I －12　REVISION OF SEMI-RIGID CONNECTION DATA BASE

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1. INTRODUCTION

The concept of semi-rigidity arose from the fact that all sorts of real connections display some sort of flexibility ranging from low to high. The ideally pinned connection allows rotation without producing any moment while the fully rigid connection transfers moment without allowing any relative rotation. All the real connections lies in between these two extreme cases. Thus semi-rigidity of connections drew attention to many researchers for last several decades. Recognizing the semi-rigidity of connections, American Institute of Steel Construction also permitted the incorporation of partially restrained connection in frame design in its LRFD specification. But very few guidelines have been laid for this incorporation. Researchers tried to develop prediction models for connections in different times either empirically or analytically. Whatever may be the approach, analytical, empirical or semi-empirical, these proposed models should be justified by making comparison with real experimental tests data. The first step to establish rational methods for semi-rigid frame design is to compile a data base of moment-rotation characteristics for steel connections, and then develop a practical means to model the moment-rotation behavior of semi-rigid connection. Thus compilation of a comprehensive database on moment-rotation response of connections, no doubt, carries an unlimited importance.

This updated database is a result of expansion of Kishi-Chen's (1986) data base. Therefore this compilation encompasses a period from 1936 to till date. Total number of experimental tests on moment-rotation response of steel connections is 382. The experimental moment-rotation curves are also compared with Fry-Morris polynomial model, Kishi-Chen's modified exponential and power model in all possible cases.

Table 1 Connection Types

1	Single Web-Angle Connections & Single Plate Connections.
2	Double Web-Angle Connections.
3	Top- and Seat-Angle Connections with Double Web-Angle.
4	Top- and Seat-Angle Connections.
5	Extended End-Plate Connections.
6	Flush End-Plate Connections.
7	Header Plate Connections.

2. GENERIC CLASSIFICATION OF CONNECTION TYPE

Traditionally, many connection types have been developed, designed and detailed by the steel fabricators. There is an unfortunate lack of consensus concerning connection nomenclature. For instance, 'double web-angle connection' are frequently named as 'web cleat connection', 'framed beam connection' and 'standard connection. 'Top-and seat-angle connection' has been variously referred to as 'seated beam connection', 'cap and seat angle connection' and 'flange cleat connection', among others. To keep the compilation effort manageable, disregarding the minute geometric variability, the total experimental tests collected are grouped into seven main groups (Table 1).

3. OUTLINE OF THE REVISED DATA-BASE

The eventual aim of the data base compilation is to make available information regarding moment-rotation response of connections at the disposal of researchers and in turn it will be in assistance in the development of design requirements as well as in the formulation of prediction models. Sufficient informations are tried to extract from collected literatures. Accessing to the data base through a computer network is made by using Kishi-Chen's Steel Connection Data Bank (SCDB) Program.

Difficulties are faced with the problem of variability of reporting manner by different researchers. Some publications do not report sufficient details for the results, for instance, connection fabrication details or ultimate stress are often omitted. Thus, unreported informations are disregarded. Moment-rotation data has rarely been presented in an explicit form by the investigators. In most cases $M-\theta_r$ curves are reported in graphical form rather than in tabulated form. Thus, the reported moment-rotation curves are digitized. The test data has been incorporated in this data base without changing the identification number used by the original investigators. Mainly following informations are tried to collect and incorporate in this data base: (a) connection type, (b) source of the test, (c) physical, material and geometrical characteristics of connection components, (d) presence or absence of column stiffener, (e) presence or absence of bolt-tensioning, and (f) moment-rotation test data. Collected experimental tests are listed in Table 2.

