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Progressive collapse behaviour of steel framed substructures with various beam-column connections

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Abstract: This paper presents experimental efforts in steel framed substructure with bolted flange plate (BFP) connections subjected to progressive collapse. A simplified modelling approach based on the shell element and ductile fracture criterion is introduced. The method shows a close correlation to the existing experimental data and demonstrates better computing efficiency than solid element based simulation strategy. A number of parametric studies were conducted based on the validated numerical method to evaluate the influence of varying connection configurations on the progressive collapse resistance of the steel substructures. It is indicated that the connection with welded flange plates can lead to greater flexural strength than that with bolted angles and the application of welded haunch plates can arrest fracture failure on the welds within the beam-column joint. Predictions are made for fracture potential of different connections in resisting progressive collapse.

Keywords: Progressive Collapse; Steel Frame; Experimental Analysis; Numerical Simulation; Beam-column Flange Plate Connection.

1. Introduction

Progressive collapse design aims to maintain structural robustness and avoid the continuous collapse of structures under blast, fire, impact or other localized damages. Previous researches on the issue of progressive collapse were started with reinforced concrete (RC) structures [1-4] and then extended to steel structures, efforts have been made to investigate the effect of compressive arch action (CAA) and

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catenary action (CA) on the structural resistance, aimed to develop an analytical, experimental or numerical model for predicting the progressive collapse resistance of each specimen [5-8]. Specific design requirements, such as ASCE-7 [9], DoD [10] and GSA [11], were proposed to cope with possible progressive collapse events in structural buildings.

For steel frames, previous studies have shown that the progressive collapse response of the structure was mainly depended on the ability of the connections between the main structural elements [12]. And a series of studies have been conducted aiming to reveal the effect of connection configurations on the progressive collapse performance of the steel-framed structures. For shear connections, Weigand and Berman [13, 14] described the response of single plate and bolted angle connections subjected to simultaneous rotation and axial extension. Oosterhof and Driver [15] found that the failure of shear-tab specimens under column removal scenario was governed by bolt tear-out failure mode, and the failure of bolted-bolted angle connections subjected to progressive collapse was controlled by tearing of the section near the angle-heel. Moreover, it was concluded by Daneshvar and Driver [16] that the contribution of bolt hole bearing deformation is significant to the ductility of the connection for WT shear connections under progressive collapse.

For moment connections, Lew et al. [17] presented an experimental study of welded unreinforced flange bolted web connection and a reduced beam section welded (RBS) connection under progressive collapse scenario. It was revealed that the latter has a better rotation capacity when compares with the former, and the failure of the RBS specimen was characterized by the fracture of the bottom flange in the reduced section. Similarly, Dinu [18] investigated the performances of four types of beam-to-column connections, namely, welded cover plate flange connection (CWP), haunch end plate bolted connection (EPH), RBS connection, and unstiffened extended end plate bolted connection (EP), against progressive collapse. The CWP, EPH, and RBS specimens showed good ductility, with the CA made a significant contribution to the ultimate load resistance. Specimen EP demonstrated the lowest ductility and ultimate load resistance, with the bolts in the rows under tension fracture before the CA was developed. Qin et al. [19] and Wang et al. [6] investigated beam-to-SHS column connections under a middle column loss scenario and proposed a reinforcing technique by using bolted cover plates to develop full plastic strength of the beams under the large deformation phase and take full advantage of CA produced by axial tension. Wang et al. [20] studied the progressive collapse performance of steel

beam to concrete-filled steel tubular (CFST) column connections and concluded that the frame structure with these joints was enabled to form the resistance mechanism and can prevent the occurrence of progressive collapse after the failure of column connected to joints. Zhong et al. [21] studied the anti-progressive collapse performance of welded unreinforced flange-bolted web connection (WUF), top-seat angle with double web angle connection (TSDWA), and double web angle connection (DWA) of a steel frame. They found that the resistance of the DWA specimen was mainly provided by CA and flexural action contributed more significant effect to resistance in the other two specimens. The axial force in a TSDWA-loaded beam could be fully developed, which exhibited a greater anti-collapse bearing capacity for reserve strength.

It can be summarized that the configuration of beam-column connections has a significant effect on the progressive collapse performance of steel structures. However, the understanding of welded-bolted flange plate connections is currently limited by a deficiency of physical test data. To this end, steel beam-column sub-assemblages with bolted flange plates (BFP) connections were designed and tested under monotonically increased vertical displacement atop the unsupported centre column. In the test, structural resistance and failure mechanism for each specimen were introduced. In addition, finite element (FE) simulations were presented and validated with existing test outcomes. More sub-frames with flange plate connections were investigated based on the numerical method. Behaviours of each beam-column subassembly against progressive collapse, including load-carrying capacity, failure mode and rotation capacity, were emphasized.

