, the column foot was strengthened by steel plates and rigidly fastened to the steel base using high-strength bolts. The steel base was fixed on the strong floor using anchor bolts (Fig. 2d). A schematic diagram of a test frame is presented in Fig. 3.

To obtain the representative stress-strain curves of the steels used for the frame specimens, three coupons were cut from each structural steel section. Monotonic tensile test was performed in accordance with a standard procedure [26]. Table 2 lists the mean values of the measured mechanical properties, where σ_y, ε_y, σ_u, and ε_u are yield stress,

yield strain, ultimate strength and ultimate strain corresponding to σ_u, respectively, and Δ% represents the percentage of elongation after rupture.

Table 2. Material property of structural steel for steel frame specimens

Sections	Steel grade		σ_y/MPa	ε_y	σ_u/MPa	ε_u	Δ %
H54x50x4x4	Q345	web	417.0	0.0020	575.0	0.116	24.2
		flange	394.5	0.0019	508.2	0.145	27.1
H80x50x3x4	Q235	web	326.2	0.0016	467.7	0.175	27.2
		flange	318.3	0.0016	446.7	0.164	31.5
H100x100x6x8	Q345	web	409.8	0.0020	563.0	0.165	27.6
		flange	386.2	0.0019	547.7	0.156	29.4

3. Experimental apparatus: specific considerations

3.1 Out-of-plane supporting setup

In order to avoid the out of plane instability of the planar steel frame specimens, an out-of-plane supporting system was designed and installed. Fig. 4 shows an overall view of the out-of-plane supporting setup. The main supporting rigs were fixed on the ground using anchor bolts.

The struts that restrained the out-of-plane movements were fixed at one end onto the midspan of each beam of the test steel frame, and attached at the other end to the flanges of the side-supporting frames through a set of rollers. In this way, the test frame was allowed to move freely within its plane while any out-of-plane movement was prevented.

3.2. Gravity load setup

The attachment of added loads to the test steel frame was an important aspect of the test setup. The design of the holding basket (box), illustrated in Figure 5, was made to

ensure the following: a) avoid out-of-plane movement, b) the whole set of the basket including the weights rigidly connects to the steel beam and moves in a synchronous manner, and c) avoid slipping – this is achieved by welding steel plate stoppers, as also shown in Figure 5.

As a main objective of this study is to develop a better understanding of the load transfer mechanisms and the progressive collapse failure modes of the steel frames, the applied loads of the beams were larger than the design loads specified in codes in order to yield large deformations following the column removal and expose the catenary action in the steel beams. Based on the finite-element analysis, the amount of applied weights on each beam was determined considering the possible progressive collapse modes, namely a connection fracture mode, a loss of stability mode, and a catenary action mode associated with large deflections of beams. Fig. 6 and Table 3 show the loads applied on each beam of the test frames. The weight in each suspended basket is $2P_{1m}$ for middle spans and $2P_{1s}$ for side spans.

Table 3. Loads imposed by each holding basket

Load/kN	FRAME1	FRAME2	FRAME3
P_{1m} (Middle bay)	3.30	3.85	3.85
P_{1s} (Side bay)	1.70	2.10	2.10

3.3. Column removal setup

The dynamic effect due to a sudden loss of column plays a significant role in the progressive collapse process of structures [27]. It is generally known that the dynamic effect increases with decreasing column removal time; the faster the column removal, the larger the dynamic effect [23, 24]. To simulate such dynamic effect in a realistic manner in an experiment, the target column should be removed over a time period that is no more than 1/10 of the period associated with the dominant structural response

mode, as stated in DOD 2013[23] and GSA2003[24]. In the present experiment, a special removal (dummy) column mechanism along with a pendulum hammer was devised to achieve a rapid removal of the target column, as schematically illustrated in Fig. 7.

The removable column was made up by a three-hinged strut, as shown in the photos in Fig. 8. Prior to the actual test, the hinged strut was kept in a straight position to simulate the effect of the column by inserting a brittle (glass) locking rod through an additional hole at the middle hinge, making it effectively a fixed connection. Gravity loads were then applied through the holding baskets. During the test, the pendulum hammer was pulled to a designated position and subsequently released to strike the removable column at the middle position (as shown in Fig. 9a). As the middle hinge started to rotate, the glass locking rod was broken by the impact of the pendulum hammer (Fig. 9a), and this triggered the hinge mechanism and eliminated the vertical load carrying capacity of the removable column, and thus initiated the progressive collapse of steel frame specimens (Fig. 9b).

3.4 Instrumentation

The instrumentation arrangement was similar for the three test experiments, as shown in Fig. 10. The instrumentation consisted of the following components.

(1) Strains gauges: For each specimen, 29 uniaxial strain gauges with a measuring range up to 50000 $\mu\varepsilon$ were used to measure the strain at selected sections of the steel beams and at critical locations in the columns. The detailed locations of the strain gauges at the sections B_1 to B_7 of beams and sections C_1 to C_4 of columns are indicated in Fig. 10b. The measured strains can be used to calculate the internal forces in the beams and columns and to observe the development of catenary action in the beams during the progressive collapse of steel frames due to a column removal.

(2) Displacement transducers: Cable-Extension displacement transducers with a measuring range of 2000 mm were used to measure the displacements of the steel frame specimens. As shown in Fig. 10a, two displacement transducers, D_1 and D_2, were used to measure the horizontal displacements at the side of the steel frame, while 4 displacement transducers, D_3 to D_6, were installed along the central beams to measure the in-plane vertical deflections.

Strain and displacement data were collected using dynamic data acquisition system DH5921 at a sampling frequency of 1000 Hz.

(3) Digital Image Correlation (DIC): Around the column loss location, two high-speed video cameras were used to capture the dynamic movement of the steel frames in the short time period during and after the removal of the middle column of the ground storey. Images were recorded at a rate of 150 frames per second, and then processed by the video gauge software to obtain the displacement time histories of measurement points D_7 at the location of the removed column and D_8 on the holding basket, as shown in Fig.10a. The velocity and acceleration time histories of measurement points D_7 and D_8 could be determined through the displacement time histories. The images recorded have been also used to determine the time when the column is completely removed.

4. Effectiveness of the experimental setup

4.1 Effectiveness of the column removal setup

From the analysis of the high speed camera records (Fig. 11), the effective column removal time can be identified based on the period between the moment when the pendulum hammer came into contact with the removable column and the moment when the column was completely disengaged. The results indicate that the time taken for the removable column to be removed, ΔT, was around 0.02 sec.

On the other hand, the natural period associated with the vertical response following the removal of the column can be determined by the dynamic vertical displacement measurement. The vertical displacement time history at the measuring point D_6 of FRAME3 after the column removal is shown in Fig. 12a. It can be seen that the dominant natural period T_1 was around 0.5sec. With a similar procedure, the dominant natural periods for the other two test framed were found to be 0.45 and 0.6 s, respectively. Therefore the column removal time ΔT was considerably smaller than the $1/10^{th}$ of the period T_1. This indicates that the column removal setup successfully achieved the simulation of an instantaneous removal of column as specified by DOD2013 [23] and GSA2003 [24] for including realistically the dynamic effect on the progressive collapse of structures.

