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A COMPARISON STUDY OF MODELING METHODS OF SIMPLE STEEL CONNECTIONS SUBJECT TO COLUMN REMOVAL SCENARIOS

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A B S T R A C T

When investigating the global behavior of steel and composite structures, numerical modeling is preferred to physical testing as the latter is much more costly and time-consuming. To date, various models have been used to simulate the behavior of steel and composite connections subjected to catenary action. This is a key attribute to structural robustness under column removal scenarios. The main difference of these methods lies in the type of elements chosen to model the connections, including three-dimensional solid elements, component-based model and plastic hinge model. Although each model has its own merit and strength, not all of them are suitable for building up the whole structural model for analysis of global behavior, which is required for assessing the robustness of structures against progressive collapse. To evaluate the pros and cons of different methods for whole structural model, a systematic comparison study of connection elements used by these methods, i.e. solid, spring (or fiber) and hinge elements, is presented. All three types of models are validated by physical test data and they show good agreement with test data. Based on accuracy and time, some helpful suggestions on selection of modeling method are provided.

1 INTRODUCTION

Progressive collapse is defined as “the spread of an initial local failure from element to element, resulting eventually in the collapse of an entire structure or a disproportionately large part of it” in ASCE (2010). Initiated by the Ronan point apartment building collapse (Krauthammer 2008) and the World Trade Center collapse (Hamburger el al. 2002), progressive collapse has attracted the interest of engineering fraternity in mitigating such event.

Suitable joint modeling is necessary to conduct analysis to evaluate the resistance of structures against progressive collapse. In the literature, a wide variety of modeling methods have been developed for beam-column joints, including finite element (FE) models using three-dimensional (3-D) solid elements, component-based models (also referred to as spring or fiber models) and plastic hinge models.

Three-dimensional solid element is available in commercial software such as ABAQUS, ANSYS, LS-DYNA, etc. The configuration of beam-column joints can be well replicated by solid elements. With proper definition of failure criteria, the behavior of beam-column joints can be well captured by the 3-D FE model.

Component-based model is used to discretize beam-column joints into basic components or springs. In comparison with 3-D solid elements, component-based model neglects the subtle details of beam-column joints but maintains fundamental components which dominate the joint behavior. This makes the component-based model much more computationally efficient than 3-D solid element model. Component-based model has been incorporated into EC3 Part 1-8 (BSI 2005) for the design of conventional joints. For joints subject to catenary action, several models have been developed for specific types of joints to date (Del Savio et al. 2009, Bzdawka & Heinisuo 2010, Stylianidis 2011, Main & Sadek 2012, Piluso et al. 2012, Taib 2012, Oosterhof 2013, Yang & Tan 2013, Koduru & Driver 2014, Main & Sadek 2014, Yang et al. 2015).

Plastic hinge model has been widely used to simulate beam-column joints in seismic analysis when axial force can be neglected (Ikeda & Mahin 1986, Scott & Fenves 2006). It is a model with the highest level of simplification. Thus, compared to component-based models, even more details of beam-column joints are omitted.

In this paper, all three types of modeling techniques are introduced and validated by experimental test data. A comparison study among these models is conducted and suggestions on the global structural modeling will be made.

