Steel and Composite Structures, Vol. 9, No. 4 (2009) 381-395 381
DOI: http://dx.doi.org/10.12989/scs.2009.9.4.381

Advanced analysis of cyclic behaviour of plane steel frames with semi-rigid connections

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(Received November 11, 2008, Accepted July 14, 2009)

Abstract. This paper presents the details of an advanced Finite Element (FE) analysis of a plane steel portal frame with semi-rigid beam-to-column connections subjected cyclic loading. In spite of several component models on cyclic behaviour of connections presented in the literature, works on numerical investigations on cyclic behaviour of full scale frames are rather scarce. This paper presents the evolution of an FE model which deals comprehensively with the issues related to cyclic behaviour of full scale steel frames using ABAQUS software. In the material modeling, combined kinematic/isotropic hardening model and isotropic hardening model along with Von Mises criteria are used. Connection non-linearity is also considered in the analysis. The bolt slip which happens in friction grip connection is modeled. The bolt load variation during loading, which is a pivotal issue in reality, has been taken care in the present model. This aspect, according to the knowledge of the authors, has been first time reported in the literature. The numerically predicted results using the methodology evolved in the present study, for the cyclic behaviour of a cantilever beam and a rigid frame, are validated with experimental results available in the literature. The moment-rotation and deflection responses of the evolved model, match well with experimental results. This proves that the methodology for evolving the steel frame and connection model presented in this paper is closer to real frame behaviour as evident from the good comparison and hence paves the way for further parametric studies on cyclic behaviour of flexibly connected frames.

Keywords : semi-rigid connection; steel frame; finite element modeling; cyclic loading.

1. Introduction

During the Northridge earthquake in California in 1994 and Kobe earthquake in 1995, many steel buildings suffered brittle fracture of the welded moment connections (Elnashai, *et al.* 1998) due to several factors such as stress concentration in weld access hole region, weld defects, material deficiencies, defective workmanship, etc. One of the alternatives suggested to alleviate the above problem was the semi-rigid connection. Semi-rigid connections which were not in existence for some decades became popular because of economy. Typical semi-rigid connections are top and seat angles, top and bottom tees, top and bottom flange plates with web connections, extended end plate connection and top and seat angles with double web angle connections.

In reality, steel frame behaviour is affected by parameters of connection such as bolt diameter, number of bolt rows and columns, bolt spacing, bolt grade, plate dimensions, stiffener, column and

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beam sizes, bolt pretension force, yield strength of steel, slip coefficient of contact surfaces, etc. The frames, as a whole also undergoes large deformation while its members going into the inelastic state. Experimental tests cannot cover such a wide range of variables for both connection and frames. A study using finite elements only can model both the connection and the member behavior representing cyclic plasticity. Also much of the knowledge needed to include semi-rigid connection behaviour for design has been derived from detailed advanced finite element studies of bolted connections. Thus, FE models have been frequently used to develop moment-rotation curves of flexibly connected frames, to verify design methodologies based on yield lines and other plastic design concepts and also to assess the local behavior of the connection components such as bolts and end plates along with the overall frame behavior.

1.1 Semi-Rigid connections

A semi-rigid connection can be defined as one that is more flexible than a rigid connection, but stiffer than a pinned one (Nader and Astaneh 1992). Semi-rigid connections are capable of developing a moment at the beam end between 20% and 90% of the fixed end moment. In other words, the rotational stiffness of the connection should lie between $\frac{1}{2}(EI/L)_b$ and $18(EI/L)_b$, where $(EI/L)_b$ represents the flexural stiffness of the beam. As per the Indian code of steel construction (IS 800: 2007) a semi-rigid connection is defined by a non dimensional moment parameter ($m^1 = M_u / M_{pb}$), where M_u and M_{pb} are ultimate moment capacity of connection and moment in the beam at the intersection of the beam column centre line respectively and a non-dimensional rotation parameter ($\theta^1 = \theta_r / \theta_p$), where θ_r and θ_p are relative elastic rotation and plastic rotation respectively, which were originally proposed by Bjorhovde, *et al*. (1990).

1.2 Review of Literature

Limited experimental and analytical studies have been undertaken on the cyclic behaviour of steel frames with semi-rigid connections. Most of the finite element studies have been carried out on beam column joints under cyclic loading and studies on full scale steel frames with semi-rigid connections are scarce. This is one of the reasons for choosing the topic of the present paper. Regarding full frame analyses, Elnashai, *et al*. (1998) conducted experimental investigation on steel frames with semi-rigid connections comprised of top, seat and web angles. They compared the results with fully rigid connection to assess the effect of connection stiffness and capacity on the frame response. Calado (2003) proposed an analytical model to represent the cyclic behaviour of top and seat with web angle for connections using damage accumulation and bolts in cyclic shear. Pucinotti (2006) presented a cyclic mechanical model to simulate the behaviour of top and seat with web angle beam-to-column connections. Subsequently he developed a finite element model and compared the proposed cyclic mechanical model with experimental results, FE results and EC3 predictions. Yang and Kim (2007) conducted experimental investigation on the cyclic behaviour of bolted and welded beam to column connections in a steel portal frame. They studied the inelastic behaviour of the portal frames with three different types of connections (flexible, semi-rigid and rigid).

The connection of interest in the present study is Top and Seat angle with Double Web Angle (TSDWA) connection, which exhibits semi-rigid connection behaviour. From the literature review it is seen that the main drawback of most of the research on semi-rigid connection is that they all deal with isolated joint and sub frames. However, in practical construction, connections do not occur in isolation but as part of a structure. Hence studies on full scale frame behaviour are important to confirm whether the experimentally observed performance of isolated joints and sub frames is indeed representative of their behaviour when they form part of an extensive frame (Miklos Ivanyi 2000).