Fig. 12b shows the vertical velocity time histories at the location of the removed columns of FRAME2 and FRAME3 respectively. The maximum velocity at the measuring point D_7 was 1.5m/s and 1.1m/s for the two frames, respectively. These dynamic measurements demonstrate clearly the dynamic phase of the response after the removal of removable column.

4.2 Effectiveness of out-of-plane supporting setup

The effectiveness of out-of-plane support was checked by the displacement time histories at specific measurement points. In most of the cases the maximum out-of-plane displacement as measured at point D_7 (Fig. 13) after the column removal was less than 10 mm. This indicates that the out-of-plane supporting setup was effective in preventing the out of plane instability of the planar steel frame specimens.

4.3. Effectiveness of the gravity load setup

To validate the effectiveness of the loading setup, measurements were taken of the vertical accelerations at the location of removed column (points D7) and at the top of the adjacent holding basket (points D8), as illustrated in Fig. 14. It can be seen that these vertical accelerations are in good agreement, indicating that the holding baskets moved synchronously with the steel frame after the removal of middle column.

5. Experimental results and analysis

5.1 Global behaviour of steel frame specimens

After the instantaneous removal of the first storey middle column, all three steel frame specimens resisted progressive collapse and no severe material failure such as fracture of the beam-to-column connections took place. However, significant local buckling occurred at the bottom flange of beams near the beam-column connections in the adjacent spans to the removed column, as shown in Fig. 15. Note that in the figure "I" "II" stands for the first and second storey of the steel frames, respectively, and ② stands for the column number illustrated in Fig. 1.

A summary of the experimental results on the global deformation of the steel frames after the middle bottom column removal is listed in Table 4. The symbols are shown in Fig. 16, where Δ_{Vmax} is the maximum vertical displacement at the location of the removed columns, θ_{max} is the maximum rotation at the other end of the beams adjacent to the removed column, and Δ_{Hmax} is the maximum inward horizontal displacement on the two sides of the steel frames. It should be noted that the rotation θ_{max} is obtained by dividing the maximum vertical deflection Δ_{Vmax} by the respective

original span length of the frames, namely 2100mm for FRAME1 and FRAME3, and 2054mm for FRAME2.

Fig. 17 shows an overall view of the deformed shapes of the steel frame specimens after the removal of the middle bottom column. FRAME 1 and 3 exhibited large permanent deformations, whereas FRAME2 (the strong beam-weak column frame) effectively failed as the columns adjacent to the removed column failed in bending and compression.

Table 4. Global deformations of steel frame specimens

Specimen	Δ_{Vmax} (mm)	θ_{max} (rad)	Δ_{Hmax} (mm)
FRAME1	252	0.120	15.4
FRAME2	454	0.221	90.0
FRAME3	249	0.119	16.0

5.2 Displacement-time histories

The time histories of vertical deflections of the three steel frame specimens at the location of the removed column, namely Points D_6 and D_7 in Fig. 10, are shown in Fig. 18. The negative values represent the downward deflections, and the test started at around 0.5 s in the time axis. It can be seen that the maximum dynamic deflections were 252 mm, 454 mm and 249 mm for FRAME1, FRAME2 and FRAME3, respectively and the permanent deflections were 228 mm, 443 mm and 225 mm for the three frames, respectively. The vertical deflections measured by the displacement transducers (D_6) were very close to those measured by the high speed cameras (D_7), and this

demonstrates that both methods worked well in capturing the dynamic measurement of large deformations.

It is worthwhile to note that FRAME3 was subjected to an increase of about 20% in the gravity load as compared to FRAME1, while the middle bay beams and columns were of the same section sizes except that the connections in FRAME3 were strengthened, as can be seen in Fig. 2. From the measured responses it can be observed that FRAME3 experienced almost the same deflection time histories as FRAME1, and this suggested that by strengthening the connections the progressive collapse resistance of the steel frame was effectively enhanced.

FRAME2 had relatively small column sections, therefore, when it was subjected to the same gravity load as FRAME3, it exhibited an increase of almost 100% in the peak vertical deflection from that of FRAME3 under the middle bottom column removal scenario.

Fig. 19 shows the horizontal displacement time histories measured at first storey side joint and second storey side joint (Points D_1 and D_2 in Fig. 10) after the sudden removal of the first-storey middle column in FRAME1 and FRAME2. The dynamic maximum horizontal displacements at these points were 11.3 mm and 15.4 mm for FRAME1 and 18.5 mm, 26.4 mm for FRAME2, respectively, and the permanent horizontal displacements were 8.5 mm, 10.8 mm and 13 mm, 20 mm respectively. The storey drift angle of the remaining columns at the first story was approximately 0.01 rad in both frames.

5.3 Load redistribution

As in a typical progressive collapse scenario, the experimental program on the steel frame specimens was executed in two phases. In the first phase, the uniformly distributed vertical load was applied to the beams by attaching the weights in the holding baskets. In the test phase after applying the loads, the column removable process was carried out and the dynamic responses were recorded.

The initial strains after applying the loads at sections B_1 and B_4 of FRAME1 and FRAEM2 are listed in Table 5. The negative values of strains indicate compression. Before the column removal, all strains were less than the nominal yield strain of 2×10^{-3} and the steel was well in the elastic stage. The neutral axis was approximately in the mid-height of the web, indicating that these beam sections behaved primarily in flexure with the top flange (B_{XT}) in tension and the bottom flange (B_{XB}) in compression. It is also noted that the strain levels in the beam sections B_1 and B_4 for FRAME1 and FRAME2 were basically the same.

Table 5. Initial strains at beam sections $B_1$and B_4 (after applying the gravity loads)

Specimens	B_1（$\mu\varepsilon$）			B_4（$\mu\varepsilon$）		
	B_{1T}	B_{1M}	B_{1B}	B_{4T}	B_{4M}	B_{4B}
FRAME1	1243	22.5	-1164	954	-1.5	-1104
FRAME2	1130	120	-1180	1068	-145	-1230

Fig. 20 shows the strain time histories of beam sections B_1 and B_4 after the removal of the middle column at the first storey for the FRAME1 and FRAME2. It can be seen

that all strains at the flanges were larger than the nominal yield strain and the tensile strains at the bottom flanges B_{1B} exceeded the compressive strains at the top flanges B_{1T}, which indicates the presence of tensile axial forces and the development of the catenary action in the beams above the removed column. It is generally understood that as the deformation increases the applied loads on the frame would gradually be resisted by the vertical components of the axial forces developed in the beams after the column removal [28].

Table 6. Final strains and tensile axial force at beam sections $B_1$and B_4 after the column removal

Specimens	B_1（$\mu\varepsilon$）			B_4（$\mu\varepsilon$）			N_{B1}	N_{B4}
	B_{1T}	B_{1M}	B_{1B}	B_{4T}	B_{4M}	B_{4B}		
FRAME1	-5398	6601	17931	-3444	-533	-4827	0.220N_y	0.061 N_y
FRAME2	-23265	6545	40460	-12584	150	15084	0.097 N_y	0.035 N_y

The final strains at beam sections B_1 and B_4 are listed in Table 6. Considering the strain-hardening and the measured stress-strain relationships mentioned in Section 2, the tensile axial forces at beam sections B_1 and B_4 can be calculated [29], and these are also listed in Table 6, where N_y is the yield axial force of the steel beam. It can be observed that after the sudden column removal, significant tensile axial forces developed in the beams at the first floor of the steel frames, whereas less than half of these axial forces developed in the beams at the second floor. This may be explained by the fact that the development of the catenary action requires effective axial restraint at the beam ends. The stronger the axial support to the beams provided by the columns and adjacent beams, the larger the possible catenary action to be developed in the beams.