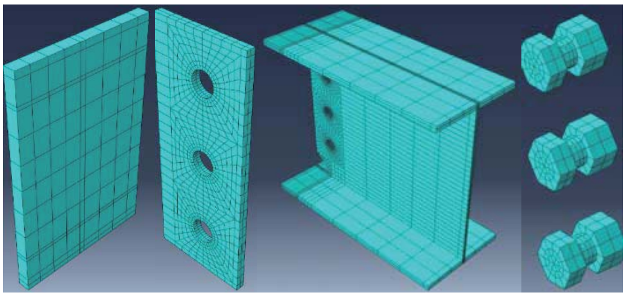
2 THREE DIMENTIONAL SOLID FE MODEL

2.1 *Modeling Techniques*

The numerical analyses are conducted using ABAQUS (Dassault Systèmes 2011). Prior to the conduct of analyses, several assumptions are made in the numerical models as follows: Bolts are placed eccentrically in bolt holes to build contact pairs. Beam stubs near the middle joint, with its length equal to the height of I-shaped beams, are modeled by solid elements (C3D8R), whereas other parts of the beams are modeled by beam elements (B31). Steel assemblies are restrained against lateral displacements. Deformations of column stubs are neglected since columns are strengthened. Welds are simulated by ties instead of merging two parts together to accommodate various element sizes. Besides, weld failure is not considered in the simulations. Ideal hinges are used to simulate the boundary conditions at the beam ends except for the cases in which rotational stiffness of supports is obtained from experimental results. Displacement-controlled loads with smooth step are applied on the middle joint. Explicit dynamic solver is utilized to conduct quasi-static analyses of steel joints.

Material properties are obtained from coupon tests conducted along with the joint tests. Engineering stress-strain relationships obtained from the coupon tests are converted to true stress-strain curves in the numerical models. Ductile damage for metals is chosen as the damage model for steel. In the analyses, equivalent plastic strain of 0.3 is assumed as the start of damage, based on S275 steel coupon tests. Linear damage evolution in terms of equivalent plastic displacement is used. As the failure point is mesh-size dependent, trial and error is adopted to best fit the experimental results.

Mesh design is the key to joint modeling. In this study, different mesh sizes are used for different parts of the joint. Figure 1 shows the mesh size for simulating a column flange, a fin plate, a beam stub and some bolts. A relatively coarse mesh is adopted for the column flange. Based on mesh convergence analysis (Daneshvar & Driver 2011), two layers of elements are used in the thickness direction of the fin plate. The strength of the beam web is lower than that of the fin plate and failure is expected to initiate at the beam web. Thus, four layers of elements are used in the thickness direction of the beam web to simulate block tearing. For beam flanges welded to column flanges, yielding and fracture of steel may occur according to the experimental results (Li et al. 2015). A fine mesh size of about 5 mm is adopted at these locations. For beam sections away from the column flange, a coarser mesh size of up to 30 mm is utilized to save computational cost. A coarser mesh is also defined for bolt shanks (see Fig. 1) since their strength and stiffness are substantially larger than those of beam webs and fin plates.



(a) (b) (c) (d)

Figure 1. Mesh sizes for each part of the joint: (a) Column flange; (b) Fin plate; (c) Beam stub; (d) Bolts.

Contacts between different components are defined in the model. A set of contact relations is provided by ABAQUS (Dassault Systèmes 2011). In this study, contact pairs are defined to model the force transfer between components. For a single bolt row, five contact pairs are required between bolt heads and the fin plate, bolt nuts and the beam web, bolt shanks and the fin plate, bolt shanks and the beam web, and the fin plate and the beam web. Tangential behavior of contact pairs is simulated by defining a penalty friction coefficient of 0.3 according to Coulomb friction model, whereas normal behavior is represented by a hard contact formulation with a penalty constraint enforcement method.

2.2 *Model validation*

Numerical models of steel beam-column joints are verified by the experimental results of Oosterhof (2013). In the experimental program, nine steel joints with fin plate connections were tested under column removal scenarios. Two types of connections with three or five bolts were included. Figure 2 shows a typical numerical model for the test specimen with five bolts in ABAQUS.

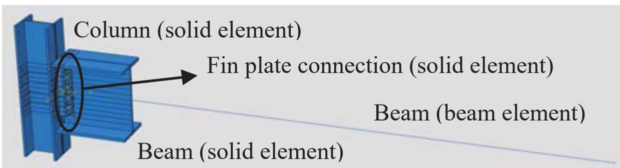


Figure 2. Numerical model for specimen ST5A-1 by Oosterhoof (2013).

Figure 3 shows a comparison between experimental and numerical test results. Only the results for specimen ST5A-1 are depicted in this paper because of the limitation of space. Numerical results agree well with experimental results in terms of vertical load-beam rotation curve and horizontal force-beam rotation curve. Tables 1 and 2 summarize the maximum vertical loads and horizontal reaction forces. It can be seen that 3-D solid element models can replicate the experimental results well.