This paper first presents a short description of the test frame with top seat and double web angle connections that was used in an experimental program reported in the literature (Yang and Kim 2007). The test program is used as the verification basis for the present numerical model for cyclic load analysis. The loading details are given in Yang and Kim (2007). The cyclic load analysis results are validated for load deflection behaviour of the steel frame and moment-rotation behaviour of the connection.

2. Details of the experimental test frame chosen for validation

A single bay and single storey steel portal frame with a top and seat angle with double web angle connection had been taken for the experimental investigation is shown in Fig. 1. The sections used for beam and columns are H-250 X125 X 6 X 9 and H-125 X 125 X 6.5 X 9 respectively and the semi-rigid connection of top and seat angle with double web angle consisted of L-50 X 50 X 6 for web angles, bolted to column flange and beam web, and L-75 X 75 X 6 for top and seat angles, bolted to column and beam flanges. The section and connection details are shown in Fig. 2. The ratio of length of the beam (1600 mm) to the length of the column (1800 mm) is about 0.89. The nominal diameter of all the HSFG bolts used in the specimens is 12 mm. In addition, the bolt holes in the angles, column and beam were 2 mm oversize. A torque-control method was used in tightening the bolts with a torque of 100 Nm. The distance from the back of the beam flange to the column flange air gap was 10 mm.

All the members were made of H-shaped SS400 steel, equivalent to F_y 250 steel. Table 1 shows the

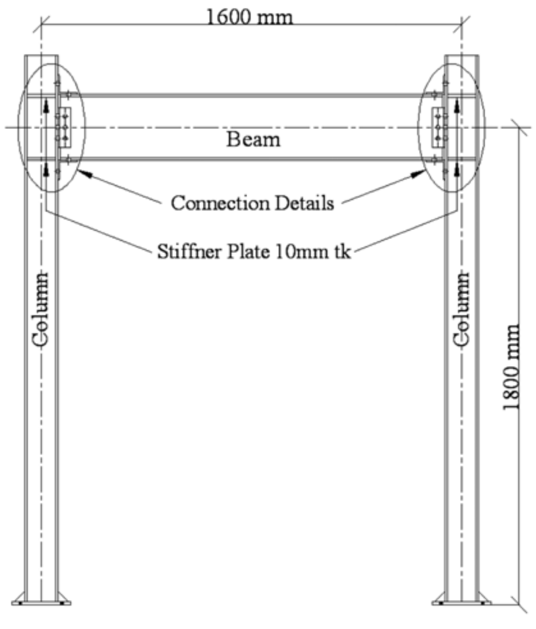


Fig. 1 Schematic arrangement of test frame

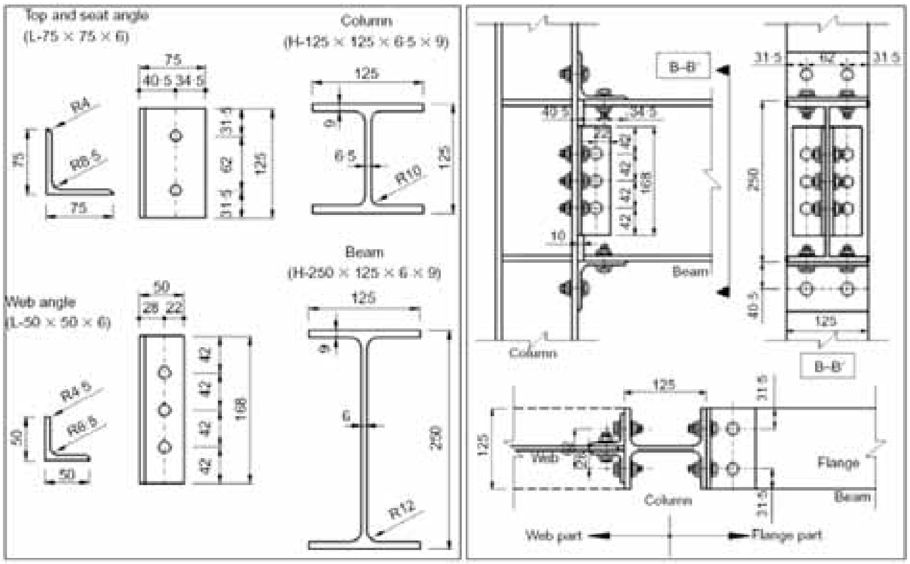


Fig. 2 Geometrical details of beam-column connection

Table 1 Mechanical property of the sections and connections

Members		Steel grade	Yield strength (MPa)	Tensile strength (MPa)	Yield ratio	Elongation (%)
Beam	Flange	SS400	328	447	0.73	31.25
	web		322	423	0.76	32.50
Column	Flange		347	466	0.74	27.50
	web		333	461	0.72	33.75
Top & seat angles			307	432	0.72	32.50
Web angle			350	471	0.74	28.75
Average value	Flange		338	457	0.74	29.38
	web		328	442	0.74	33.13
	angle		329	452	0.73	30.94
Total average value			332	450	0.74	31.15

mechanical properties of all the members obtained from the coupon tests. The deformation controlled lateral cyclic loading was applied at the top of the column until failure occurred.

The deformation parameter for controlling the loading history is the storey drift angle, defined as the storey displacement divided by storey height. The tests had been carried out under displacement control up to collapse of the specimen. The typical cyclic displacement load history of the tests is shown in Fig. 3 which illustrates that the displacement-controlled load history consists of symmetric, step-wise increasing displacements imposed by a hydraulic actuator at the end of the beam. The displacement – controlled history used in these tests was designed to simulate the effects of earthquakes and therefore cause severe failure in the joint area. The displacement increment had been affected as follows. When the displacement is below or equal to 18 mm, a 3 mm increment had been applied; when the displacement increased from 18 to 50 mm, a 4 mm increment had been applied for each cycle; and when the displacement increased above 50 mm, a 6 mm increment had been applied for each cycle.