In the tested frame specimens, the columns provided larger axial restraint to the beams at the first floor than at the second floor, therefore even the vertical deflections were similar after the column removal, and the tension force in the beams at the first floor was much larger than that in the beams at the second floor.

In between the frames, the axial force in the beams at the first floor of FRAME2 after the column removal was less than that of FRAME1, although the beam deflection of FRAME2 was larger than that of FRAME1 after the column removal. This can be explained by the fact that the column sections of FRAME2 were smaller than that of FRAME1, thus provided smaller restraint to the beams.

Section B_7 of the beams located in the side span where no column was removed. The initial strains (after applying the gravity loads) at beam section B_7 of specimens FRAME1 and FRAEM2 were found to be well within the elastic range with the maximum value being less than 400 $\mu\varepsilon$. After the removal of the column, it was found that tensile axial forces were also produced in the beam section B_7; however the amount of the tensile axial force was generally insignificant.

Fig. 21 depicts the final displacement profile of the beams in the middle bay of FRAME1 and FRAME2. It can be observed that the deflection profiles of the beams tend to change from a largely bending mode (FRAME1) to a three-hinge mode (FRAME2) as the final deflection increased.

For the strains in columns, in FRAME3 the initial strains at the middle column section C_1 and its adjacent column section C_2 after applying the gravity loads were -22 $\mu\varepsilon$ and -23 $\mu\varepsilon$, respectively. After the removal of the middle column at the first

storey, the strain of the section C_1 and C_2 were 2 $\mu\varepsilon$ and -51 $\mu\varepsilon$, respectively. It is evident that the force resisted by the removed column was transferred to the surrounding columns of the frame after the column removal. It can be concluded therefore that there was no problem with the adjacent columns due to the removal of the column in this frame.

The initial strains at the column bottom section C_3 of FRAME1 and FRAME2 after applying the gravity loads are listed in Table 7. It is seen that these columns were primarily subject to compression.

Table 7. Initial strains at column bottom sections C_3 after applying the gravity loads

Specimens	C_3（$\mu\varepsilon$）		
	C_{3L}	C_{3M}	C_{3R}
FRAME1	-61	-48	-50
FRAME2	-264	-210	-170

The strain time histories of the column bottom section C_3 for FRAME1 and FRAME2 are shown in Fig. 22. Tensile strain developed in the left flange of the column while compressive strain in the right flange of the column continued to increase. Combined with the strains due to the applied gravity loads, the total strain at the section C_3 was less than 1000 $\mu\varepsilon$ for FRAME1 and about 1500 $\mu\varepsilon$ for FRAME2, and they were still within the elastic range. The bottom ends of these adjacent columns were primarily subjected to bending after the removal of the middle column.

For a comparison, the initial strain at the adjacent column bottom section C_4 of FRAME2 (the strong beam-weak column frame) were -150 $\mu\varepsilon$, -314 $\mu\varepsilon$, and -214 $\mu\varepsilon$ in the left and right flanges and the web, respectively, due to the applied gravity loads.

After the removal of the middle column at the first storey, the strain time histories at this column bottom section for FRAME2 are shown in Fig. 23. It can be observed that much larger tensile strain in the right flange of column developed at the section C_4 than that at section C_3. The final strain at the above-mentioned locations of section C_4 for FRAME2 (the strong beam-weak column frame) were -15555 $\mu\varepsilon$, 8205 $\mu\varepsilon$, -3745 $\mu\varepsilon$, respectively. These strains demonstrate that the column at section C_4 was primarily subjected to bending moment after the removal of the middle column and was well into the plastic stage. This indicates that FRAME2 collapsed due to the failure of the columns adjacent to the removed column in bending and compression.

6. Conclusions

A dedicated experimental setup has been developed to simulate the dynamic response and the load transfer mechanisms for steel frames in a progressive collapse scenario involving a sudden removal of a column. In particular, a removable column element allowing a three-hinged mechanism to activate upon an imposed impact has been devised. Three planar steel frame specimens have been designed and tested. By analysing the measuring results obtained from the experiments under the middle first storey column removal scenarios, the effectiveness of the experimental setup has been evaluated and the dynamic behaviour of steel frames has been investigated. The following conclusions can be drawn:

1. The column removable mechanism performed well in simulating a sudden loss of the column. The time duration for the complete removal of the column was around 0.02 sec, which was well within $1/10^{th}$ of the dominant natural frequency (about 0.5s) after the removal of the column. This satisfies the requirement for an

instantaneous column removal as specified by DOD2013 [23] and GSA2003 [24] for a realistic representation of the dynamic effects in a structural progressive collapse. The setup for simulating the gravity loads via rigid holding baskets also proved to work effectively. The column removal mechanism and the weight-holding method are generally applicable for this type of experiments.

2. The test results confirmed that the progressive collapse resistance of the steel frame was effectively enhanced by strengthening the connections. However, steel frames designed in accordance with a strong beam-weak column approach is prone to a progressive collapse due to buckling of the columns adjacent to the removed column.
3. The measured response experimentally demonstrated the path of load transfer from the suddenly removed column to the surrounding columns of the frame through the beam-catenary mechanism as a dynamic process. The dynamic amplification of the gravity load forced the beams above the removed column to respond into the catenary stage before a re-balanced state was attained.
4. In the test steel frames, the catenary action induced by the tensile axial force in the beams appeared to be considerably more significant at the first floor of the frame, whereas for the beams of the second floor, the tensile axial force was relatively small and the catenary action was not significant. This phenomenon is believed to be related to the frame configuration and the horizontal restraining condition on the beams above the removed column, and it is a phenomenon that should be appropriately taken into account when a substructure, often in the form of a double-span beam assembly, is analysed as a representation of the frame.

It should be noted that in the large deformation stage, damage and even failure of the beam-column connections could occur. The consequence of severe damage or failure

at the beam-column connections can be significant in terms of the overall progressive collapse resistance. Therefore, a realistic preservation of the connection behaviour in the test specimens is an important aspect that should be considered in future experimental studies.

Acknowledgments

The study presented in this paper was supported by the National Natural Science Foundation of China through Grant No.51120185001, which is gratefully acknowledged.

References

[1] ASCE.SEI, Minimum Design Loads for Buildings and Other Structures. Standards. Vol. ASCE 7-05. 2005, Washington DC: American Society of Civil Engineers.

[2] Fabian, C.H., Analysis of Events in Recent Structural Failures. Journal of Structural Engineering, 1985. 111(7): p. 1468-1481.

[3] Kumalasari, W. and C.H. Fabian, Study of Recent Building Failures in the United States. Journal of performance of constructed facilities, 2003. 17(3): p. 151-158.

[4] Khandelwal, K. and S. El-Tawil, Collapse Behavior of Steel Special Moment Resisting Frame Connections. Journal of Structural Engineering, 2007. 133(5): p. 646-655.

[5] Starossek, U., Typology of progressive collapse. Engineering Structures, 2007. 29(9): p. 2302-2307.

[6] Krauthammer, T., Blast-resistant structural concrete and steel connections. International Journal of Impact Engineering, 1999. 22(9): p. 887-910.