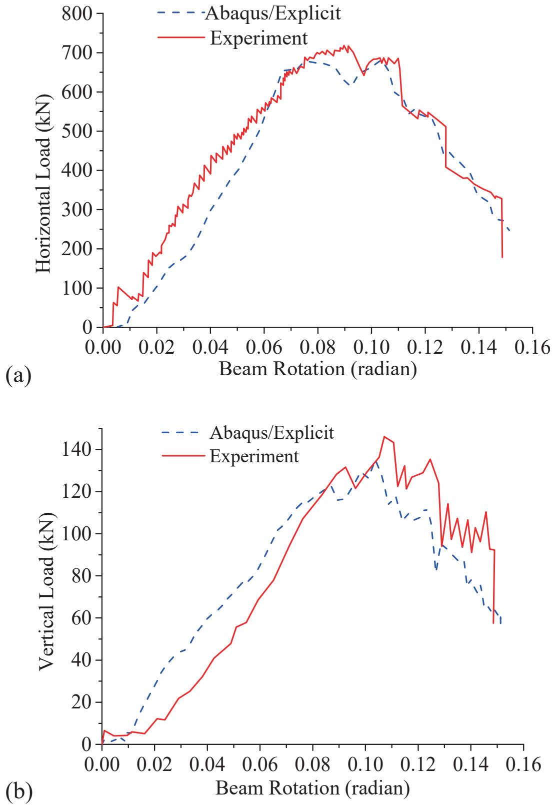


Figure 3. Comparisons of experimental and numerical force vs beam rotation curves for specimen ST5A-1: (a) Horizontal force; (b) Vertical force.

Table 1. Comparison between numerical analyses and experimental tests (horizontal force).

Specimen ID	Peak force (kN)		Relative
	ABAQUS	Test	error (%)
ST3A-1	504.9	515.7	-2.1
ST3A-2	504.9	507.7	-0.6
ST3A-3	505.3	522.1	-3.2
ST3B-1	343.8	330.3	4.1
ST3B-2	340.3	334.8	1.6
ST5A-1	674.5	706.5	-4.5
ST5A-2	765.8	823.0	-7.0
ST5B-1	445.8	471.7	-5.5
ST5B-2	488.4	503.9	-3.1

3 COMPONENT-BASED MODEL

3.1 *General concept*

A component-based model consists of a group of basic components. Each component has its own constitutive relationship in terms of force and corresponding displacement. As for fin plate connections, two types of components are included, viz. spring elements between column and beam flanges to simulate the behavior of gaps and a single bolt spring connecting the column flange to the beam web, as shown in Figure 4. A single bolt connection spring consists of a series of components, namely, bolt in bearing between the fin plate and the beam web, bolt in shear and the friction between these components. It is notable that the component of column web panel in shear is not incorporated in the model, since it is much stronger than the aforementioned springs.

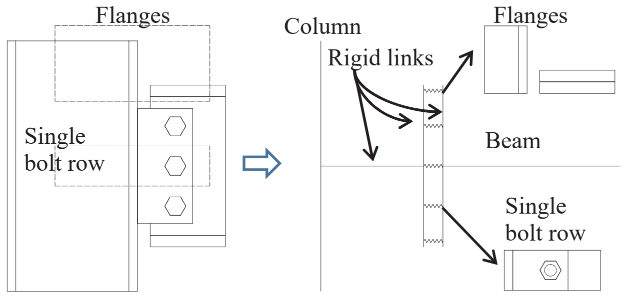


Figure 4. Assembly of component-based model for fin plate connection.

Table 2. Comparison between numerical analyses and experimental tests (vertical force).