2. Overview of Laboratory Tests

2.1 Specimen Design and Test Setup

In the test, two identical 1/3 scale 2-bay 2-storey steel framed substructures, which represent part of an actual office building prototype, were prepared (as shown in **Fig. 1**). It is visible that the lower middle column of the specimen was removed to accommodate the initiation of progressive collapse. The floor height and the span of the frame were 1 m and 2 m respectively. An H200×100×5.5×8 mm section was applied to the columns and the beams had an H100×100×6×8 mm section. All steel components used in the test have nominal yield strength of 235 MPa and tensile strength of 440 MPa, respectively, and the material property was measured by the specimen supplier. Monotonically increasing displacement was

imposed atop the unsupported centre column by using a screw jack. The load was applied under displacement control mode at a rate of 2 mm/min and the load data was recorded via a load cell placed on the top surface of the central column. Horizontal restraints were provided by a reaction system on both sides of the sub-frame. The out-of-plane movement of the central column was restrained by a pin support. Side columns of the specimen were supported by short connection beams (HN 200x100x5.5x8 mm) with 12 M16 bolts in grade-8.8 (with a yield strength of 640 MPa and without pretension). The displacement of the specimen was measured by 14 LVDTs and strain gauges were mounted on critical sections to monitor the strain distribution on each steel component.

Configuration of the beam-column connections is plotted in **Fig. 2**. The beam-column BFP connections used in the test were designed based on UFC 4-023-03 guideline [10]. In the connection, shear tab and flange plates were connected to the column flange by fillet weld and bolted to the beam using 8.8-grade M16 bolts.

(a) Test setup

(b) Photograph of the specimen-1.

Fig. 1: Elevation view of the test setup.

(a) Photograph of the connection

(b) BFP connection details (unit in mm)

Fig. 2: Photograph and details of connections.

2.2 Summary of the Experimental Results

Development of the applied load against middle joint displacement (MJD) of the specimens is shown in **Fig. 3**. In the figure, F_p represents the vertical capacity when full plastic yield moment of the beam is achieved; and rotation is calculated by MJD over beam span length. It can be found that the curves are linearly ascending in the early stage. As yield load of the beam was exceeded (point A/A'), cracks were observed on the top flange plate of the west lower beam (**Fig. 5 (a)**) and degradation of the frame stiffness was found. When plastic hinges formed in the beam ($F/F_p=1.0$), more cracks were spotted on the bottom flange plate of the central joints and top flange plate of the side joints and the tilt of the side columns was changed from outward to inward (see **Fig. 4**), indicating compressive arch action (CAA) was arrested and CA was initiated (point B/B'). Under the CA stage, the existing cracks continued expanding as load increasing (**Fig. 5 (b)**). When the ultimate load reached (point C/C'), a severe fracture was happening in sudden on the weld at the top beam flange of the side joint on the east upper beam (**Fig. 5 (c)**). Then, the resistance of the frame dropped sharply as the east upper beam totally detached from the side column, and the test was terminated (point D/D') to prevent any possible hazards. It implies in **Table 1** that the CA can be mobilized in both specimens when the rotation of the beam reaches 0.03 rad and the rotation is 0.1 rad when the ultimate resistance achieves. The rotation capacities captured in the test reflect that the acceptance criterion of rotation capacity at collapse prevention (CP) stage for BFP connection (0.015 rad) in the ASCE 41[22] is too conservative. The resistance of the specimen was illustrated in **Fig. 6**, which based on the strain gauges attached on the

frame. It can be seen that the axial force was insignificant in the early stage and the resistance of the frame was mainly offered by flexural action of the beam. When the first peak load was achieved, the flexural resistance experienced a decrease as compression mobilized in the beam under the CAA. After the CAA stage, the axial tensile force of the beam gradually increased while the bending moment decreased, indicating the load-resistance mechanism of the frame was changed from “beam mechanism” to “catenary action”. Meanwhile, the resistance provided by CA became predominant.

Fig. 3: Applied force versus MJD.

Fig. 4: Plot of horizontal displacement of the west side column in specimen 1.

Fig. 5: Fracture mode on the east upper beam of specimen-1.

Fig. 6: Plot of resistance development for specimen-1.

Table 1: Summary of test results.

Description	Specimen-1		Specimen-2		Observations
	Load (kN)	Rotation (rad)	Load (kN)	Rotation (rad)	
Elastic limit (A/A')	34.1	0.01	27.0	0.01	Limited flexural cracks observed on the flange plate
Start of CA (B/B')	45.6	0.03	45.4	0.03	More cracks are emerged
Max. load-carrying capacity (C/C')	108.6	0.10	104.2	0.10	Fracture was occurring on the weld
End of the test (D/D')	76.2	0.14	92.9	0.13	The east beam was separated from the side column

2.3 Load resistance mechanism

Based on the test observations and recorded data, it is logical to divide the resistance of the sub-frame with flange plates into two stages (**Fig. 7**): resistance when catenary action initiated (F_c) and ultimate resistance (F_u). When plastic hinges formed in the beam (**Fig. 8**), the CA initiated and the resistance can be achieved according to the plastic bending moment of the section (**Eq. 1**); in the catenary stage, the failure behaviour of the frame was controlled by the following two scenarios: 1) tension fracture of the

net section of flange plates or shear tab; 2) cleavage fracture of the welds (Eq. 2). The according displacement of the specimen can be predicted based on the rotation of the beam (Eqs. 3 and 4).