[7] Lee, C.H., et al., Simplified nonlinear progressive collapse analysis of welded steel

moment frames. Journal of Constructional Steel Research, 2009. 65(5): p. 1130-1137.

[8] Bao, Y., et al., Macromodel-based simulation of progressive collapse: RC frame structures. Journal of Structural Engineering, 2008. 134(7): p. 1079-1091.

[9] Kripakov, N.P., M.C. Sun, and D.A. Donato, ADINA applied toward simulation of progressive failure in underground mine structures. Computers & Structures, 1995. 56(2): p. 329-344.

[10] Brunesi, E. and R. Nascimbene, Extreme response of reinforced concrete buildings through fiber force-based finite element analysis. Engineering Structures, 2014. 69(Supplement C): p. 206-215.

[11] Yang, B. and K.H. Tan, Experimental tests of different types of bolted steel beam–column joints under a central-column-removal scenario. Engineering Structures, 2013. 54: p. 112-130.

[12] Lew, H.S., et al., Performance of steel moment connections under a column removal scenario. I: Experiments. Journal of Structural Engineering, 2012. 139(1): p. 98-107.

[13] Tsitos, A., et al. Experimental investigation of progressive collapse of steel frames under multi-hazard extreme loading. in The 14th World Conference on Earthquake Engineering. 2008

[14] Yi Weijian, HE Qingfeng and XIAO Yan, Collapse performance of RC frame structure. Joumal of Building structures, 2007. 28(5): p. 104-109. (in Chinese)

[15] Sagiroglu, S., Analytical and experimental evaluation of progressive collapse resistance of reinforced concrete structures, in Department of Civil and Environmental Engineering. 2012, Northeastern University.

[16] XIE Fuzhe and SHU Ganpin, Quasi-static experimental research on progressive collapse of space steel frames. Journal of PLA University of Science and Technology (Natural Science Edition), 2013. 14(2): p. 195-201. (in Chinese)

[17] XIE Fuzhe, Analysis and assessment and experimental research on progressive collapse of steel frame structure [D]. Nanjing: Southeast University, 2012. (in Chinese)

[18] Chen, J., et al., Experimental Study on the Progressive Collapse Resistance of a Two-Story Steel Moment Frame. Journal of Performance of Constructed Facilities, 2012. 26(5): p. 567-575.

[19] Xiao, Y., et al. Collapse test of a 3-story half-scale RC frame structure. in Structures Congress, ASCE, Reston, VA. 2013.

[20] Li, Fengwu, et.al., Experimental and analytical study on progressive collapse of RC frame with sudden side columns removal. CHINA CIVIL ENGINEERING JOURNAL, 2014. 04(04): p. 9-18. (in Chinese)

[21] Sasani, M. and S. Sagiroglu, Progressive Collapse Resistance of Hotel San Diego. Journal of Structural Engineering, 2008. 134(3): p. 478-488.

[22] Sasani, M., et al., Progressive Collapse Resistance of an Actual 11-Story Structure Subjected to Severe Initial Damage. Journal of Structural Engineering, 2011. 137(9): p. 893-902.

[23] Defense, U.S.D.o., Unified Facilities Criteria (UFC) 4-023-03:Design of Buildings to Resist Progressive Collapse 2013, Department of Defense: Washington, D.C.

[24] Administration(GSA), U.S.G.S., Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects. 2003, General

Services Administration, Office of Chief Architect.

[25] Izzuddin, B.A., et al., Progressive collapse of multi-storey buildings due to sudden column loss — Part I: Simplified assessment framework. Engineering Structures, 2008. 30(5): p. 1308-1318.

[26] GB/T228-2002, Metallic materials: tensile testing at ambient temperature. 2002, Standards Press of China: Beijing.

[27] Russell, J., J. Owen, and I. Hajirasouliha, Experimental investigation on the dynamic response of RC flat slabs after a sudden column loss. Engineering Structures, 2015. 99: p. 28-41.

[28] Khandelwal, K. and S. El-Tawil, Pushdown resistance as a measure of robustness in progressive collapse analysis. Engineering Structures, 2011. 33(9): p. 2653-2661.

[29] Li, L.-L. and G.-Q. Li, The internal force relationship of rectangular and I-section for bi-linear hardening material with limit strain. International Journal of Steel Structures, 2016. 16(1): p. 243-255.

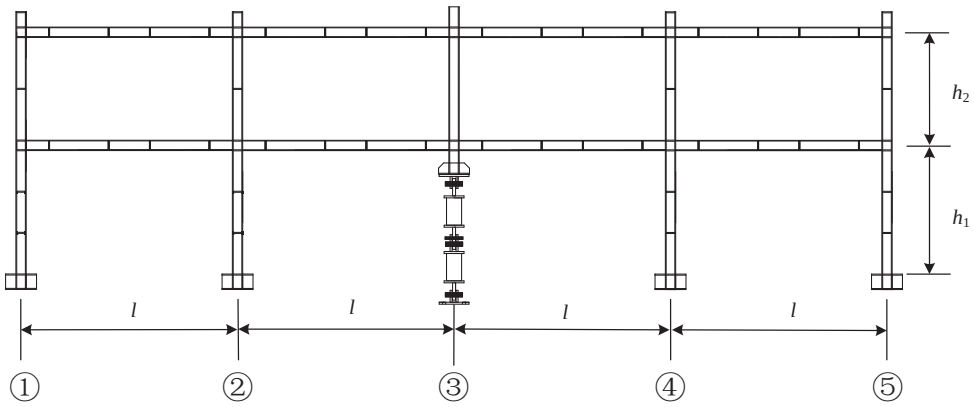
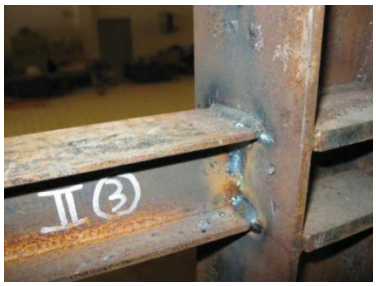


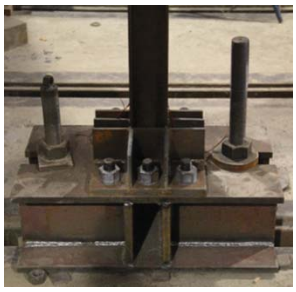
Fig.1 Layout of the steel frame specimen