Specimen ID	Peak force (kN)		Relative
	ABAQUS	Test	error (%)
ST3A-1	59.4	61.8	-3.9
ST3A-2	118.7	126.7	-6.3
ST3A-3	100.8	105.6	-4.5
ST3B-1	66.5	70.0	-5
ST3B-2	53.5	57.7	-7.3
ST5A-1	135.1	135.3	-0.1
ST5A-2	112.8	135.7	-16.9
ST5B-1	76.2	74.0	3.0
ST5B-2	74.0	67.5	9.6

3.2 *Constitutive relationship for basic components*

Single bolt row shown in Figure 4 consists of components including bolts in bearing and shear. Several models have been proposed to predict the ultimate strength $R_{n,br}$ of bolts in bearing in steel plates and included in national codes such as EC3 Part 1-8 (BSI 2005), AISC 360-10 (2010) and CSA S16-09 (2009). In this section, an equation in AISC (Equation J3-6b) is adopted as follows:

$$R_{n,br} = 1.5(L_e - \frac{d_b}{2})t\sigma_u \leq 3td_b\sigma_u \qquad (1)$$

where L_e = the edge distance, d_b = the nominal diameter of bolt, t = the thickness of the plate, and σ_u = the ultimate strength of steel plate. It is noteworthy that nominal strength of steel is used in the equation instead of design value.

The stiffness of bolt in bearing k_i is determined from Equation (2) proposed by Rex & Easterling (1996):

$$k_i = \frac{1}{\frac{1}{k_{br}} + \frac{1}{k_b} + \frac{1}{k_v}} \tag{2}$$

$$k_{br} = 120 t \sigma_y d_b^{(4/5)} \tag{3}$$

$$k_b = 32Et\left(\frac{L_e}{d_b} - \frac{1}{2}\right)^3 \tag{4}$$

$$k_v = (20/3)Gt\left(\frac{L_e}{d_b} - \frac{1}{2}\right) \tag{5}$$

where k_{br}, k_b and k_v = the stiffness of bolt bearing, edge steel plate bending and shearing, respectively, σ_y = the yield strength of steel plate, E and G = the respective moduli of elasticity and shear.

Since the diameter of bolt holes is greater than that of bolt shanks, the bolt shanks will move without any contact with fin plates or beam webs. During the movement of bolt shanks, only friction force exists and its magnitude depends on the surface treatment of plates and bolt types. An estimated value of 30 kN is suggested for non-preloaded bolts by Oosterhoof (2013) when snug-tight installation is used.

Rex & Easterling (2003) also proposed the force-displacement relationship of bolts in bearing based on experimental results. Based on this relationship, the model gives good prediction of the behavior of steel joints (Taib 2012, Oosterhof 2013, Koduru & Driver 2014, Weigand 2014). Thus, it is used in this section to represent the constitutive model for bolts in bearing, as expressed in Equation (6).

$$F_{br} = R_{n,br}\left[\frac{1.74\overline{\Delta}}{(1+\overline{\Delta}^{0.5})^2} - 0.009\overline{\Delta}\right] \tag{6}$$

$$\overline{\Delta} = \frac{\Delta K_i}{R_{n.br}} \tag{7}$$

where F_{br} = the resultant force, Δ = the displacement. Other parameters are the same as before.

The ultimate strength of bolts in single shear is determined by Equation (8):

$$R_{nv,bolt} = 0.6\frac{\pi d_b^{\,2}}{4}\sigma_{ub} \tag{8}$$

This equation has been included in design codes such as EC3 Part 1-8 (BSI 2005), AISC 360-10 (2010) and CSA S16-09 (2009). According to test results of bolts in shear (Moore 2007), a coefficient of 1.25 can be used to convert the nominal strength of steel to its ultimate strength. Besides, the predicted shear resistance should be reduced by a factor of 0.7, if shear plane goes through bolt threads.

The stiffness and resistance of beam and column flanges in compression are much larger than that of a unit bolt row due to the contribution of the effective area of flanges. Therefore, it is assumed that the stiffness and resistance of the beam and the column flanges are infinite when the gap between the beam and the column flange closes up. Thus, the force-displacement curve of the beam and the column flanges in compression can be determined as shown in Figure 5.