Fig. 7: Plot of the analytical load-MJD curve.

Fig. 8: Plot of load resistance in the beam mechanism.

$$F_c = \sum \frac{4M_p}{L} \quad (\text{Eq. 1})$$

$$F_u = \min(\sum f_y(A_{st} + A_{fl}), \sum f_{y,w}(A_{st,w} + A_{fl,w})) \quad (\text{Eq. 2})$$

$$\Delta_c = L \times \tan(\theta_c) \quad (\text{Eq. 3})$$

$$\Delta_u = L \times \tan(\theta_u) \quad (\text{Eq. 4})$$

Where, M_p is the plastic moment of the beam (kN-m); L is the effective length of the span (m); F_c and F_u are the applied load when CA started and ultimate load achieved (kN); f_y and $f_{y,w}$ are the yield strength of the steel and the welds (MPa); A_{st} and A_{fl} are the net cross-sectional areas of the shear tab and flange plate (mm^2); $A_{st,w}$ and $A_{fl,w}$ are cross-sectional area of the shear tab to column welds and flange plate to column welds (mm^2); Δ_c and Δ_u are the deflection of the beam when CA initiated and ultimate load reached (mm); θ_c and θ_u are the rotation of the beam when CA initiated and ultimate load achieved (rad).

3. Numerical simulation

3.1 Finite element models

As conventional solid modelling strategies [23] could be time-consuming, a number of simplified simulation approaches have been prompted to carry out progressive collapse analysis. Khandelwal and El-Tawil [24] used calibrated micromechanical fracture models to investigate a number of key design variables influencing the formation of CA in steel special moment-resisting frame subassemblies. Lee et al. [25] developed a computationally efficient macro-model for practical progressive collapse analysis of welded steel moment frames. Sadek [26] presented a reduced model represents the response of steel beam-column assemblies with moment connections under column removal scenario. Zhang et al. [27] proposed a multi-spring component (MSC) model for assessing the rotational stiffness of semi-rigid beam-to-column connections. Li et al. [28] presented a shell element based finite element (FE) modelling study on the progressive collapse of steel frames under a sudden column removal.

After comparing the existing modelling approaches, the structural performance of the specimens subjected to a downward load was predicted using ABAQUS package [29] with shell element based models in the current study. This analysis approach requires limited computation resources [30] and can accommodate crack initiation and avoid convergence problems in the analysis [31]. In the model, steel components and welds were modelled by 3D shell element S4R, respectively. An overall view of the frame model is plotted in **Fig. 9**. Note that the model geometry (dimensions, cross-sections and connection details), support conditions were in full agreement with the test. Symmetry was maintained during the simulation for saving the CPU time [26]. A basic mesh size of 50 mm was applied to the steel component and a refined mesh of 6 mm was applied to the elements within the connection. Extensive trial-outs indicated that the adopted mesh is capable of representing the structural behaviour of the model. The load was applied via a reference point which coupled with the top surface of the central column. The central column was supported by symmetric restraints and the side column was supported by lateral restraints, while pin supports were used to prevent all translations at the column end. Displacement controlled dynamic explicit analysis was adopted in the model. “Tie” contacts were used to simulate the interaction between the welds and beam/column/plate interface. This contact strategy can connect two separate surfaces together and arrest relative motion between them. A “point-to-surface” coupling interaction was developed for the bolt connections between the flange plate/shear tab and the beam. In this contact, bolts were defined as master point and bolt-holes were assumed as slave surface, the motion of the slave surface was constrained by the motion of the master point. This

simplification is reasonable because the resistance of the bolt is greater than that of the plate and no bolt shank failure was observed during the test.