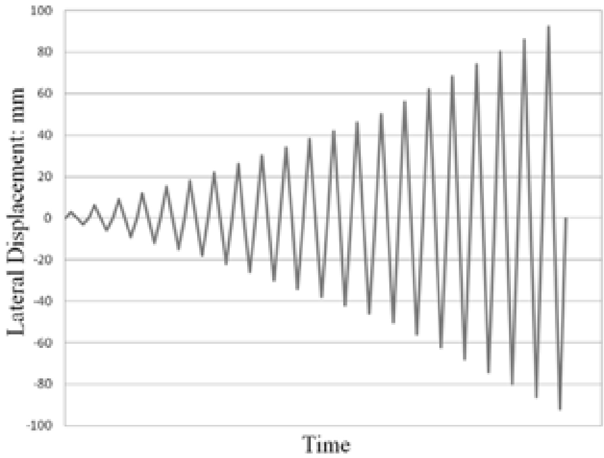


Fig. 3 Loading history

3. Finite element modeling of the frame

The general purpose computer program ABAQUS finite element code was used for the current study. A representative three dimensional FE model of the frame with top and seat angle with double web angles connection is shown in Fig. 4. Accurate modeling of the connection elements as well as realistic simulation of the different material and geometric nonlinearities of the connection characteristics are required in order to achieve desired results. There are several factors affecting the accuracy of FE results. These include the mesh size, different material behaviour of bolts, plate and profiles, choice of elements, and most importantly the modeling of the contact behaviour between elements and modeling

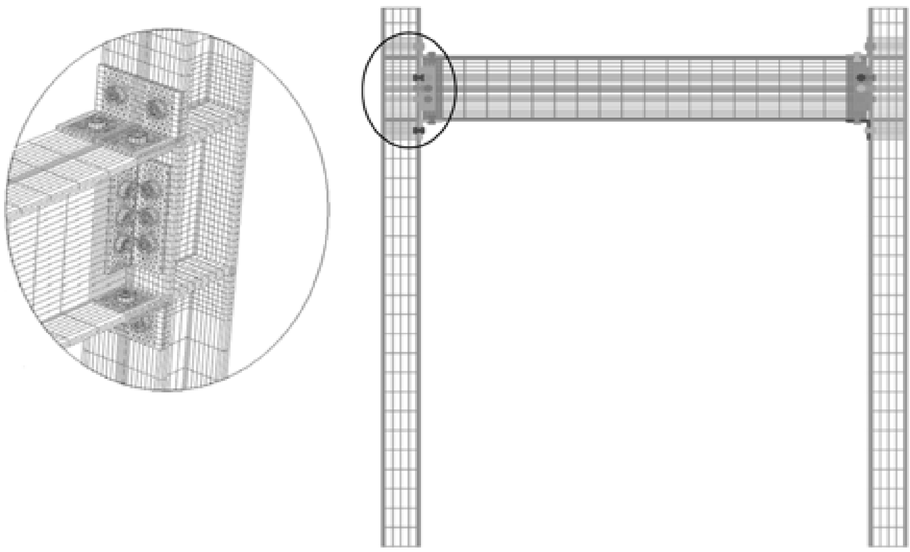


Fig. 4 3D FE model of a TSD connection

of the pretension in the bolts.

The finite element model involves eight angles, thirty four bolts and three I-sections. The total number of elements is about 35000 and the total number of equations (including Lagrange multipliers) is 159000. The number of elements in the mesh is decided by two consequent finer meshes producing the same results. The angles are assumed to be made of SS400 steel, with a modulus of elasticity of 200,000 MPa, yield stress 332 MPa, and Poisson's ratio 0.3.

The beam and columns are discretized using C3D8R eight noded constant-strain brick elements. The angles are idealized by neglecting the fillet radius as shown in Fig. 5(a), and Hexagonal bolt heads and nuts are idealized as circular bolt heads and nuts to simplify the analysis as shown in Fig. 5(b) and washers are not modeled.

The following are the features considered in the present model

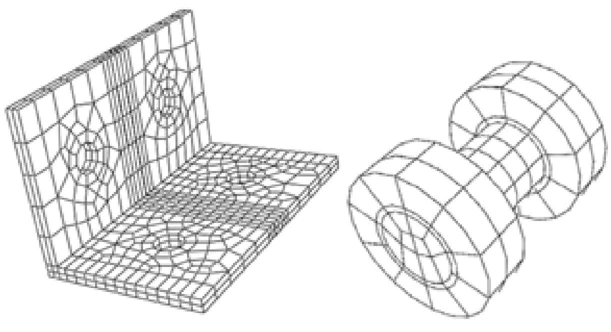
- Material non-linearity taken into consideration by implementing cyclic plasticity material models.
- Non-linearity due to connection behaviour (contact–separation–re-contact phenomenon between the plate and the supporting surface and/ or between the bolt heads and the beam/column flange/ web).
- Bolt slip phenomenon using surface to surface contact between bolt head to component (studied under monotonic loading)
- Bolt load variation according to the response of the model obtained from monotonic loading.

With the inclusion of features mentioned above in the proposed finite element model, the predicted results are realistic and comparable to those of the test frame.