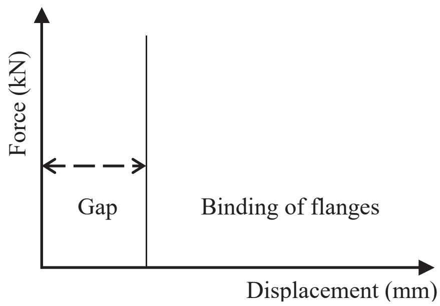


Figure 5. Force vs displacement curve for beam and column flanges in compression.

The failure of single bolt row is dominated by its weakest component. Experimental tests on fin plate subject to catenary action (Oosterhof 2013, Yang 2013, Weigand 2014) indicate two possible failure modes, namely, shear failure of bolts and tear-out failure of fin plates, depending on the relative resistance between the bolts and the fin plate. In component-based models, the deformation capacity of each bolt row is defined in tension and compression separately. Oosterhoof (2013) provided the ultimate deformations of bolt rows in tension. The value is about 0.8 to 1.0 time of edge distance. For bolt rows in compression, shear failure of bolts is dominant over tear-out failure of fin plate, and the ultimate deformation is around 0.23 times of bolt diameter.

3.3 *Model validation*

Component-based model can be applied using Excel, MATLAB code and FE packages. In this section, the FE package ABAQUS is chosen. Components are represented using CONNECTOR element (Dassault Systèmes 2011). Figures 6 and 7 show the properties of two typical components. Failure criteria of the components are determined by the average deformation capacity of bolt rows.

After determining the spring properties, nonlinear springs are assembled in the beam-column joint. Thereafter, displacement-controlled vertical load is applied to the middle joint. Figure 8 depicts a comparison of load-displacement curves between experimental results (Oosterhof 2013) and component-based joint model predictions for specimen ST5A-1. Table 3 lists the maximum horizontal forces for all the specimens. It indicates that the component-based model is capable of predicting the overall load-displacement responses with reasonably good accuracy.

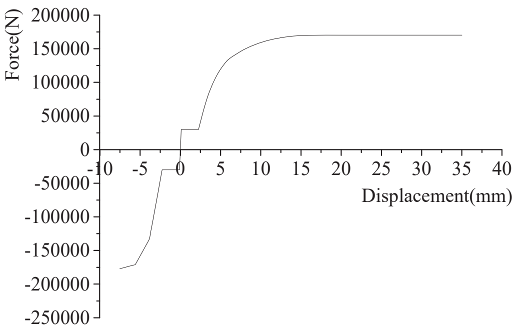


Figure 6. Constitutive curve for bolt row with 22mm bolt and 9.5mm fin plate.

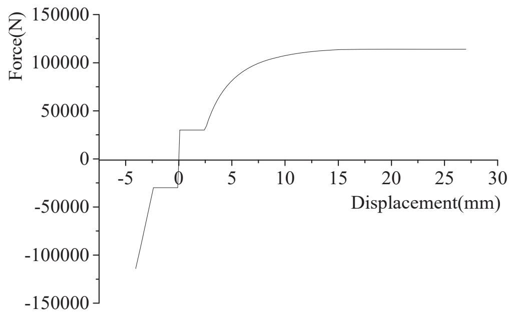


Figure 7. Constitutive curve for bolt row with 19mm bolt and 6.4mm fin plate.

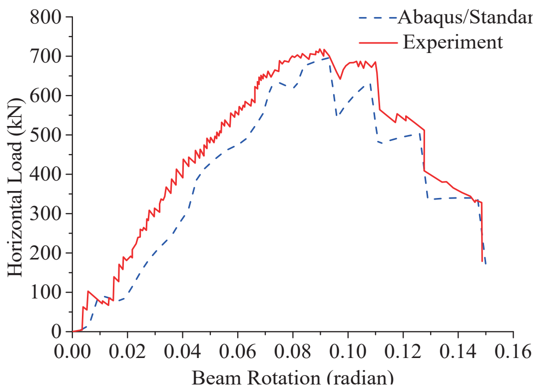


Figure 8. Comparison between component-based modeling and test results for specimen ST5A-1 by Oosterhoof (2013).