For steel material, a bilinear stress-strain curve was adopted in the model, and the values were the same as the tests (**Table 2**). In order to simulate the fracture mode, the failure criterion applied in the model was characterized by the calibrated model presented by Yu and Jeong [32], in which the fracture locus was expressed as:

$$\bar{\epsilon}_0^{pl} = \begin{cases} \infty, & (-1/3 \leq \eta) \\ C_1/(1 + 3\eta), & (-1/3 \leq \eta \leq 0) \\ C_1 + (C_2 - C_1)(\eta/\eta_0)^2, & (0 \leq \eta \leq \eta_0) \\ C_2 \eta/\eta_0, & (\eta_0 \leq \eta) \end{cases} \quad (\text{Eq. 5})$$

where $\bar{\epsilon}_0^{pl}$ is fracture initiation strain; η is stress triaxiality; C_1 is $\bar{\epsilon}_0^{pl}$ in pure shear ($\eta = 0$) and C_2 is $\bar{\epsilon}_0^{pl}$ in uniaxial tension ($\eta = \eta_0$). C_2 can be decided by the reduction in area A_R of a uniaxial, axisymmetric tensile specimen as:

$$C_2 = -\ln(1 - A_R) \quad (\text{Eq. 6})$$

and C_1 can be calculated as:

$$C_1 = C_2 \left(\frac{\sqrt{3}}{2} \right)^{1/m} \quad (\text{Eq. 7})$$

$$\sigma = K_p \epsilon^m \quad (\text{Eq. 8})$$

where m is the hardening exponent of a power law for isotropic strain hardening; K_p is the strength coefficient. In the study, $C_2=0.63$, $m=0.18$, $K_p=700$ MPa and $\eta_0=1/3$ were applied, the values were decided according to the recommendations proposed by [33-35]. Ductile damage constitutive model for the steel material and welds was calculated according to **Eqs. 5-8**; the achieved curve is depicted in **Fig. 10**. In the model, it was assumed that the damage of the shell element starts when the specified equivalent plastic strain at the onset of damage was attained. The damage evolution, which is characterised by the progressive degradation of the material stiffness, was also defined. Once the fracture strain is approached, the damaged elements will be deleted from the model.

Fig. 9: The view of the FE model for the specimen with BFP connections.

Fig. 10: Fracture properties adopted in the model.

Table 2: List of steel properties.

Components	Elastic modulus E (GPa)	Yield strength (MPa)	Tensile strength (MPa)
Steel components (BFP)	2.06	235	440
Column flange	2.10	262	395
Column web	1.98	293	405
Beam flange	1.95	266	397
Beam web	1.90	312	422
Shear tab	1.94	270	408
Angle (L70)	2.09	268	405
Angle (L140)	2.01	265	402

Note: Material properties of the BFP specimen are provided by the supplier, and properties of other specimens are according to [21].

3.2 Model verification

In the study, laboratory test data of the sub-frame specimens with BFP, WUF, TSDWA and DWA (**Fig. 11**) connections were used to verify the applicability of numerical modelling approach. The last three

connections were tested by Zhong et al. [21], in their test, beam-column assemblages with aforementioned connections was installed vertically and ends of the side columns were connected with horizontal hinges to achieve the hinge constraints. A displacement controlled load was applied on the top of the unsupported column to simulate the progressive collapse condition. In this exercise, the load versus MJD response, failure behaviour, and rotation represented by the model are presented and compared against test results. As shown in **Fig. 12**, a good agreement of load against MJD curve can be observed between the FEM and experimental results. The CA can be developed for all specimens through the connections following plastic hinge formation. In **Fig. 13**, it is visible that the FE models capture the observed fracture well. For the specimen with BFP and WUF connections, the failure was controlled by the tensile failure of the flange and shear tab; for the specimen with DWA and TSDWA connections, the failure is governed by the tear-out of the angles net section. Bolted-bolted angle connections (DWA and TSDWA) demonstrate greater rotation capacity under CA than welded connections, due to the unfolding ability of the angle section. The TSDWA specimen has greater load-carrying resistance when compares with the DWA specimen because of the additional tensile strength provides by the top and bottom seated angles. Comparisons of the ultimate load and failure mode between the specimens are tabulated in **Table 3**. It is visible that the proposed analytical method and FEM can get a close agreement against test results. The result implies that the acceptance criterion of rotation capacity in ASCE 41 is too conservative in predicting the rotation of all specimens, and the average ratios are 4.47 and 6.23 between the rotations of test specimens and the codified values at LS and CP stage.

(a) WUF specimen

(b) TSDWA specimen

(c) DWA specimen

Fig. 11: Plot of test specimens [21].

(a) BFP specimen

(b) WUF specimen

(c) TSDWA specimen

(d) DWA specimen

Fig. 12: Experimental vs. numerical applied load versus MJD curves.

(a) BFP specimen (test)

(b) BFP specimen (FEM)

(c) WUF specimen (test)

(d) WUF specimen (FEM)

(e) TSDWA specimen (test)

(f) TSDWA specimen (FEM)

Fig. 13: Comparison of failure modes.

Table 3: Comparison of ultimate load and failure mode.