a. FRAME1

b. FRAME2

c. FRAME3



d. Column base fixture

Fig. 2 Beam-column connections in different frames and column base fixture

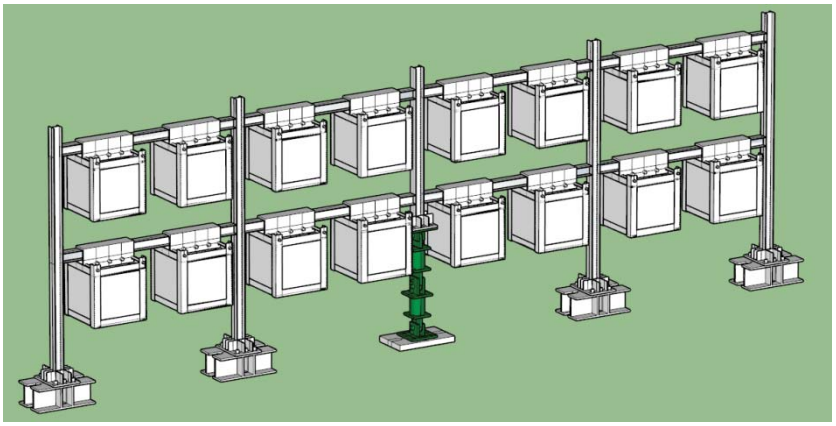


Fig. 3 Schematic diagram of test steel frame

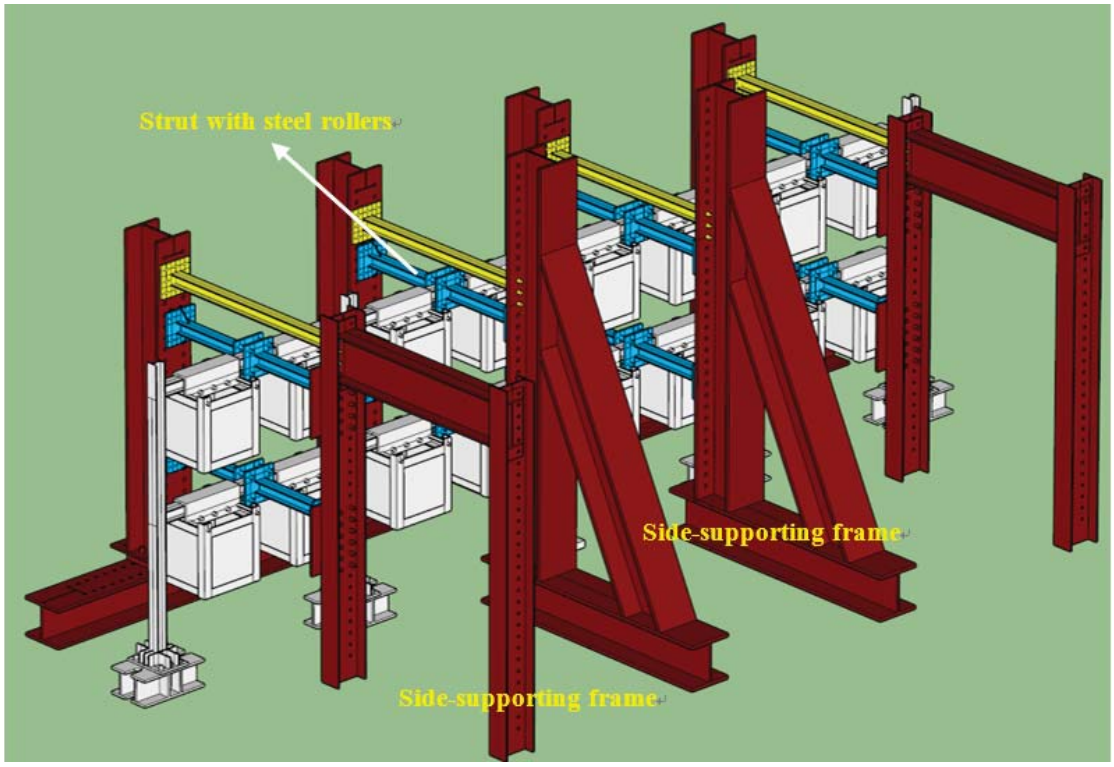


Fig.4 Schematic of out-of-plane supporting setup

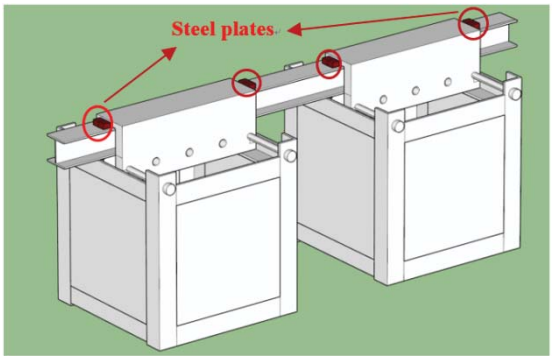


Fig.5 Schematic diagram of attachment of weight-holding baskets

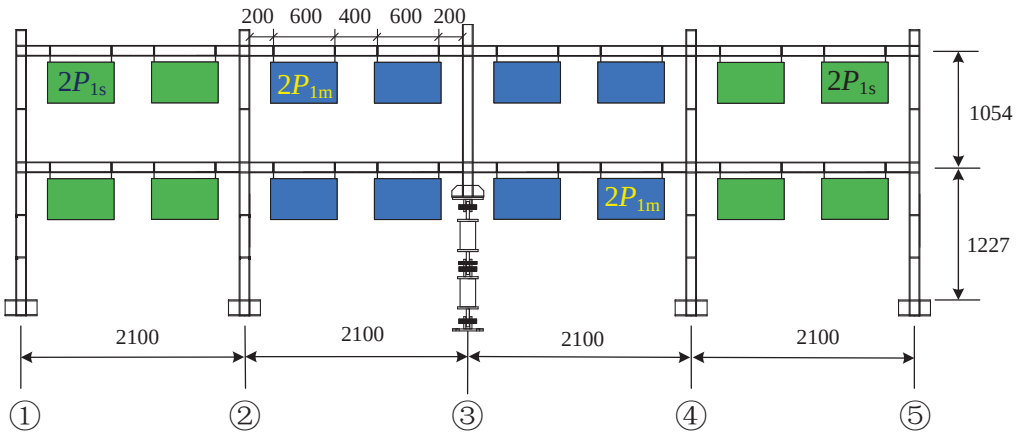


Fig.6 Illustration of distribution of loads (unit: mm).

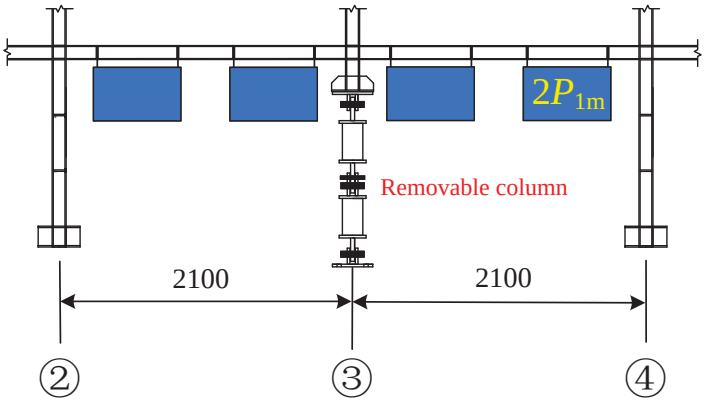
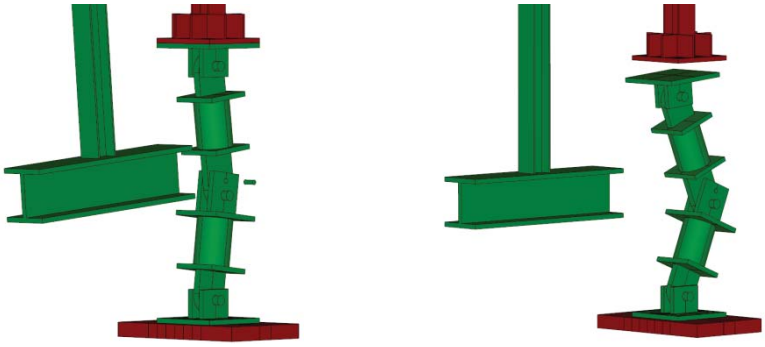