3.1 Contact Modeling

Surface-to-surface contact interaction capabilities available in ABAQUS are used to account for the various forces generated between interacting parts of the model. The semi-rigid behaviour is incorporated by using the contact interface master slave algorithm between the bolt head and plates, plate to plate and bolt shank to bolt hole as shown in Figs. 6 (a), (b) and (c). To implement the tangential and normal behaviour between the surfaces, penalty algorithm friction formulation and augmented Lagrange constraint enforcement method respectively are used.

The coefficient of friction is required in calculating tightening torques and resulting bolt tensile forces and stress and in calculating the resulting friction between the connected surfaces. The contact and



(a) (b)

Fig. 5 Finite element meshes for the angle and a bolt

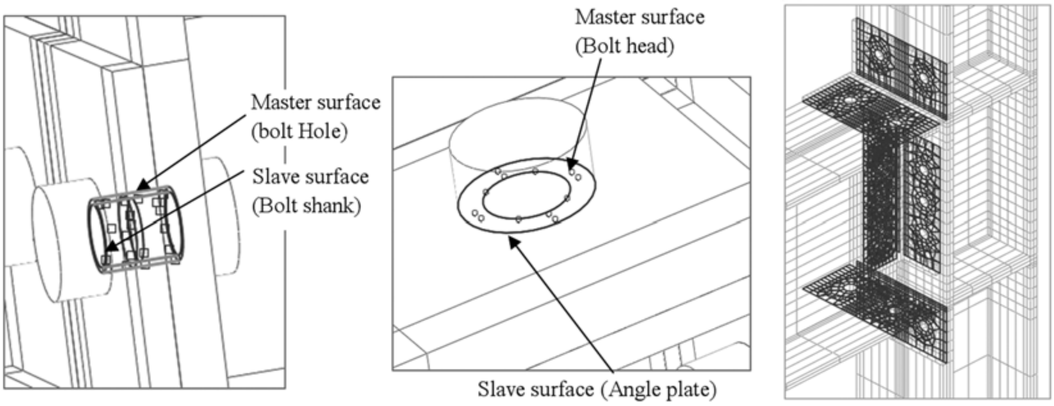


Fig. 6 Surface-to-surface contact between (a) bolt hole to bolt shank (b) bolt head to plate (c) plate to plate

bearing interactions between the bolt shanks and bolt holes are included in this study. From the literature, it was observed that the friction coefficients were much smaller than the statistical value of 0.33 suggested by test specimens with different bolt configurations. An average value of the cyclic friction coefficients measured from the cyclic tests was about 0.25. The cyclic slip behaviour was significantly different from the static slip behaviour. A few cycles of loading may change the clamping forces and the surface condition altering the slip resistance. Therefore the friction coefficient between the surfaces is adopted as 0.25.

3.2 Boundary condition and loading

For validation, the loads and boundary conditions applied to the numerical model replicated the experimental setup. The column for the numerical model was fixed at the base and the out of plane movement prevented as in the experimental setup. Prestressing forces are applied to the bolts as initial stresses to simulate the behaviour of HSFG bolts. A prestress force of 44.5 kN equivalent to a torque of 100 Nm is applied to each bolt initially by using bolt load option in ABAQUS. In subsequent steps, cyclic loads corresponding to those used during the experimental tests were applied at the top of the column. The bolt load variation according to the response of the frame is also included by using the option 'fix the bolt at its current length'.

3.3 Material modeling

A nonlinear combined isotropic/kinematic hardening model and isotropic hardening model are used in ABAQUS to simulate the behaviour of materials that are subjected to cyclic loading. The evolution law in these models consists of a kinematic hardening component (which describes the translation of the yield surface in stress space) and an isotropic hardening component (which describes the change of the elastic range). The Bauschinger effect is considered by this material model and the procedure to implement in a combined isotropic/kinematic sense is described in ABAQUS (2008). The nonlinear isotropic/kinematic hardening model is found to provide more accurate predictions. In this study, except for bolts all other component materials are modeled using nonlinear combined isotropic/kinematic hardening model.

Bolts are modeled with isotropic hardening material model as data for the isotropic/kinematic hardening model were not available for bolts. To incorporate these models, the hysteresis stress strain curve of material subjected to cyclic loading is required. The incorporated material model is verified by modeling two simple problems such as a cantilever beam (ATC-24 1992) and a rigid frame (tested in the literature (Calado 2003)) modeled using line elements and subjected to cyclic loading (Figs. 7(a) and 8(a)). The comparative load deflection hysteresis loops of the cantilever beam model and rigid frame model are shown in Figs. 7(b) and 8(b) respectively.

It can be seen that the hysteresis curve of cantilever beam model (Fig. 7(b)) matches closely with the test results. In rigid frame model, the peak point of each loop (compression side) and the outer shape of the final cycle of the test results are compared with the numerical model. All the peak point of each hysteresis curve matches well but the stiffness reduction of the hysteresis curve does not match with the test results. In these models, the connection was considered fully rigid (which is not the reality). This may be the probable reason for the mismatch of the slope of hysteresis curves. Notwithstanding this, these two models successfully calibrate the material model adopted in the present study.