Specimen	$P_{c, Test}$ (kN)	$\Theta_{c, Test}$ (rad)	$P_{c, ANA}$ (kN)	$\Theta_{c, ASCE}$ (rad)	$P_{c, FEM}$ (kN)	$\Theta_{c, Test}/\Theta_{c, ASCE}$	$P_{c, ANA}/P_{c, Test}$	$P_{c, FEM}/P_{c, Test}$
BFP	79.0	0.07	74.8	0.02	70	3.5	0.95	0.89
WUF	60.0	0.08	61.5	0.03	67	2.7	1.02	1.12
TSDWA	68.0	0.1	61.5	0.01	66	10.0	0.90	0.97
DWA	63.0	0.19	61.5	0.11	64	1.7	0.98	1.02
Average						4.47	0.96	1.00
S. D.							0.05	0.10

Specimen	$P_{u, Test}$ (kN)	$\Theta_{u, Test}$ (rad)	$P_{u, ANA}$ (kN)	$\Theta_{u, ASCE}$ (rad)	$P_{u, FEM}$ (kN)	$\Theta_{u, Test}/\Theta_{u, ASCE}$	$P_{u, ANA}/P_{u, Test}$	$P_{u, FEM}/P_{u, Test}$
BFP	109.3	0.12	112.1	0.03	110.7	4.00	1.03	1.02
WUF	108	0.13	109	0.04	112.2	3.10	1.01	1.04
TSDWA	172	0.24	162.3	0.02	164.4	16.00	0.94	0.96
DWA	92	0.28	97.7	0.14	94.9	1.80	1.06	1.03
Average						6.23	1.01	1.01
S. D.							0.05	0.04

Note: $P_{c, Test}$, $P_{c, ANA}$ and $P_{c, FEM}$ represent the applied load in the test, analytical and FEM when plastic hinges formed and catenary stage initiated in the beam; $P_{u, Test}$, $P_{u, ANA}$ and $P_{u, FEM}$ represent the ultimate CA capacity of the specimen from the test, analytical and FEM; $\Theta_{c, Test}$ and $\Theta_{u, Test}$ represent the experimental rotation capacity of the specimen when the catenary stage initiated and ultimate capacity reached; $\Theta_{c, ASCE}$ and $\Theta_{u, ASCE}$ represent the plastic rotation of the specimen under life safety (LS) and CP stage according to ASCE 41[22].

4. Parametric study

An extensive parametric study was performed in order to investigate the effect of key design variables on the resistance and rotation capacity of the specimen under the column removal scenario. Apart from the aforementioned BFP and DWA connections, more modified connections from UFC 4-023-03 [11] were studied, namely welded flange plate double web angle connection (WFPDWA), welded Top and Bottom Haunches (WTBH) and welded Top Plate and Bottom Haunches (WTPBH). Configuration of each connection is demonstrated in Fig. 14. It needs to be noted that FE models in the parametric study were established based on the aforementioned geometric dimension, modelling approach and the bolt

shank shear fracture failure was not involved in all cases.

Fig. 14: Plot of specimens with various dimensions.

4.1 Effect of the weld throat thickness

For the BPF specimen, the thickness of the fillet weld throat plays a critical role in structural progressive collapse resistance as fracture first initiates at the place. **Fig. 15** quantifies the effect of weld thickness on the behaviour of the frame. As expected, in the beam mechanism, the resistance of the subassembly was controlled by the plastic bending moment of the beam and the effect of weld thickness is limited. While a substantial influence of the weld thickness is visible for the ultimate load-carrying capacity and ductility of the frame. For example, when the thickness increases from 6 mm to 7 mm and from 7 mm to 8 mm, the load capacity and failure displacement increase by 26.3% and 34.6%, respectively, indicating the catenary stage is not fully developed when weld thickness is too small. When the thickness of the welds exceeds the beam flange plate thickness (8 mm), the fracture will occur on the shear tab or beam flange plate and the effect of the weld thickness becomes negligible.

Fig. 15: Comparison of varying weld thickness for the BFP specimen.

4.2 Effect of the flange plate

In previous studies, it indicates that the DWA specimen can offer greater ductility under large deformation than the WUF specimen but with less load-carrying capacity in the CAA stage. Therefore, the WFPDWA specimen was designed and compared with the WUF and DWA specimen, as depicted in **Fig. 14 (c)**. The connection is evolved from the DWA specimen with welded top and bottom cover plates. The dimension of the WFPDWA specimen studied herein is the same to the DWA specimen but with two cover flange plates of 110 x 70 x 8 mm (width x length x thickness) welded to the beam and column flange. **Fig. 16** plots the comparison of applied load against MJD curves between the WUF, DWA and WFPDWA specimen. It can be seen that the difference of the MJD is insignificant between

the DWA and WFPDWA specimen when the ultimate load achieves, nevertheless the enhancement of the CA ultimate resistance introduced by the WFPDWA specimen is significant (by 35.1%). Moreover, the connection that fails by beam flange rupture (WUF specimen) is less ductile than that fails in the angle unfolding shear (WFPDWA specimen), with the MJD increase by 67.5% and 113.6%, when the first peak load and ultimate capacity are reached.