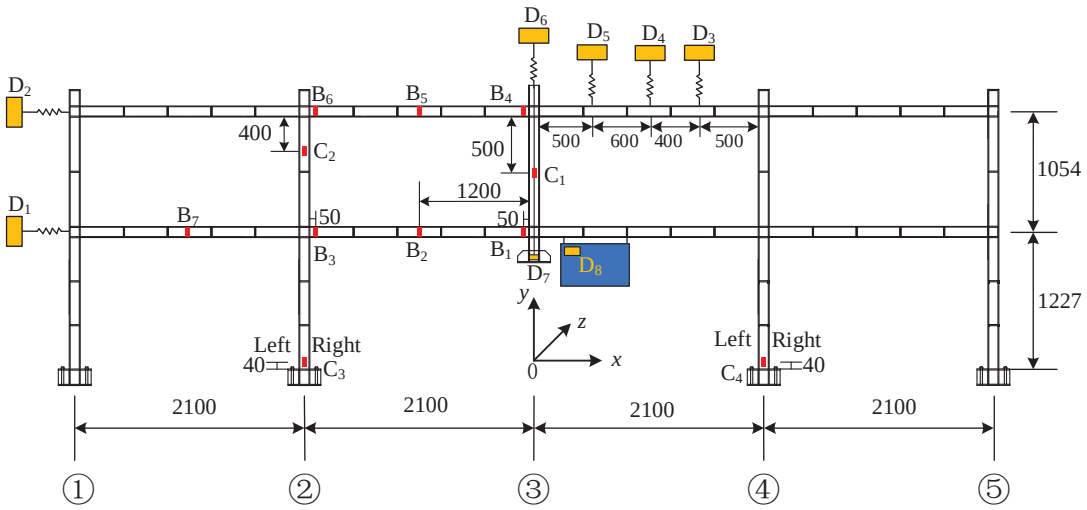
Fig.7 Removal column mechanism allowing quick removal by a pendulum hammer

Fig. 8 Individual parts and assembled removal column

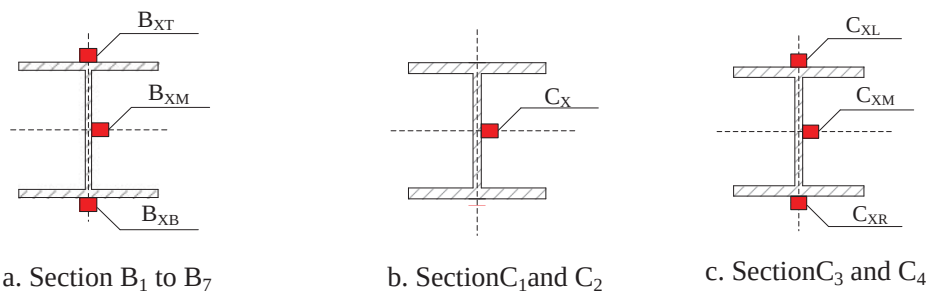


a. Breaking of the glass rod b. Activation of the hinged mechanism

Fig. 9 Schematic of “column” removal process with a pendulum hammer



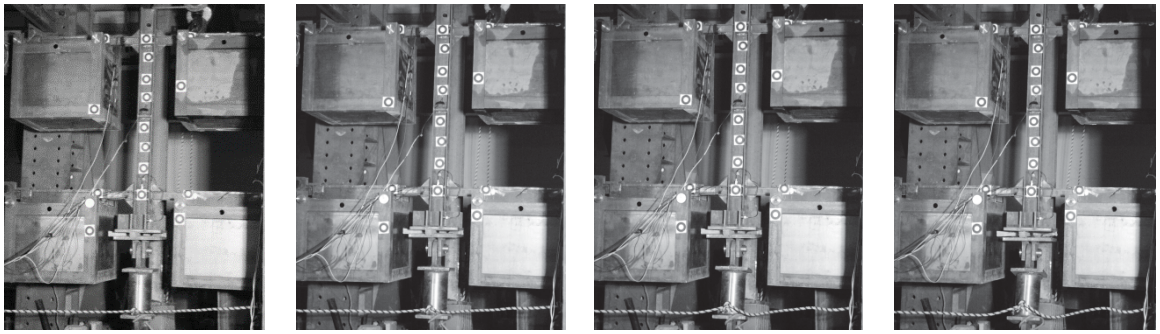
a. Arrangements of strain gauges and displacement transducers (unit: mm).



a. Section B_1 to B_7 b. Section$C_1$and C_2 c. SectionC_3 and C_4

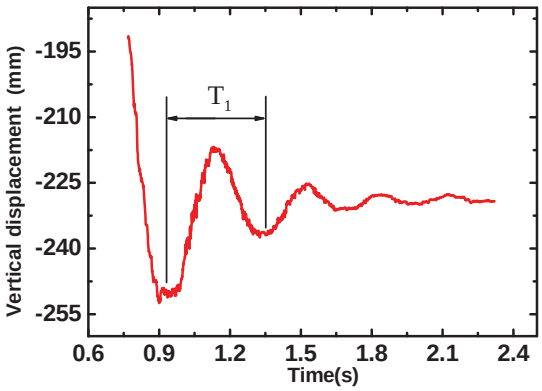
b. Detailed locations of strain gauges at the section level

Fig. 10 Locations of strain and displacement measuring points (dimension unit: mm).

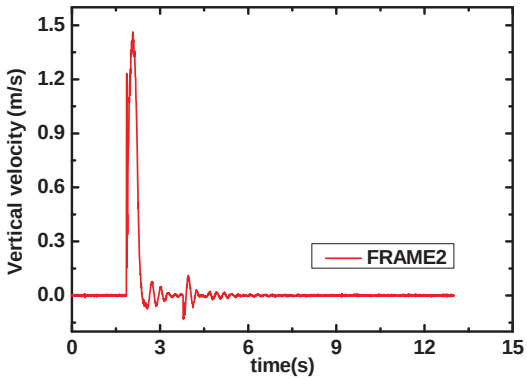


a. t = 0s（impending strike of pendulum onto removal column）　b. t = 1/149s (removable column still engaged with the steel frame)　c. t = 3/149s（disengaging of removable column）　d. t = 5/149s（collapse of removable column）

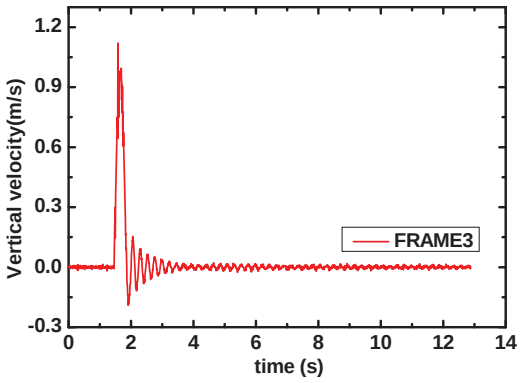
Fig.11 The removing process of removable column



a. Close-in of the displacement-time history measured at point D_6 (Frame 3)



a. FRAME2



b. FRAME3

b. Vertical velocity time histories at measuring point D_7

Fig. 12 Typical measured displacement / velocity time histories from the test frames