4. Finite element analysis results

Having calibrated the present numerical model successfully, three dimensional nonlinear finite element

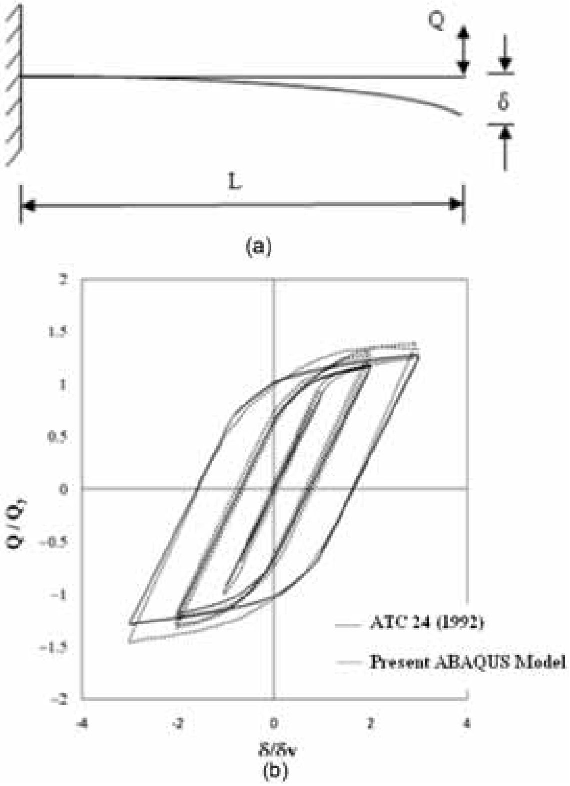


Fig. 7 (a) Steel cantilever beam specimen, (b) Load-deflection curve of cantilever beam

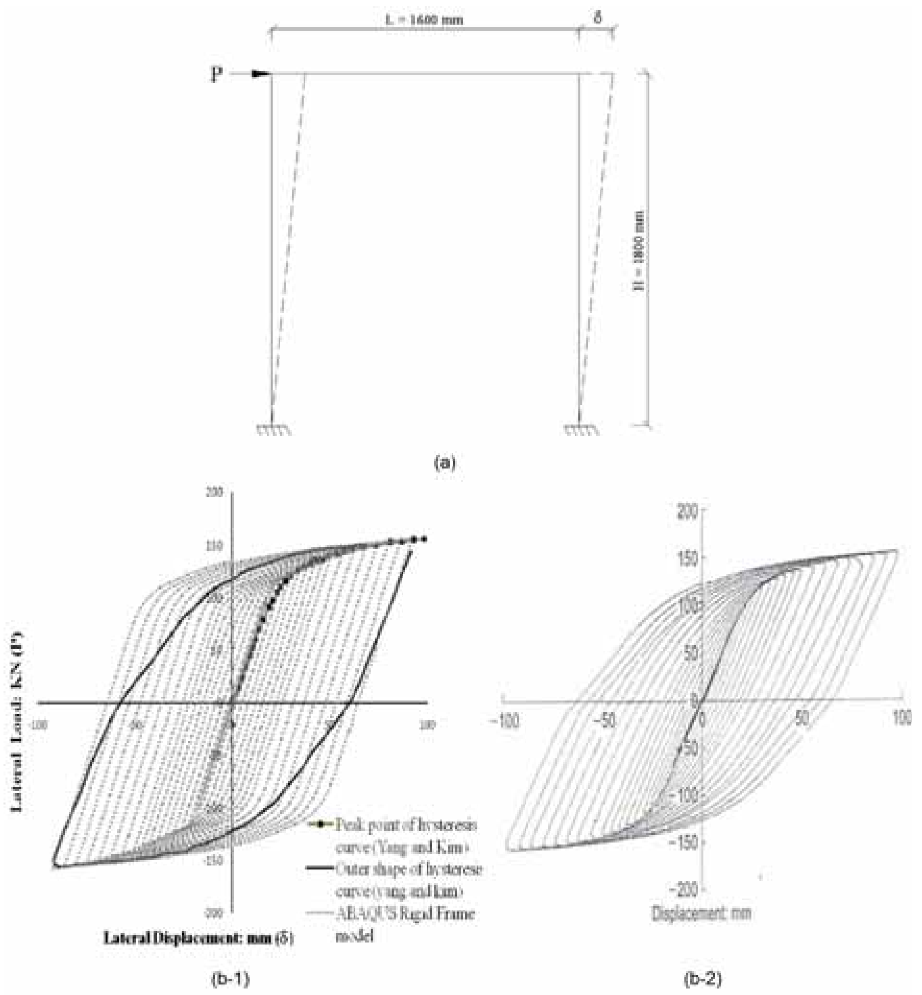


Fig. 8 (a) Rigid frame, (b) Load-deflection curve of rigid frame (b-1) Comparison of model and test results (b-2) Test result by Yang and Kim (2007)

analyses have been carried out on the steel frame model using ABAQUS. The behaviour of connection components and their failure are evaluated by observing Von Mises stress contour of the model. From the Von-Mises stress contour, it can be stated that the top angle nearer to the application of load showed a failure mode which is as observed in the experimental investigation. The hysteresis load-deflection curve of the frame and hysteresis moment rotation curve of connection are plotted (in Fig. 9) and from the curve, the following results are discussed.

The primary response quantity used for model calibrations are the hysteresis load-displacement history of the frame and hysteresis moment rotation behaviour of the connection. Predicted and measured load-displacement histories of the frame and moment rotation behaviour of connection as obtained from the numerical model and corresponding experimental values for the selected connection details are illustrated in Figs. 9 and 10 respectively.

