I −100

ELASTO-PLASTIC ANALYSIS OF TOP- AND SEAT-ANGLE WITH DOUBLE WEB-ANGLE CONNECTIONS

N. Kishi	M. JSCE,	Muroran Institute of Tech.
K. G. Matsuoka	F. JSCE,	Muroran Institute of Tech.
M. Komuro	M. JSCE,	Muroran Institute of Tech.
Ali Ahmed	S.M. JSCE,	Muroran Institute of Tech.

1. INTRODUCTION

In the AISC-LRFD Specification (1994), steel beam-to-column connections are classified into three types of connections: rigid, flexible and semi-rigid. Fully rigid connection transfers moment without allowing any relative rotation between the connecting members, but ideally pinned connection allows rotation without producing any moment. The terminology semi-rigid connection is used to refer to those connections which provide intermediate rigidities between fully rigid and flexible connections. The former two connections are widely used in practical field and their detailed design guidelines are already established in all modern codes. On the other hand, practical application of semi-rigid connection still remains limited because of inadequate understanding and knowledge on moment-rotation behavior of semi-rigid connections. In the recent decades, the subject on semi-rigid connection and its behavior under monotonic and cyclic loading has received a great amount of attention to the researchers with the prospect of establishing more realistic design, ensuring better structural safety and achieving potential economy.

In engineering practice, there are several steel beam-to-column connections which can be categorized as semi-rigid connection. One of those, popular in practice, is top- and seat-angle with double web-angle connection. It received a keen attention to many because this connection can be a good candidate for semi-rigid connection and it is relatively simple to design and to fabricate.

In pursuance of predicting moment-rotation characteristics of top- and seat-angle with double web-angle connections, a good amount of experimental and analytical works had been done so far. Experimental works have been used to examine the validity of analytical works. The main approach found in the analytical works is that the researchers came out with a set of mathematical formulations which are capable of linking connection details with the moment-rotation curves of the connection (Frye and Morris 1975, Kishi and Chen, 1990 etc.). However, the use of such prediction models is limited to monotonic loading. With the background of seismic loading problems, finite element technique can be a good alternative approach to pursue the problem.

This study is aimed to develop finite element (FE) methodology for predicting moment-rotation behavior of top- and seat-angle with double web-angle connections. As a preliminary step, the present study is kept limited for monotonic loading. The validity of the finite element method is examined by comparing with the experimental results and a three-parameter power model (Kishi and Chen, 1990).

2. TOP- AND SEAT-ANGLE WITH DOUBLE WEB-ANGLE CONNECTION

Top- and seat-angle connection connects a beam to a column using four angles. Two of which are web-angles bolted to beam web and column flange. The other two are top- and seat-angles located above and below the beam flanges. These angles are also bolted to beam and column flanges. A typical connection is shown in Fig. 1.

Azizinzmini et al. (1985) conducted a series of tests in order to investigate the effect of different geometric parameters on the behavior of connection under monotonic and cyclic loading. The parameters investigated by Azizinamini et al. (1985) were the thickness of flange angle $t_t=t_s$, depth of beam $D_b=l_u+l_p+l_l$, length of flange angle $l_t=l_s$, gauge of flange angle $g_t'=g_s'$, length of web angle l_p, thickness of web angle t_a and bolt diameter d_{bo}. The relevant data of the test specimens are listed in Table 1. ASTM A36 steel was used for the beam, column and angles. Their yield and ultimate stresses, and strain-hardenning modulus are taken as 40.65, 68.425 and 139 ksi, respectively. A325 steel is used for bolt and its yield and ultimate stresses are taken as 100 and 150 ksi, respectively. Twelve static testing models with various connection parameters are used for FEM analysis.