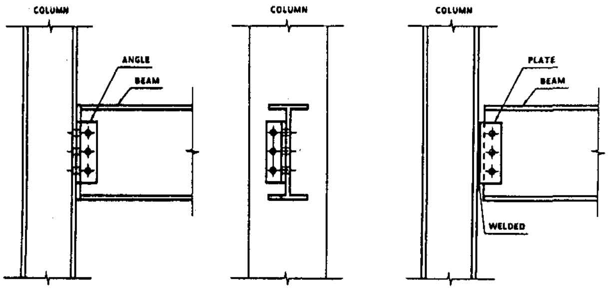


Fig. 1 (a) Single Web-Angle (b) Single Plate

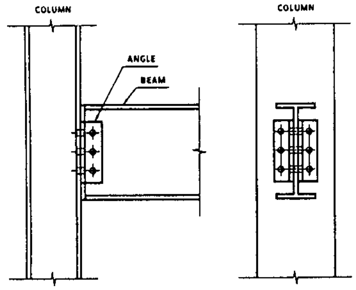


Fig. 2 Double Web-Angle

3.1. Single Web-Angle Connections and Single Plate Connections

A single web-angle connection consists of an angle either bolted or welded to both the column and the beam web as shown in Fig. 1(a). On the other hand, in case of single plate connections a single plate connection is used in place of the angle as shown in Fig. 1(b). This connection requires less material than a single web-angle connection.

Major geometric parameters which influence single web-angle and/or plate connection behavior have been identified as: (i) number of beam web bolts, (ii) method of fastener tightening, (iii) angle/plate thickness and depth, (iv) column flange (or web) thickness.

54 sets of experimental tests have been incorporated in this data base.

3.2. Double Web-Angle Connections

Double web-angle connections consist of two angles either bolted or riveted to both the column and the beam web as shown in Fig. 2.

The connection rigidity of the double web-angle connections is stiffer than that of the single web-angle and single plate connections.

Major geometric parameters which influence connection behavior have been identified as: (i) number of beam web bolts, (ii) method of fastener tightening, (iii) angle thickness and depth, (iv) column flange (or web) thickness, (v) gauge distance of column bolts, (vi) location of angle-column fillet welds.

All total 82 number of tests are collected.

Table 2 Collected Experimental Tests

Connection type	Reference (author, year)	Number of tests
Single web-angle and Single plate connections	S. L. Lipson (1968)	30
	L. E. Thompson et al. (1970)	12
	S. L. Lipson (1977)	8
	R. M. Richard et al. (1982)	4
Double web-angle connections	J. C. Rathbun (1936)	7
	W. C. Bell et al. (1958)	4
	C. W. Lewitt et al. (1966)	6
	W. H. Sommer (1969)	4
	L. E. Thompson et al. (1970)	48
	B. Bose (1981)	1
	J. B. Davison et al. (1987)	4
	A. K. Aggarwal (1990)	8
Top- and seat-angle connections with double web-angle	J. C. Rathbun (1936)	2
	A. Azizinamini et al. (1985)	20
Top- and seat-angle connections	J. C. Rathbun (1936)	3
	R. A. Hechtman et al. (1947)	12
	S. M. Maxwell et al. (1981)	12
	M. J. Marley (1982)	26
	J. B. Davison et al. (1987)	2
Extended end-plate connections	L. G. Johnson et al (1960)	1
	A. N. Sherbourne (1961)	5
	J. R. Bailey (1970)	26
	J. O. Surtees et al. (1970)	6
	J. A. Packer et al. (1977)	3
	S. A. Ioannides (1978)	6
	R. J. Dews (1979)	3
	P. Grundy et al. (1980)	2
	N. D. Johnstone et al. (1981)	8
	Y. L. Yee (1984)	16
	D. B. Moore et al. (1986)	2
	J. B. Davison (1987)	1
	R. Zandononnini et al. (1988)	9
	A. Mazroi (1990)	24
Flush end-plate connections	J. R. Ostrander (1970)	24
	J. B. Davison et al. (1987)	3
Header plate connections	W. H. Sommer (1969)	20
	J. B. Davison et al. (1987)	1
	A. K. Aggarwal (1990)	7

3.3. Top- and Seat-Angle Connections with Double Web-Angle

A typical top- and seat-angle connection with double web-angle is shown in Fig. 3. Generally top and seat-angle connections with or without double web-angle are said to be semi-rigid connection. Double web-angle are used to improve the connection restraint characteristics of top- and seat-angle connections.

Major parameters which influence connection behavior have been identified as: (i) thickness and length of angles, (ii) gauge distance of bolts in vertical angle leg, (iii) column flange (or web) thickness.

22 tests are included from two series.