It can be seen that the results of the present model agree well with the test results. The shape of the

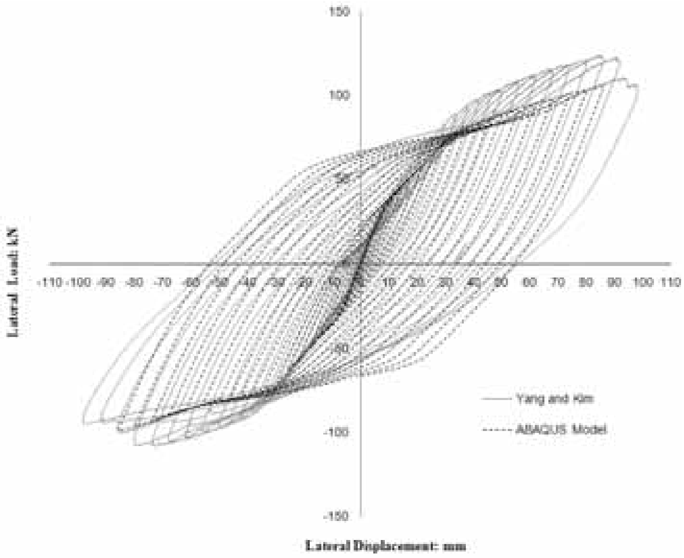


Fig. 9 Cyclic load-lateral displacement curve

hysteresis loops match exactly with test results, but the peak point at each cycle values do not match. A probable reason for this mismatch may be that the model uses only the average value of the yield and ultimate strengths for all the members. Similar to the experimental results the strength degradation in the model also is tri-linear. Some of the key parameters such as initial frame stiffness, initial connection stiffness, and yield and ultimate states are compared with test results as shown in Tables 2 and 3. The maximum percentage variations of the results are about 10%; however it is possible to attain more approximate values by refining the model.

The formations of yield line at the critical region in the test as well as in the numerical study are

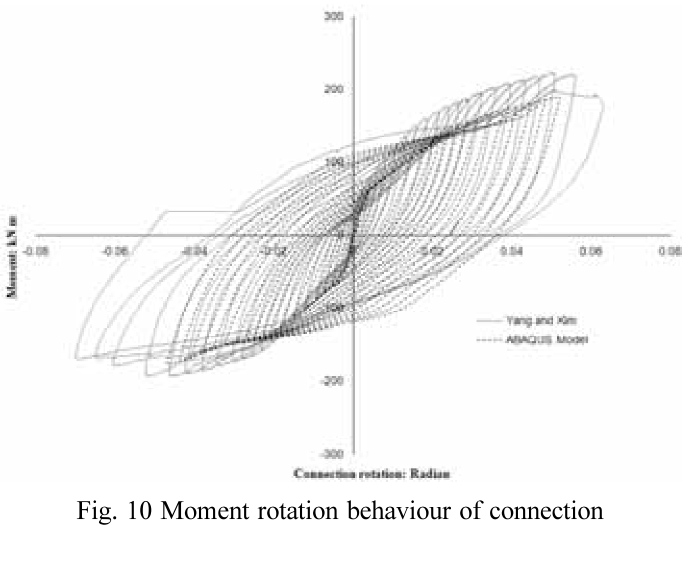


Fig. 10 Moment rotation behaviour of connection

Table 2 Comparison of test and model load deflection behaviour

Key parameters		Yang and Kim (2007)	Model results
Yield state at 22 mm deflection	Load: kN	63.60	58.53
Storey drift ratio 3% radian	Load: kN	99.10	88.59
Ultimate state	Load: kN	113.30	105.23
Initial frame stiffness : kN mm		4.30	4.00

Table 3 Comparison of test and model moment rotation behaviour

Key parameters		Yang and Kim (2007)	Model results
Yield state	Moment: kN m	109.40	105.36
	Rotation: radian	0.01	0.01
Storey drift ratio 3% radian	Moment: kN m	170.40	159.46
	Rotation: radian	0.02	0.03
Ultimate state	Moment: kN m	194.70	189.40
	Rotation: radian	0.05	0.05
Initial connection stiffness: kN m/radian		30542.00	25000.00

shown in Fig. 11 which is found to match well. Three plastic hinges are formed as in the test: at bolt line, column base and top angle toe of the fillet. First plastic hinge formed at the bolt line of the top angle leg attached to the column flange is shown in Fig. 11. The final fracture of the top and seat angles was in the vicinity of the toe of the fillet attached to the beam flange as shown in Fig. 11.

The energy dissipated by the specimens was calculated as the sum of the areas enclosed by a cyclic load-displacement hysteretic loop and moment rotation hysteretic loop. Figs. 12 and 13 shows a plot of the energy dissipation, according to the cycles, for the frame and the also for moment resisting connection.

From the energy dissipation graph of the frame and the connection, it can be seen that the present model closely match with test results. The amount of variation was not significant for frames and

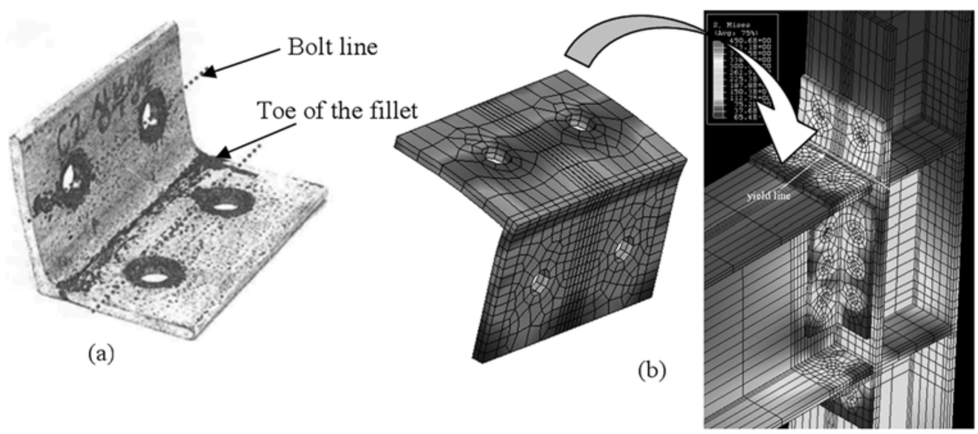


Fig. 11 Plastic hinges and fracture line of top angle: (a) Test result (b) Model result