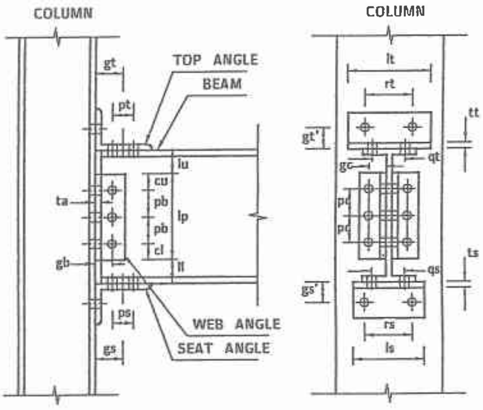


Fig. 1. A typical top- and seat-angle with double web-angle connection

3. FINITE ELEMENT MODEL SELECTION

Primarily four three-dimensional finite element models are set with the ABAQUS code in order to simulate the stiffness and strength pattern of the proposed models. The connection is modeled by using C3D8 brick elements. The material behavior of all connection components was represented by a bilinear stress-train curve. Large displacement phenomena and development of plastification zones are taken into account in the analysis. The four models can be characterized as follows:

(1) The first model is shown in Fig. 2(a). In this model, all bolts are considered to provide support the angles monolithically acting with the beam/column flange/web. Therefore, in designing the mesh, all bolts are considered as a part of the monolithic support and are not represented in the mesh. Contact interaction areas for the flange angles have an width equal to angle length $l_t=l_s$ and a length equal to $(g-w/2)$ of lines between column flange. Here, g is the gauge distance showing the distance from the bolt center line to the point of angle heel; and w is the width of bolt head. Similar interaction area for web-angles is also assumed.

(2) Likewise to the first model, all bolts are also assumed to support the angles monolithically acting with beam/column flange/web. The difference between the first and second models is that in the latter interaction area starts from bolt center line. That is the contact interaction areas for the flange angles have an width equal to angle length $l_t=l_s$ or l_p and a length equal to g of lines between column flange (Fig. 2(b)).

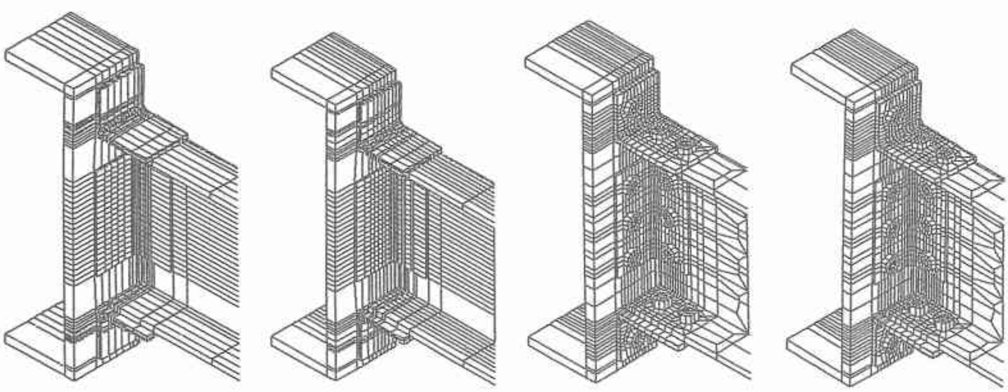
(3) The mesh pattern of the third model is shown in Fig. 2(c). The connection is represented by all major connection components: angles, beam, column and bolts. The bolts are represented in details such as: bolt shank, head and nut. The bolts in top- and seat-angles are assumed to behave into two halves: top-half and bottom-half. The top-half is considered to be a monolithic part of angle while the other half is