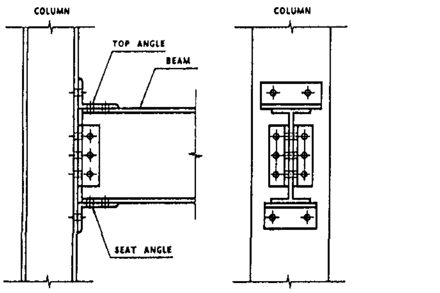


Fig. 3 Top- and Seat-Angle with Double Web-Angle

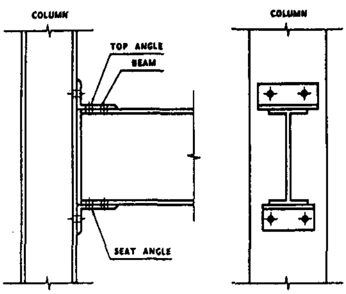


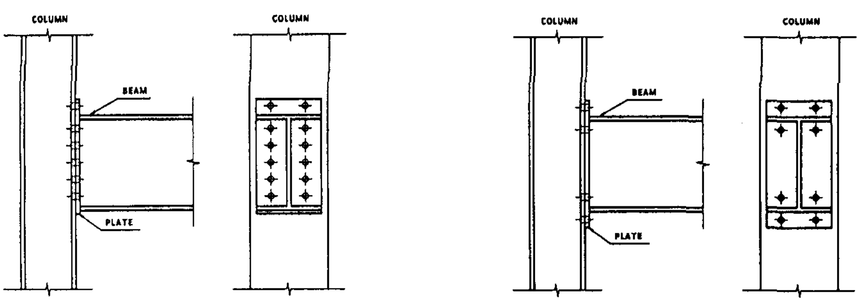
Fig. 4 Top- and Seat-Angle

3.4. Top- and Seat-Angle Connections

A typical top- and seat-angle connection is shown in Fig. 4. The AISC specification describes the top- and seat-angle connection as (a) The seat angle transfers only vertical reaction and should not give significant restraining moment on the end of the beam , (b) The top angle is merely for laterally stability and is not considered to carry any gravity loads.

Major geometric parameters which affect the moment-rotation behavior of connections are: (i) Number of beam flange bolts, (ii) Method of fastener tightening.

Total 55 tests are collected from five series.



(a) End-Plate on Tension Side Only (b) End-Plate on Both Sides

Fig. 5 Extended End-Plate

3.5. Extended End-Plate Connections and Flush End-Plate Connections

In general, the end plate connections are welded to the beam end along both the flanges and web in the fabricator's shop and bolted to the column in the field. The end-plate connections have been used extensively since 1960s. The end-plate connections are classified into two types as end-plate connection extended either on the tension side only or on both the tension and compression sides as shown in Fig. 5(a) and (b). A typical flush end-plate connection is shown in Fig. 6.

Major geometric parameters which influence the behavior of extended end-plate connection behavior have been identified as: (i) End plate thickness, (ii) Column flange thickness, (iii) Moment arm for column flange bolts.

On the other hand, parameters which affect the behavior of flush-end connection are listed as: (i) End-plate thickness, (ii) Gauge of column bolts, (iii) Moment arm for column flange bolts.

A good number of experimental tests for extended end-plate connections (114) have been collected. The number of collected tests for flush end-plate connections are 27.

3.6. Header Plate Connections

A header plate connection consists of an end plate, whose length is less than the depth of the beam, welded to the beam web and bolted to the column as shown in Fig. 7. The moment rotation characteristics of these connections are similar to those of double web-angle connections. Accordingly, a header plate connection is used mainly to transfer the reaction of the beam to the column.

28 tests from three series of tests are reported (Sommer (1969), Davison (1987) and Aggarwal (1990)).

Major geometric parameters which influence connection moment-rotation behavior are listed as: (i) Plate thickness, (ii) Plate depth, (iii) Gauge distance of column bolts.