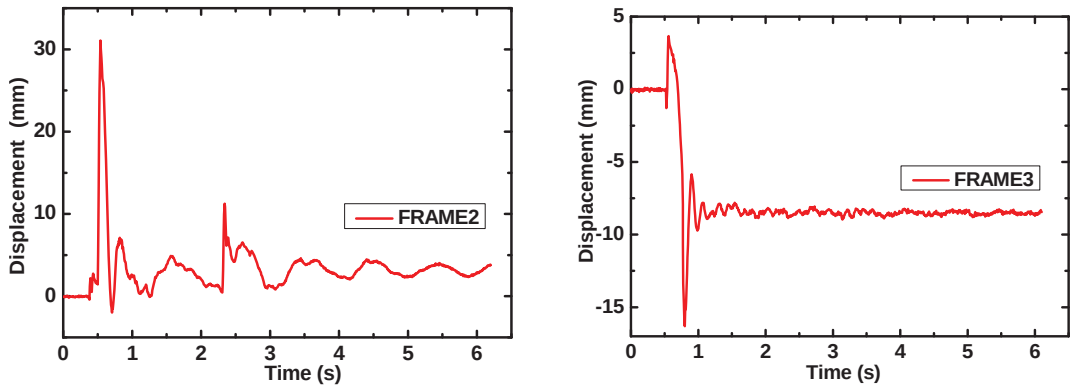
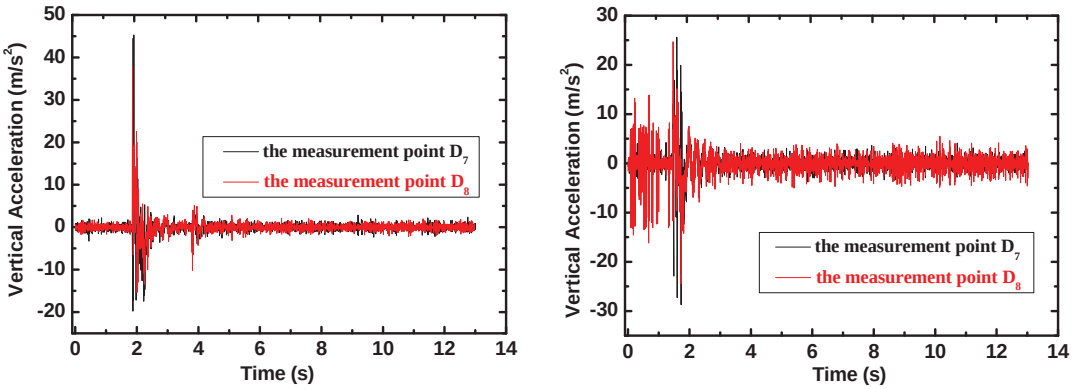


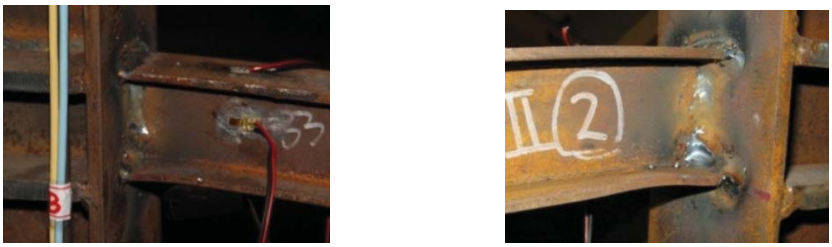
Fig.13 Out-of-plane displacement time histories of measuring point D_7



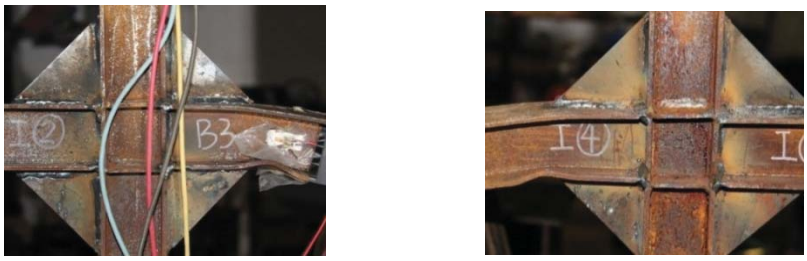
a. FRAME2

b. b. FRAME3

Fig.14 Vertical acceleration time histories of measuring points D_7 and D_8



(a) FRAME1, at connection I② and II②



(b) FRAME2, at connection I② and I④

(c) FRAME3, at connection I② and I④

Fig. 15 Local buckling of beams near beam-to-column connection in the three test frames

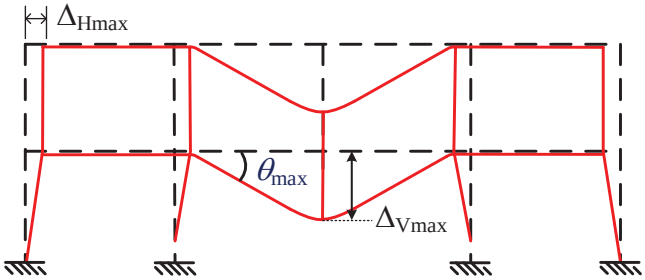
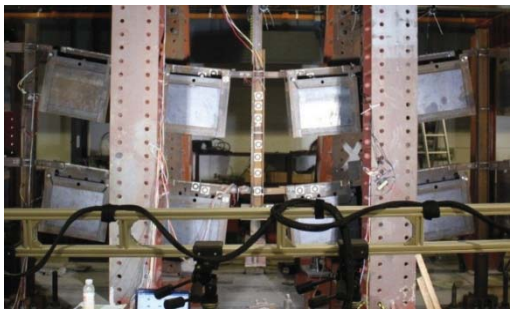


Fig. 16 Schematic diagram of displacement and rotation for the steel frame



a. FRAME1

b. FRAME3

c. FRAME2 (middle and right photos show damages of columns)

Fig. 17 Permanent deformation profiles of test frames

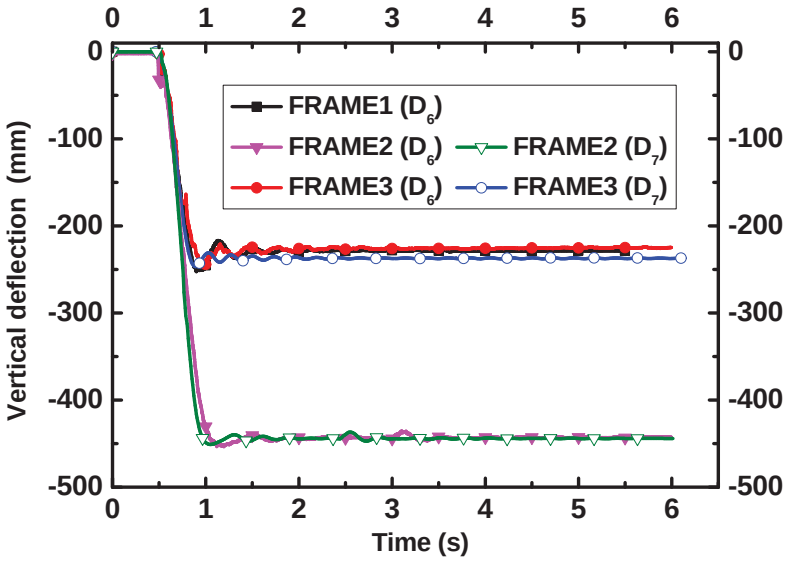


Fig. 18 Vertical deflection time histories (at Points D_6 and D_7 as shown in Fig.10)