For the specimen with WFPDWA connections, the effect of flange plate dimension on the load-carrying capacity and rotation is studied. From **Fig. 17 (a)**, it indicates that the load-carrying capacity of the WFPDWA specimen is sensitive to the flange plate thickness. As increasing the flange plate thickness from 6 to 8 mm, the first peak load, CA initiated load and the ultimate CA capacity increase by 48.2%, 10.6% and 24.0%, respectively. Meanwhile, the increase is 18.9%, 9.4% and 7.0%, respectively, when the thickness increased from 8 to 10mm. This is because that in the beam mechanism, with increasing flange plate thickness, the flexural resistance of the beam-column joint get improved; in the CA stage, the tension area of the joint increased by 33.3% and 25.0% when the flange plate thickness increased from 6 mm to 8 mm and from 8 mm to 10 mm. **Fig. 17 (b)** demonstrates the resistance of the WFPDWA specimen is linearly influenced by the flange plate width. As the flange plate width increases from 110 to 150 mm, the FPL, CAI and ultimate CA capacity of the frame is increased by 12.8%, 12.3% and 39.3%, respectively. The analysis shows that the rotational behaviour of the specimen is insensitive to the flange plate thickness and width. Finally, several equations are proposed in the figure to predict the resistance of the specimen regarding varying flange plate dimensions under progressive collapse scenario.

Fig. 16: Comparison of the load-MJD curve.

(a) With various thickness (case1)

(b) With various width (case2)

Fig. 17: Comparison of resistance and rotation for the WFPDWA specimen with various flange plate thickness and width. (Note: the notation FPL, CAI and CA in the figure represent the stage when the first peak load reached, CA initiated, and the ultimate CA load achieved.)

4.3 Effect of the Haunch plate

The connection with welded haunches can provide redundant flexural resistance and reduce the demands at the beam flange groove welds [36]. The plastic hinge formation mode and failure mode of the WTBH specimen are plotted in **Fig. 18**. It can be seen that the plastic hinges are close to the haunch

plate-to-beam flange interface, and the fracture of the specimen initiates at the beam flange. The simulation result suggests that the existing of haunch plates can protect the beam-column joint by shifting ductile fracture to the beam flange region, thereby making the connection stronger and more ductile. In **Fig. 19**, it is visible that strengthen by adding haunch plates could considerably improve the load-carrying capacity of the specimen. Another observation is that the specimen ductility is limited influenced by the haunch plate, and the WTBH specimen has a similar rotation capacity when compares with the DWA and WFPDWA specimen. In addition, the connection with bolted angles (TSDWA) is slightly weaker than corresponding welded flange plates (WFPDWA) because the former is susceptible to net section fracture through the bolt holes. Meanwhile, when the haunch angle increases from 25 to 35°, the load-carrying capacity of the specimen are enhanced (**Fig. 20**), but the relevant figure is reduced when the angle increases from 35 to 40°, which implies that the greater efficiency can be achieved when the haunch angle around 35°.

Since the top haunch in the WTBH connection will result in construction difficulties to the installation of the floor, the WTPBH connection was thus proposed. The connection keeps the lower haunch plate and adopts flange cover plate on the top flange of the beam. It indicates that the load-carrying capacity of the WTPBH specimen is slightly less than that of the WTBH (by 4.4%), and a similar ductility can be noticed between two connections, as shown in **Fig. 21**.

In order to evaluate the fracture initiation in the connection, the distribution of the PEEQ index (PEEQI), defined as the plastic equivalent strain (PEEQ) divided by the yield strain, was investigated. The index was found in proportion to rupture index (RI) [37] and can be used as a simplified way to evaluate the potential of ductile fracture. For the linear Drucker-Prager plasticity model, PEEQ is defined as $\bar{\epsilon}^{pl} |_0 + \int_0^t \dot{\epsilon}^{pl} dt$; where $\bar{\epsilon}^{pl} |_0$ is the initial equivalent plastic strain and $\dot{\epsilon}^{pl}$ is the equivalent plastic strain rate [29]. In the study, PEEQI on four locations is captured to define the fracture tendency of the connection, marked as a, b, c and d in **Fig. 14**. They are represented by fracture of the weld or the net section of the flange plate (or seated angle); tear-out of the net section of the shear tab (or web angle); bearing of the bolt hole and fracture of the beam flange. Comparison of PEEQI between various specimens is shown in **Fig. 22**. It indicates that the BFP and WFPDWA specimen could be vulnerable to the fracture of the flange plate-column welds, while the fracture of the

TSDWA specimen will initially occur on the top seated angle. In the WTBH and WTPBH specimen, PEEQI is concentrated on the beam flange, implying the modification configuration by adding haunches can effectively change the fracture tendency from the joint to the beam, and thus ensuring the full development of the beam capacity.

Fig. 18: Location of the plastic hinge and failure mode of the specimen with WTBH connection.