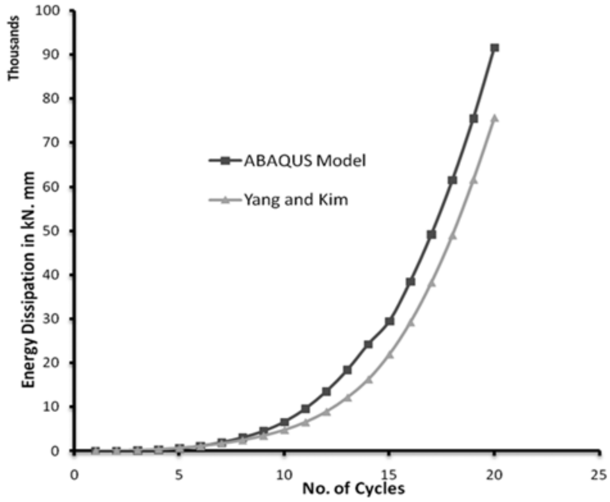


Fig. 12 Energy dissipation curve of frame

matches very closely in the case of connection as in Fig. 13. With these comparisons, it can be stated that the proposed finite element model predicts the behaviour of the steel frame with semi-rigid connection satisfactorily.

5. Studies on bolt behaviour

In a cyclic load analysis it is difficult to study the bolt slip phenomena and the bolt load variation. In order to understand this behaviour, the monotonic load up to a lateral deflection of 96 mm is applied at the end of the column. The predicted load-displacement behaviour of the frame and moment rotation

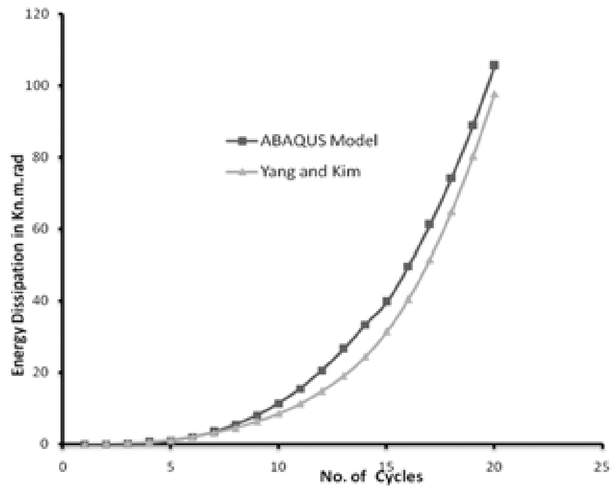


Fig. 13 Energy dissipation curve of connection

behaviour of connection as obtained from the numerical model and the test results plotted by joining the peak values of each cycle in the same direction of hysteresis curve for the selected connection details are illustrated in Fig. 14. Comparison between the model and test results is found to be very good. From the model results, the initial stiffness indicates that the force transfer mechanism is due to the friction between the bolt head and the component. When the load reaches the slip resistance limit of the bolted-angle 'A', slip occurs and the model loses its stiffness until the bolt bears against the bolt hole edge as shown in Fig. 15 and the stiffness recovery 'B' is as shown in Fig. 14.

It can be noted from the results of the above monotonic loading study that when a steel frame is subjected to external actions, the pretension force in the bolts varies according to the response of the frame. Variation of the bolt loads which subjected to the ultimate load during the lateral deflection of