Table 1. Connection Geometry of Tests Used in the Analysis

Speci-men ID	Beam section	Top and seat flange angles				Web angles	
		Angle section	Length, '$l_t=l_s$' (inches)	Gage in leg on column flange, '$g'_t=g'_s$' (inches)	Bolt spacing in leg on column flange, '$r_t=r_s$' (inches)	Angle section	Length, 'l_p' (inches)
3/4 – inch diameter bolts							
14S1	W14×38	L6×4×3/8	8	2½	5½	2L4×3½×¼	8½
14S2	W14×38	L6×4×½	8	2½	5½	2L4×3½×¼	8½
14S3	W14×38	L6×4×3/8	8	2½	5½	2L4×3½×¼	5½
14S4	W14×38	L6×4×3/8	8	2½	5½	2L4×3½×3/8	8½
8S1	W8×21	L6×3½×5/16	6	2	3½	2L4×3½×¼	5½
8S4	W8×21	L6×3½×3/8	6	4½	3½	2L4×3½×¼	5½
8S5	W8×21	L6×3½×5/16	8	2½	5½	2L4×3½×¼	5½
8S6	W8×21	L6×4×5/16	6	2½	3½	2L4×3½×¼	5½
8S7	W8×21	L6×4×3/8	6	2½	3½	2L4×3½×¼	5½
7/8 – inch diameter bolts							
14S8	W14×38	L6×4×5/8	8	2½	5½	2L4×3½×¼	8½
14S9	W14×38	L6×4×½	8	2½	5½	2L4×3½×¼	8½
8S8	W8×21	L6×3½×5/16	6	2	3½	2L4×3½×¼	5½

assumed to be a part of flange. Similarly, bolts of web-angles also constitute of two parts: they belong to angles and beam web/column flange, respectively.

(4) Figure 2(d) shows the mesh pattern of the fourth model. Similar to the third model, this model also represents the connection with bolt details (shank, head and nut). But in this model, the bolts are assumed to interact with the angles and flange/web and completely independent from angle/flange/web.



(a) First model (b) Second model (c) Third model (d) Fourth model

Fig. 2(a)–(d). Mesh patterns of FE models

The comparison of performances of the above mentioned four models in terms of moment-rotation behavior is shown in Fig. 3. The values of initial connection stiffness and ultimate moment capacity corresponding to this figure are listed in Table 2. It is obvious from that among the four models, the fourth model performs best in predicting moment-rotation behavior of the connection.

Table 2. Pertinent data of Fig. 2.

Model ID or Test	Initial connection stiffness		Ultimate moment	
	Quantity (kip-in./rad.)	Difference (%)	Quantity (kip-in.)	Difference (%)
1st model	0.6161×10^6	+31.76	1730.6	+70.51
2nd model	0.3417×10^6	−26.92	1108.8	+9.25
3rd model	0.3542×10^6	−24.25	1088.1	+7.21
4th model	0.3527×10^6	−24.57	987.8	−2.67
Test	0.4676×10^6	–	1014.9	–

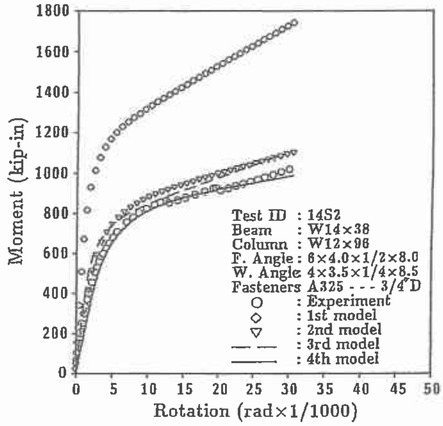


Fig. 3. Comparison among models

The mesh arrangements of the first model is simple but can not contain all the essential characteristics and components of connection. On the other hand, the second, third and fourth model represent the real situation of connection assemblages with better accuracy. The fourth model proves that it best represents the real interactions among the connection components. With this background, the fourth model is finally chosen for the present study.

4. THE THREE-PARAMETER POWER MODEL

Similar to Colson and Louveau (1983) and based on Richard and Abott's (1975) stress-strain formula, Kishi and Chen (1990) proposed a three-parameter power model disregarding the strain-hardening stiffness. Kishi-Chen power model containing three parameters: initial connection stiffness R_{ki}; ultimate moment capacity M_u; and shape parameter n; has a following form:

$$M=\frac{R_{ki}\theta_r}{[1+(\theta_r/\theta_0)^n]^{1/n}} \tag{1}$$

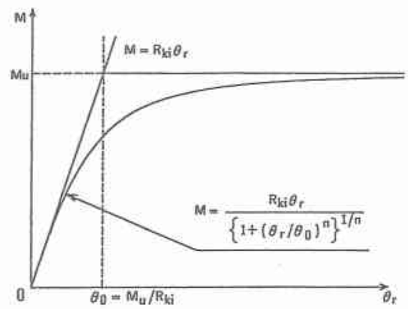


Fig. 4. Three-parameter power model

where M and θ_r are moment and relative rotation in connection, respectively. General shape of moment-rotation curves of eq. (1) is shown in Figure 4 with a positive real number of shape parameter n. In one extreme, if the n is taken to be infinity, the model reduces to a bilinear curve with the initial connection stiffness R_{ki} and the ultimate moment capacity M_u.