Fig. 19: Comparison of rotation and resistance for various connections.

Fig. 20: Load versus MJD curve with various haunch angle.

Fig. 21: Comparison of resistance between the WTBH and WTPBH connection.

Fig. 22: Comparison of PEEQ between various connections. (Note: the dimension of the BFP and TSDWA specimen is according to the tests; the flange cover plate for the WFPDWA specimen is 110x70x8 mm; the haunch plate angle for WTBH and WTPBH is 30°.)

5. Concluding remarks

The aim of this research is to investigate the response of steel substructures under column-removal scenario and to provide necessary data for understanding the effect of connection configuration on the structural progressive collapse behaviours. The following conclusions can be drawn from the study:

1. The test results imply that the acceptance criterion in ASCE 41 is too conservative in the prediction of the rotation capacity for the specimen with BFP connections.
2. For steel frames which are not involved in bolt shank shear failure when subject to progressive collapse, the computational model based on shell element with simplified bolt-surface coupling and fracture criterion was found appropriate for assessing the resistance and failure mode of the structure. The method can lead to a more efficient solution than the refinement solid modelling strategy.
3. Specimen adopting the WFPDWA connection has greater progressive collapse resistance at both CAA and CA stage when compares with the DWA connections. Increase the thickness and width of the cover plate of the WFPDWA specimen causes a linear improvement in terms of load-carrying capacity. On the contrary, flange plate dimensions have a negligible influence on the rotation capacity of the WFPDWA specimen.
4. The connection with haunch plates leads to an effective strengthening on the load-carrying capacity of the frame, and the failure is shifted from the joint to the beam. Specimen with the WTPBH connections demonstrates a similar performance to the WTBH connections but with better construction efficiency.

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References

- [1] J. Russell, J. Sagaseta, D. Cormie, A. Jones. Historical review of prescriptive design rules for robustness after the collapse of Ronan Point. *Structures*. 20 (2019) 365-73.
- [2] N.S. Lim, K.H. Tan, C.K. Lee. Experimental studies of 3D RC substructures under exterior and corner column removal scenarios. *Engineering Structures*. 150 (2017) 409-27.
- [3] S. Elkoly, B. El-Ariss. Progressive collapse evaluation of externally mitigated reinforced concrete beams. *Engineering Failure Analysis*. 40 (2014) 33-47.
- [4] S. Li, S. Shan, C. Zhai, L. Xie. Experimental and numerical study on progressive collapse process of RC frames with full-height infill walls. *Engineering Failure Analysis*. 59 (2016) 57-68.
- [5] B. Yang, K.H. Tan. Experimental tests of different types of bolted steel beam-column joints under a central-column-removal scenario. *Engineering Structures*. 54 (2013) 112-30.
- [6] W. Wang, C. Fang, X. Qin, Y. Chen, L. Li. Performance of practical beam-to-SHS column connections against progressive collapse. *Engineering Structures*. 106 (2016) 332-47.
- [7] F. Luo, Y. Bai, J. Hou, Y. Huang. Progressive collapse analysis and structural robustness of steel framed modular buildings. *Engineering Failure Analysis*. 104 (2019) 643-56.
- [8] H. Tavakoli, M. Afrapoli. Robustness analysis of steel structures with various lateral load resisting systems under the seismic progressive collapse. *Engineering Failure Analysis*. 83 (2018) 88-101.
- [9] ASCE, Minimum design loads for buildings and other structures, SEI/ASCE 7-10, Reston, VA, 2010.
- [10] DoD, Department of Defense, Unified Facilities Criteria (UFC): Design of Buildings To Resist Progressive Collapse, Unified Facilities Criteria (UFC) 4-023-03, 2009.
- [11] GSA, Progressive Collapse Analysis and Design Guidelines for New Federal Offices Buildings and Major Modernization Projects, General Services Administration, Washington, DC, 2003.
- [12] P.M. Stylianidis, D.A. Nethercot. Modelling of connection behaviour for progressive collapse analysis. *Journal of Constructional Steel Research*. 113 (2015) 169-84.
- [13] J.M. Weigand, J.W. Berman. Integrity of Steel Single Plate Shear Connections Subjected to Simulated Column Removal. *Journal of Structural Engineering*. 140 (2014) 04013114.
- [14] J.M. Weigand, J.W. Berman. Integrity of Bolted Angle Connections Subjected to Simulated Column Removal. *Journal of Structural Engineering*. 142 (2016) 04015165.
- [15] S.A. Oosterhof, R.G. Driver. Behavior of Steel Shear Connections under Column-Removal Demands. *Journal of Structural Engineering, ASCE*. 141 (2015) 1-14.
- [16] H. Daneshvar, R.G. Driver. Performance evaluation of WT connections in progressive collapse. *Engineering Structures*. 167 (2018) 376-92.
- [17] H.S. Lew, J.A. Main, S.D. Robert. Performance of steel moment connections under a column removal scenario. I: Experiments. *Journal of Structural Engineering, ASCE*. 139 (2013) 98-107.
- [18] F. Dinu, I. Marginean, D. Dubina. Experimental testing and numerical modelling of steel moment-frame connections under column loss. *Engineering Structures*. 151 (2017) 861-78.
- [19] X. Qin, W. Wang, Y.Y. Chen, Y.H. Bao. A special reinforcing technique to improve resistance of beam-to-tubular column connections for progressive collapse prevention. *Engineering Structures*. 117 (2016) 26-39.
- [20] W. Wang, H. Li, J. Wang. Progressive Collapse Analysis of Concrete-filled Steel Tubular Column to Steel Beam Connections Using Multi-scale Model. *Structures*. 9 (2017) 123-33.
- [21] W. Zhong, B. Meng, J. Hao. Performance of different stiffness connections against progressive collapse. *Journal of Constructional Steel Research*. 135 (2017) 162-75.
- [22] ASCE, Seismic rehabilitation of existing buildings. ASCE/SEI 41-13. Reston, American Society of Civil Engineers, 2014.
- [23] H. Fu, J. Zhang, J. Jiang, Z. Wang. A Ductility-Centred Analytical Model for Axially Restrained Double-span Steel Beam Systems Subjected to Sudden Columns Loss. *Structures*. 10 (2017) 197-208.
- [24] K. Khandelwal, S. El-Tawil. Collapse Behavior of Steel Special Moment Resisting Frame Connections. *Journal of Structural Engineering, ASCE*. 133 (2007) 646-55.