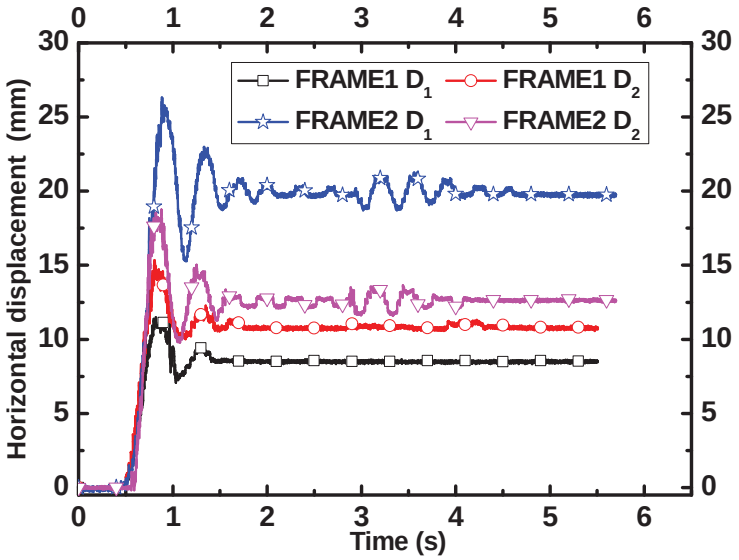
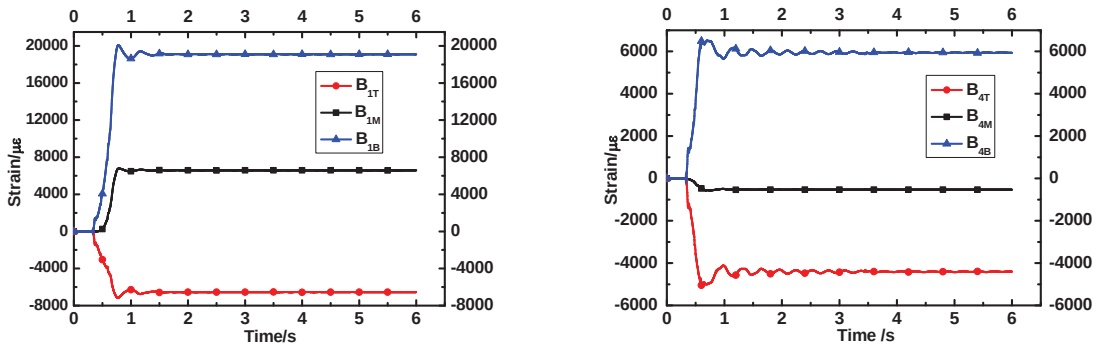
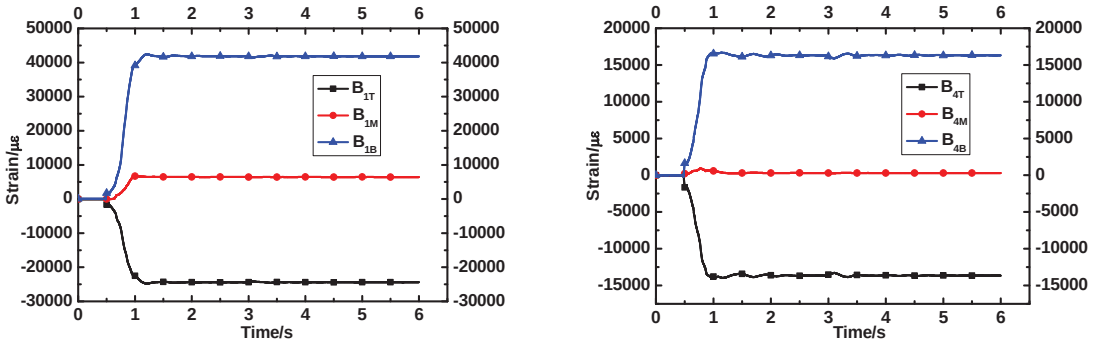


Fig. 19 Horizontal displacement time histories at side joints (points D_1 and D_2)



a. FRAME1



b. FRAME2

Fig. 20 Strain time histories of beam sections after removal of the middle column

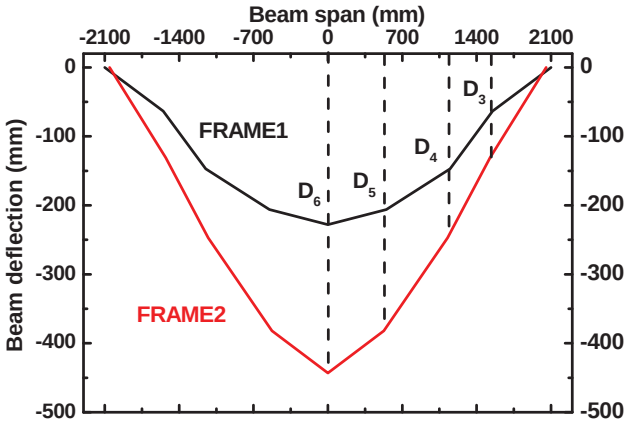
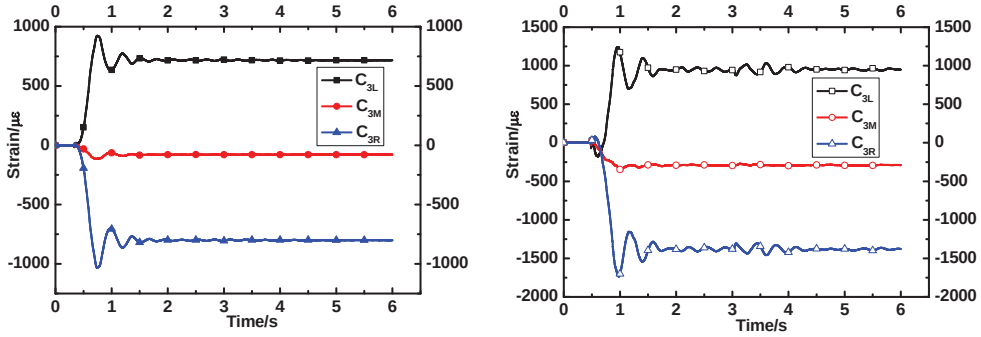


Fig. 21 Final displacement profiles of beams after removal of the middle column



(a) FRAME1 (b) FRAME2

Fig. 22 Strain time histories at column section C_3 after the column removal

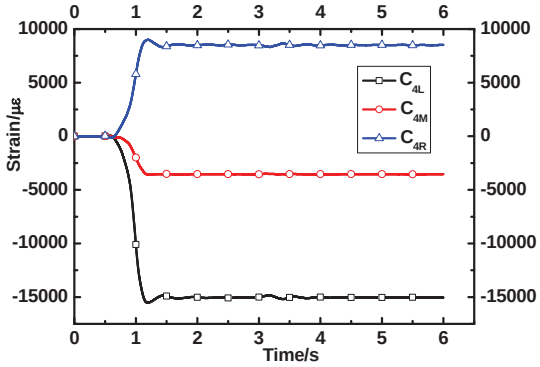


Fig. 23 Strain time histories at column section C_4 of FRAME2