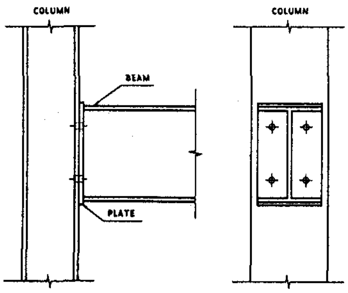


Fig. 6 Flush End-Plate

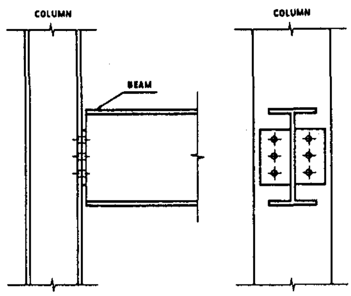


Fig. 7 Header Plate

4. COMPARISON WITH PREDICTION MODELS

Concurrent to the experimental investigation, modeling of moment-rotation characteristics were also attempted. In the Steel Connection Data Bank (SCDB) program, three prediction models are included. They are polynomial model of Frye and Morris (1975), modified exponential model of Kishi and Chen (1986) and power model of Kishi and Chen (1990). A succinct description of these models will be addressed here.

4.1. Polynomial Model

Frye and Morris (1975), developed a polynomial model which has an odd term polynomial to fit real moment-rotation curves. The model can be written in the following odd power polynomial form.

$$\theta_r = \sum C_i (KM)^i \tag{1}$$

where θ_r is the relative rotation, M is the moment , i takes values of 1,3,5 and so on, C_i are curve fitting constants and K is a standardization parameter. Polynomial model can reasonably ágrees with real moment-rotation curves but it has an inherent nature of producing negative stiffness which is physically unacceptable.

4.2. Power Model

Kishi and Chen (1990) proposed a power model for connections with angles using three parameters, initial elastic stiffness, ultimate moment and shape parameter. The initial stiffness and ultimate moment capacity of the connections are determined by simple analytical procedure. The shape parameter is then determined by the least-square curve fitting technique with experimental tests. This model can calculate moment, relative rotation and tangent stiffness of connection without any numerical iteration. The prediction equation is

$$\theta_r = \frac{M}{R_{ki}[1-(M/M_u)^n]^{1/n}} \qquad (2)$$

where R_{ki} is initial connection stiffness, M_u is the ultimate moment capacity and n is shape parameter. This model is not justified for the curves that do not flatten out near the final loadings.

4.3. Modified Exponential Model

Kishi and Chen (1986) refined Lui and Chen's exponential model to accommodate the linear components. They used exponent functions to fit experimental data.

$$M = M_0 + \sum_{j=1}^{m} C_j \cdot [1 - \exp(- \frac{|\theta_r|}{2j\alpha})] + \sum_{k=1}^{n} D_k \cdot (\theta_r - \theta_k) \cdot H[\theta_r - \theta_k] \qquad (3)$$

where M_0 is initial connection moment, α is scaling factor, C_j, D_k are curve fitting parameters, θ_k is starting rotation of k-th linear component, from experimental $M-\theta_r$ curve and H[ ] is Heavisides step function.

$$H[\theta] = 1 \quad \text{for } \theta \geq 0$$
$$H[\theta] = 0 \quad \text{for } \theta < 0$$

Comparison of experimental data, Frye-Morris polynomial, Kishi-Chen's modified exponential model and power model are conducted with the aid of SCDB (Steel Connection Data Bank program) in all possible cases. Figure 8 is a sample output obtained by executing SCDB program.

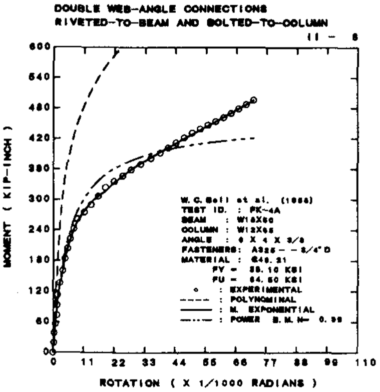


Fig. 8 Sample Out-Put Obtained by SCDB

5. CONCLUSION

The possibility of using a series of experimental tests to establish connection behavior has encouraged the authors to expand the existing data base. It is obvious that in order to obtain statically reliable results, a huge number of tests are required to be compiled. Therefore needless to say, expansion of this data-base is a never-ending demand unless and until semi-rigid behavior of connections is well understood.

6. REFERENCE

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