4 HINGE MODEL

Plastic hinge models are commonly used in numerical analyses on seismic response of building structures, in which beam-column joints are subject to cyclic loadings and axial forces in the beam are neglected. However, when beam-column joints are subject to catenary action, tension forces developed in the beam can be dominant at failure. Thus, the tension force has to be considered in the proposed plastic hinge model for beam-column joints under column removal scenarios. Lee et al. (2010) developed a plastic hinge model for rigid beam-column joints by adding an axial spring. The joint was strengthened so that deformation or failure was precluded from the joint and only beam element was used for the hinge model. However, for simple joints, fin plate connections are usually the weakest link. Thus the resistance and ductility of joints have to be taken into account in the hinge model as well.

Table 3. Comparison between numerical analyses and experimental tests (horizontal force).

Specimen ID	Peak force (kN) ABAQUS	Peak force (kN) Test	Relative error (%)
ST3A-1	507.5	515.7	-1.6
ST3A-2	507.5	507.7	0
ST3A-3	510.8	522.1	-2.2
ST3B-1	333.0	330.3	0.8
ST3B-2	337.8	334.8	0.9
ST5A-1	695.4	706.5	-1.6
ST5A-2	805.8	823.0	-2.1
ST5B-1	442.9	471.7	-6.1
ST5B-2	499.0	503.9	-1.0

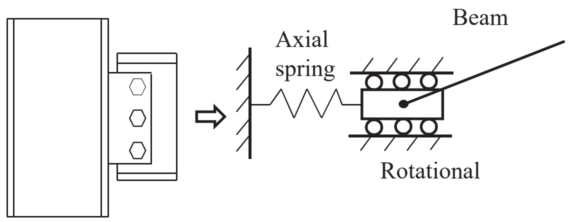


Figure 9. Illustration of hinge model.

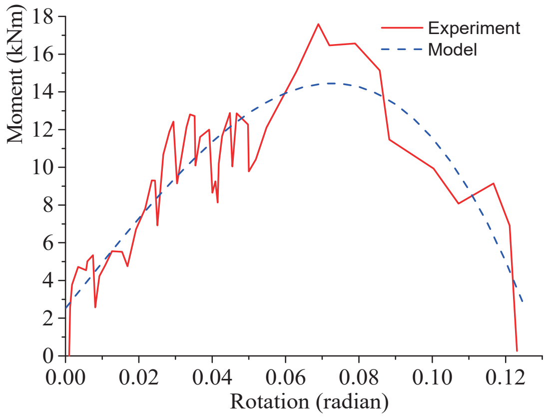
To apply plastic hinge models to the numerical analyses of structures under column removal scenarios, beam-column joints are simplified as an axial spring and plastic hinge with zero length, as shown in Figure 9. Shear springs in the joint are assumed to be rigid without failure. Therefore, shear failure of the joint is not considered in the model. Even though the moment resistances of fin plate joints are negligible, a plastic hinge is needed to take account of the rotational capacity of the joints.

The properties of axial and rotational springs can be obtained from either test results or component-based models subject to catenary action. Therefore the interaction of axial force and bending moment of the connection is considered implicitly. Figure 10 depicts the properties of axial and rotational springs of specimen ST3A-1 tested by Oosterhof (2013). Curve-fitting technique is used to derive the spring properties from test data. Third-order polynomial equations are selected to fit the test results.