- [25] C. Lee, S. Kim, K. Lee. Parallel Axial-Flexural Hinge Model for Nonlinear Dynamic Progressive Collapse Analysis of Welded Steel Moment Frames. *Journal of Structural Engineering*. 136 (2010) 165-73.
- [26] F. Sadek, J.A. Main, H.S. Lew, Y. Bao. Testing and analysis of steel and concrete beam column assemblies under a column removal scenario. *Journal of Structural Engineering*, ASCE. 137 (2011) 81–92.
- [27] J. Zhang, J. Jiang, S. Xu, Z. Wang. An Investigation of the Effect of Semi-rigid Connections on Sudden Column Removal in Steel Frames. *Structures*. 13 (2018) 166-77.
- [28] L. Li, G. Li, B. Jiang, Y. Lu. Analysis of robustness of steel frames against progressive collapse. *Journal of Constructional Steel Research*. 143 (2018) 264-78.
- [29] Abaqus-6.13. Analysis User's Manual. SIMULIA, 2013.
- [30] B. Yang, K.H. Tan. Numerical analyses of steel beam-column joints subjected to catenary action. *Journal of Constructional Steel Research*. 70 (2012) 1-11.
- [31] H.X. Yu, I.W. Burgess, J.B. Davison, R.J. Plank. Numerical simulation of bolted steel connections in fire using explicit dynamic analysis. *Journal of Constructional Steel Research*. 64 (2008) 515-25.
- [32] H.L. Yu, D.Y. Jeong. Application of a stress triaxiality dependent fracture criterion in the finite element analysis of unnotched Charpy specimens. *Theoretical and Applied Fracture Mechanics*. 54 (2010).
- [33] Y.B. Bao, T. Wierzbicki. On fracture locus in the equivalent strain and stress triaxiality space. *International Journal of Mechanical Sciences*. 46 (2004) 81-98.
- [34] Y.Q. Wang, H. Zhou, Y.J. Shi, J. Xiong. Fracture Prediction of Welded Steel Connections Using Traditional Fracture Mechanics and Calibrated Micromechanics Based Models. *International Journal of Steel Structures*. 11 (2011) 351-66.
- [35] T.H. Zhou, L. W.C., Y. Guan, L. Bai. Damage analysis of steel frames under cyclic load based on stress triaxiality. *Engineering Mechanics*. 31 (2014) 146-55.
- [36] Q. Yu, C. Uang, J. Gross. Seismic rehabilitation design of steel moment connection with welded haunch. *Journal of Structural Engineering*. 126 (2000) 69-78.
- [37] S. El-Tawil, E. Vidarsson, T. Mikesell, S. Kunnath. Inelastic behavior and design of steel panel zones. *Journal of Structural Engineering*. 125 (1999) 183-93.

Highlights

- Experimental investigation into progressive collapse performance of steel beam-column sub-frame with bolted flange plate connections.
- Numerical study of the specimen from the test using modified modelling strategy.
- Good agreement between the model predictions and experimental outcomes.
- The effects of connection configuration on the behaviour of steel subassemblies under column removal scenario were revealed based on parametric studies.

Conflict of interest statement

The authors declare that we do not have any commercial or associative interest that represents a conflict of interest in connection with the work submitted.

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