5. RESULTS AND COMPARISON

In order to discuss on applicability of FEM analysis comparing among experimental results and the power model, the values of relative rotation are evaluated as:

$$\theta_r=\frac{\delta_t-\delta_b}{D_b+t_f} \tag{2}$$

where D_b is the depth of beam, $t_f=t_t=t_s$ is the thickness of flange angle; δ_t and δ_b are the horizontal displace-

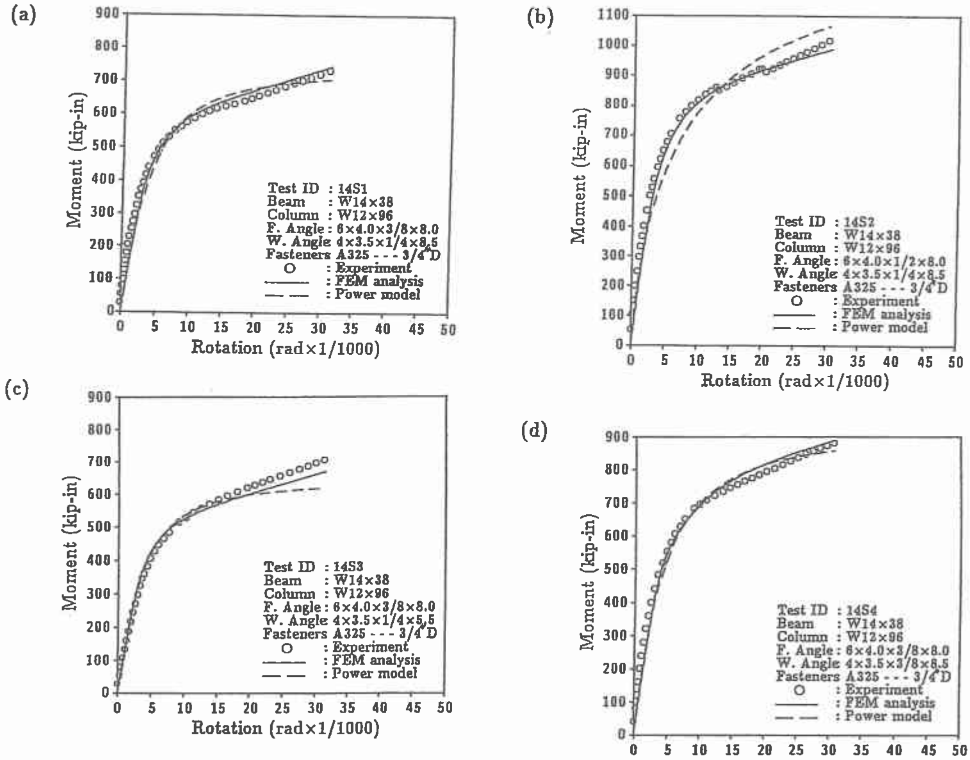


Fig. 5(a)–(d). Performance of FEM and power models

ments at the upper and lower edges of beam flanges, respectively. The connection moment M is evaluated multiplying applied force and distance from loading point to the center of rotation. The $M-\theta_r$ curves obtain from FEM analysis together with Kishi-Chen power model and experimental data are shown in Figs 5(a)–(d). Analytical values of initial connection stiffness and ultimate moment capacity are listed in Table 3. The figures show a good match among those three results. From Table 3, it is evident that with reference to ultimate moment capacity, maximum error of each FEM analysis ranges from –15.96% to 10.86%, and the comparison also shows that power model has the ability to predict moment-rotation relation of top- and seat-angle with double web-angle connection satisfactorily.