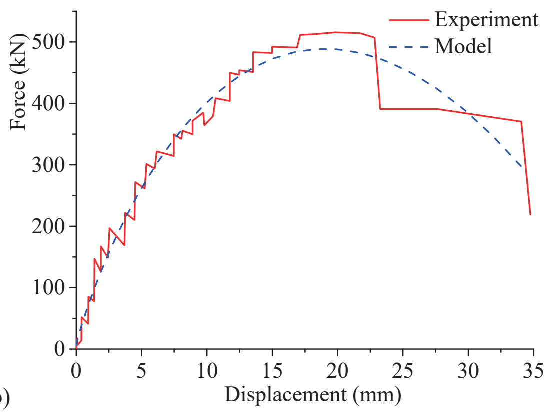
5 DISCUSSION

The computational resource for this study is a personal computer with its technical specifications listed as follows:

Processor: Intel ® Core ™ i7-3720QM CPU @ 2.6GHz
Installed memory (RAM): 16.0 GB (15.9 GB usable)
System type: Windows® 8.1 pro, 64-bit
ABAQUS Edition: v 6.11



(a)



(b)

Figure 10. Force versus displacement curves of hinge model: (a) Axial spring; (b) Rotational spring.

Table 4. Comparison of computational cost (unit: seconds).

Specimen ID	3-D solid element model	Component-based model
ST3A-1	48,567	57
ST3A-2	48,567	57
ST3A-3	51,542	55
ST3B-1	45,569	54
ST3B-2	44,153	52
ST5A-1	53,868	40
ST5A-2	53,407	39
ST5B-1	-	41
ST5B-2	45,908	53
Average	48,948	50

Table 4 lists a comparison of computational cost between different modeling methods in the unit of seconds. Since the costs of component-based model and hinge model are similar, only computational time for the former one is provided. It can seen that simplified models obviously have an advantage over 3-D solid elements in terms of computational cost, even though all of them provides good agreement with experimental results under column removal scenarios. This indicates that simplified models, including component-based models and hinge models, are more desirable when considering analysis of global behavior of an entire building. Therefore, simplified models are easier to be used in routine design of building structures against progressive collapse. However, since many details of the beam-column joints are neglected by simplified models, they may not be able to provide very accurate descriptions of the test results as shown in Figure 11. Thus detailed 3-D element models are the first choice to investigate the failure modes and detailing of beam-column connections.


(a)

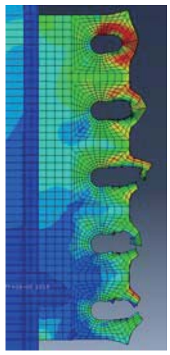

(b)

Figure 11. Comparison of failure modes between numerical and experimental results: (a) Tests; (b) 3-D solid element models.

6 CONCLUSION

In this paper, three types of numerical models, viz. detailed 3-D solid FE model, simplified component-based model and hinge model are introduced and verified by experimental test results. It is found that simplified models are more desirable to be used in numerical analyses of global building structures for routine design against progressive collapse by industry engineers. However, detailed 3-D element models can be used to investigate the failure modes and detailing of beam-column connections.

REFERENCE

AISC (2010). *Specification for Structural Steel Buildings*. ANSI/AISC 360-10. American Institute of Steel Construction, Chicago, I.L., U.S.

ASCE (2010). *Minimum Design Loads for Buildings and Other Structures*. ASCE/SEI 7-10. American Society of Civil Engineers, Virginia, U.S.

BSI (2005). *Eurocode 3: Design of steel structures—Part 1-8: Design of joints*. BS EN 1993-1-8. British Standards Institution, London, U.K.

Bzdawka, K. & Heinisuo, M. (2010). Fin plate joint using component method of EN 1993-1-8. *Journal of Structural*

Mechanics, Vol. 43, No. 1, pp. 25-43.

CSA (2009). *Design of steel structures*. CSA S16-09. Canadian Standards Association, Ontario, Canada L4W 5N6.

Daneshvar, H. & Driver, R. G. (2011). *Behavior of shear tab connections under column removal scenario*. Structures Congress 2011, Las Vegas, NV, US, pp. 2905-2916.

Dassault Systèmes (2011). *ABAQUS 6.11 analysis user's manual*.

Del Savio, A. A., Nethercot, D. A., Vellasco, P. C. G. S., Andrade, S. A. L. & Martha, L. F. (2009). Generalised component-based model for beam-to-column connections including axial versus moment interaction. *Journal of Constructional Steel Research*, Vol. 65, No. 8–9, pp. 1876-1895.