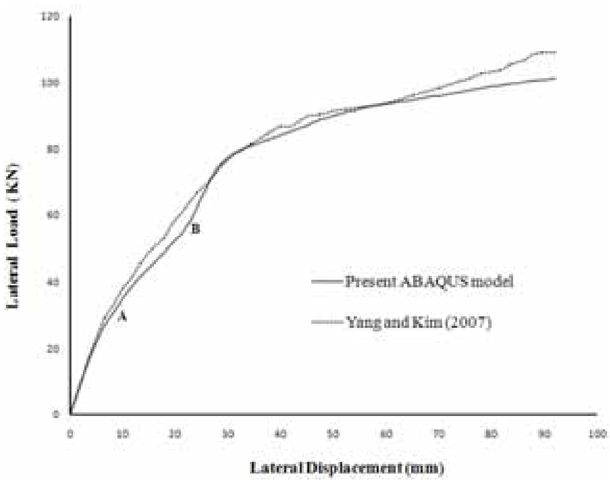


Fig. 14 Load-displacement curve

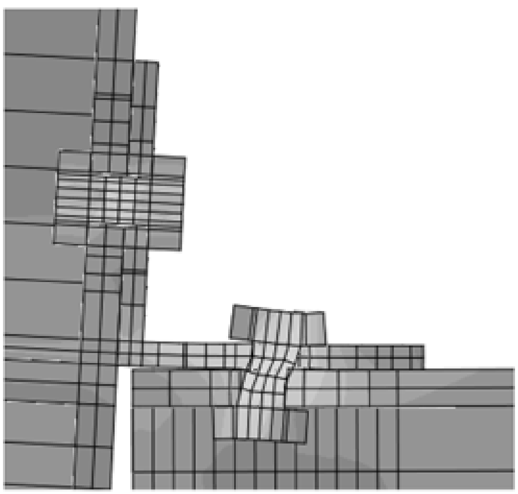


Fig. 15 Bolt slippage

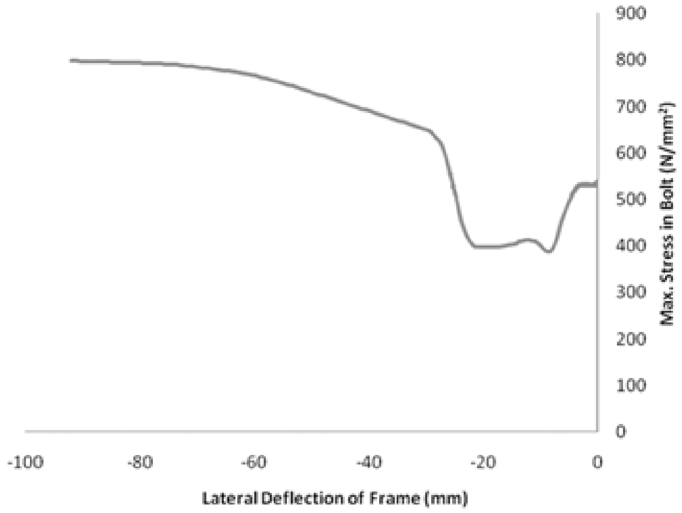


Fig. 16 Variation of bolt stress

frame is plotted in Fig. 16. Bolt load is measured at the center of the bolt. From the study it is observed that the bolt connecting top angle and beam flange of the connection which is near to the application of the load (Fig. 15) reaches the ultimate strength as in the experimental investigation. The inclusion of the above behaviour helps to predict the exact behaviour of bolted connection of steel frame under lateral loads.

6. Conclusion

A methodology for modeling of steel frames under cyclic loads with semi-rigid connection is described in this paper which takes into account the aspects such as material non-linearity, more particularly bolt slip and bolt load variations and the combined action of connections and frame behaviour. Based on a detailed 3D finite element model generated using ABAQUS, for an experimental prototype steel frame specimen tested under cyclic loads, this methodology has been proved to be reasonably efficient. The energy dissipation computed using the proposed methodology has been found to be reasonably close to the experimental results. The present study reveals that contact conditions including slip between all the connection components are of vital importance especially in semi-rigid connections. Modeling the force transfer mechanism during clamping of the components of the connection with bolts also makes it a pivotal issue for accurately modeling the bolt pretension and this has been studied in this paper. The inclusion of friction and slip in the model along with the simplicity of changing mesh geometry makes the present methodology a general approach for modeling a wide variety of bolted connections. Having validated the proposed finite element model by verification with experimental work, it is suggested that researchers may use the proposed methodology for study on the effect of controlling factors of semi-rigid connection like pitch distance, bolt diameter, pretension force, etc.

Acknowledgements

The authors gratefully acknowledge the support provided by the staffs of SERC and friends. The authors are indebted to Dr. C. M. Yang and Dr. Y. M. Kim for providing experimental data. Special thanks to Ms. Betty Macadam, Research Coordinator, ATLSS Center, LEHIGH University for the report received from them by e-mail. The paper is published with the kind permission of Director, SERC, Chennai.

References

ABAQUS Analysis Users Manual (2008), *Models for Metals Subjected to Cyclic Loading*, Hibbit, Karlson & Sorensen, Inc., pp. 18.2.2.

Adany, S. and Dunai, L. (2004), "Finite element simulation of the cyclic behaviour of end-plate joints", *Comput. Struct*., **82**(23-26), 2131-2143.

ATC (1992), "Guidelines for cyclic Seismic Testing of Components of Steel Structures", Report No.24, Applied Technology Council, Redwood City, CA.

Bjorhovde, R., Brozzetti, J. and Colson, A. (1990), "Classification System for Beam-to-Column Connections", *Struct. Eng*., **116**(11), 3059-3076.

Calado, L. (2003), "Non-linear cyclic model of top and seat with web angle for steel beam-to-column connections", *Eng. Struct*., **25**, 1189-1197.

Citipitioglu, A.M., Haj-Ali, R.M. and White, D.W. (2002), "Refined 3D finite element modeling of partiallyrestrained connections including slip", *J. Constr. Steel Res.*, **58**(5), 995-1013.

Cronvall, O. and Sarajärvi, U. (2006), *Simulation and Analysis of Data for Enhancing Low Cycle Fatigue Test Procedures*, VTT Technical Research Centre of Finland, NKS-126, ISBN 87-7893-188-6.

Dusicka, P., Itani, A.M. and Buckle, I.G. (2007), "Cyclic response of plate steels under large inelastic strains", *J. Constr. Steel Res*., **63**(2), 156-164.

Elnashai, A.S., Elghazouli, A.Y. and Denesh-Ashtiani, F.A. (1998), "Response of semi-rigid steel frames to cyclic and earthquake loads", *Struct. Eng.*, **124**(8), 857-867.

Hong, K., Yang, J.G. and Lee, S.K. (2002), "Moment rotation behavior of double angle connections subjected to shear load", *Eng. Struct.*, **24**(1), 125-132.

IS 800 (2007), *General construction in steel code of practice (Third revision)*, BIS, New Delhi.

Iványi, M. (2000), "Full-scale tests of steel frames with semi-rigid connections", *Eng. Struct.*, **22**(2), 168-179.

Kaufmann, E.J., Metrovich, B. and Pense, A.W. (2001), "Characterization of cyclic inelastic strain behaviour on properties of A572 Gr.50 and A913 Gr.50 Rolled Sections", Advanced Technology for Large Scale Structural Systems, ATLSS Report No.01-13, Lehigh University.

Nader, N. and Astaneh, A. (1992), "Proposed Code Provision for Seismic Design of Steel semi-rigid Frames", submitted to AISC Engineering Journal for review and publication.

Pucinotti, R. (2006), "Cyclic mechanical model of semi-rigid top and seat and double web angle connections", *Steel Compos. Struct*., **6**(2), 139-157.

Yang, C.M. and Kim, Y.M. (2007), "Cyclic behaviour of beam-column connections in steel portals", *Proc. of the Institution of Civil Engineers: Structures & Buildings*, **160**(SB5), 259-272.

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