6. CONCLUSION

This study is primarily focused on finding a suitable predicting technique for moment-rotation behavior of top- and seat-angle with double web-angle connections under monotonic loading. In search of that kinds of approach, FEM analysis technique has been examined. Comparison between FEM results and experiment results reveals that FEM can be a variable approach in establishing semi-rigid frame analysis and design. The promising prospect of FEM approach can be highlighted from the fact that it may be a better alternative from the conventional approaches for cyclic loading study.

This study further investigated the performance of an analytical model (Kishi and Chen power model). It is seen that as far as monotonic loading is concerned it can be a good choice for designer to use this model for estimating of moment-rotation relation of top- and seat-angle with double web-angle connection.

Table 3. Comparison of predicted initial connection stiffness and ultimate moment with experimental results

Test ID	Initial connection stiffness (kip-in./rad.)			Ultimate moment (kip-in.)		
	Experiment	Power model	FEM	Experiment	Power model	FEM
14S1	0.2814×10^6	0.1448×10^6	0.2661×10^6	727.6	734.3	741.6
14S2	0.4676×10^6	0.3527×10^6	0.3472×10^6	1014.9	1510.2	987.8
14S3	0.1266×10^6	0.1392×10^6	0.1157×10^6	703.4	640.2	668.9
14S4	0.2227×10^6	0.1873×10^6	0.3593×10^6	880.0	929.2	889.5
14S8	0.5845×10^6	0.8712×10^6	0.2581×10^6	1624.4	1657.5	1574.9
14S9	0.2457×10^6	0.4177×10^6	0.1953×10^6	1076.1	1189.8	1082.7
8S1	0.7392×10^5	0.5456×10^5	0.4620×10^5	379.4	348.6	341.2
8S4	0.1662×10^5	0.6844×10^4	0.2155×10^5	188.8	187.7	209.3
8S5	0.9130×10^5	0.4906×10^5	0.4202×10^5	376.1	388.1	388.6
8S6	0.6003×10^5	0.2210×10^5	0.3414×10^5	287.8	244.1	276.7
8S7	0.4381×10^5	0.3766×10^5	0.4041×10^5	411.6	319.5	348.2
8S8	0.6218×10^5	0.6846×10^5	0.5146×10^5	438.1	367.3	368.2

7. REFERENCES

(1) Azizinzmini, A., Bradburn, J.H. and Radziminski, J.B., "Static and cyclic behavior of semi-rigid steel beam-column connections," Structural research studies, Department of Civil Engineering, University of South Carolina, Columbia, S.C., March 1985.

(2) Kishi, N. and Chen, W.F. (1990), "Moment-rotation relations of semi-rigid connections with angles," Journal of Structural Division, ASCE, Vol. 116, No. ST7, 1813−1834.

(3) Frye, M.J. and Morris, G.A. (1975), "Analysis of flexibly connected steel frames," Canadian J. Civ. Engrg., Vol. 2, No. 3, 280−291.

(4) Richard, R.M. and Abott, B.J. (1975), "Versatile elastic-plastic stress-strain formula," Journal of the Engineering Mechanics, ASCE, Vol. 101, No. EM4, 511−515.

(5) Colson, A. and Louveau, J.M. (1983), "Connection incidence on the inelastic behavior of steel structures," Euromech Colloquium 174.

(6) Sherbourne, A.N. and Bahaari, M.R. (1994), "3D simulation of end-plate connections," J., Structural Engineering, AISC, 120(11), 3122−3136.

(7) Kshi, N., and Chen, W.F. (1986), "Data base of steel beam-to-column connections," Structural Engineering Report No. CE-STR-86-26, School of Civil Engineering, Purdue University, West Lafayette, IN.

(8) American Institute of Steel Construction, (1994), "Manual of Steel Construction, Load and Resistance Factored Design," AISC, Chicago. 1(1), 61−64.

(9) ABAQUS / STANDARD—Theory Manual, Vertion 5.7., Hibbitt Karlsson & Sorensen, Inc. 1997.

(10) ABAQUS / STANDARD—User's Manual, Vertion 5.7., Vol. I, Vol. II and Vol. III, Hibbitt Karlsson & Sorensen, Inc. 1997.