Hamburger, R., Baker, W., Barnett, J., Marrion, C., James, M. & Nelson, H. (2002). *World Trade Center building performance study: Data collection, preliminary observations, and recommendations. WTC 1 and WTC 2*. Federal Emergency Management Agency Washington, D.C., U.S.

Ikeda, K. & Mahin, S. (1986). Cyclic Response of Steel Braces. *Journal of Structural Engineering*, Vol. 112, No. 2, pp. 342-361.

Koduru, S. & Driver, R. (2014). Generalized Component-Based Model for Shear Tab Connections. *Journal of Structural Engineering*, Vol. 140, No. 2, pp. 1-10.

Krauthammer, T. (2008). *Modern protective structures*. CRC Press.

Lee, C., Kim, S. & Lee, K. (2010). Parallel Axial-Flexural Hinge Model for Nonlinear Dynamic Progressive Collapse Analysis of Welded Steel Moment Frames. *Journal of Structural Engineering*, Vol. 136, No. 2, pp. 165-173.

Li, L., Wang, W., Chen, Y. & Lu, Y. (2015). Effect of beam web bolt arrangement on catenary behaviour of moment connections. *Journal of Constructional Steel Research*, Vol. 104, No. 0, pp. 22-36.

Main, J. A. & Sadek, F. (2012). *Robustness of steel gravity frame systems with single-plate shear connections*. U.S. Department of Commerce, National Institute of Standards and Technology.

Main, J. A. & Sadek, F. (2014). Modeling and Analysis of Single-Plate Shear Connections under Column Loss. *Journal of Structural Engineering*, Vol. 140, No. 3, pp. 04013070.

Moore, A. M. (2007). *Evaluation of the current resistance factors for high-strength bolts*, M.S. Thesis, University of Cincinnati, U.S.

Oosterhof, S. A. (2013). *Behaviour of Steel Shear Connections for Assessing Structural Vulnerability to Disproportionate Collapse*, Ph.D. Thesis, University of Alberta, Canada.

Piluso, V., Rizzano, G. & Tolone, I. (2012). An advanced mechanical model for composite connections under hogging/sagging moments. *Journal of Constructional Steel Research*, Vol. 72, No. 0, pp. 35-50.

Rex, C. & Easterling, W. (1996). *Behavior and modeling of a single plate bearing on a single bolt*. Virginia Polytechnic Institute and State University, Blacksburg, V.A., U.S.

Rex, C. & Easterling, W. (2003). Behavior and Modeling of a Bolt Bearing on a Single Plate. *Journal of Structural Engineering*, Vol. 129, No. 6, pp. 792-800.

Scott, M. & Fenves, G. (2006). Plastic Hinge Integration Methods for Force-Based Beam–Column Elements. *Journal of Structural Engineering*, Vol. 132, No. 2, pp. 244-252.

Stylianidis, P. (2011). *Progressive collapse response of steel and composite buildings*, Ph.D. Thesis, Imperial College London, United Kingdom.

Taib, M. (2012). *The Performance of Steel Framed Structures with Fin-plate Connections in Fire*, Ph.D. Thesis, University of Sheffield, U.K.

Weigand, J. M. (2014). *The Integrity of Steel Gravity Framing System Connections Subjected to Column Removal Loading*, Ph.D. Thesis, University of Washington, U.S.

Yang, B. (2013). *The Behavior of Steel and Composite strctures under a Middle-Column-Removal Scenario*, Ph.D. Thesis, Nanyang Technological University, Singapore.

Yang, B. & Tan, K. H. (2013). Robustness of Bolted-Angle Connections against Progressive Collapse: Experimental Tests of Beam-Column Joints and Development of Component-Based Models. *Journal of Structural Engineering*, Vol. 139, No. 9, pp. 1498-1514.

Yang, B., Tan, K. H. & Xiong, G. (2015). Behaviour of composite beam–column joints under a middle-column-removal scenario: Component-based modelling. *Journal of Constructional Steel Research*, Vol. 104, No. 0, pp. 137-154.