A component-based model for beam-column connections

P.J. Madas & A.S. Elnashai
Imperial College, London, UK

ABSTRACT: Recent experimental and analytical evidence confirm the feasibility of using semi-rigid connections for the design of steel frames subjected to strong ground motion. To progress this type of construction further, there is an urgent need for verifiable analytical models, including beam-to-column connection hysteretic response, to enrich and steer experimental programmes aimed at deriving design guidance. Most existing analytical representation of connections utilize experimentally-observed overall behavioural patterns, hence cannot be used in speculative analyses. In this paper a model is presented where no previous knowledge of the behaviour is needed. This is based on assembling the overall connection response from component contributions, such as seat and top angles, web-to-flange connections and panel zone, the response of each of which is obtained from mechanics principles. The assembled model is verified by comparison with monotonic and cyclic test data and shown to provide reasonable representation of the behaviour without the need for more than the geometric and strength parameters of the components.

1 INTRODUCTION

In conventional analysis and design of steel frames, beam-to-column connections are assumed to behave either as pinned or as fully rigid. However, experimental investigations show that flexible joints possess some ability to resist rotational deformations and that rigid connections exhibit some flexibility. Therefore, all connections may be classified, in principle, as semi-rigid.

Although provisions for flexibly-connected frames are contained in some codes (e.g. Eurocode 3), detailed and specific guidelines for the design of such frames are not yet available. Consequently, designers do not currently use this type of construction in seismic areas, because of difficulties associated with assessing its strength and deformational characteristics.

Recent experimental investigations (Nader and Astaneh-Asl, 1989) indicated that the optimum seismic design solution is not necessarily a rigidly-connected frame. Comparison of test results from rigid, semi-rigid and flexible steel frames confirmed the feasibility of semi-rigid frames for seismic design.

Furthermore, an extensive analytical investigation (Parra Rosales, 1991) highlighted the potential of semi-rigid and flexible steel frames designed to resist earthquake ground motion. In the latter investigation, two steel frames (2 and 4 storey) were designed to resist seismic loading using existing rigid frame criteria and assumed semi-rigid design concepts. The frames were analysed using the program ADAPTIC (Izzuddin and Elnashai, 1989) under high frequency (Loma Prieta, Gilroy) and low frequency (Loma Prieta, Emeryville) earthquake records. The results obtained clearly indicated that the semi-rigid solution is feasible; indeed more economical than the rigidly-connected frame.

Whilst the above investigations serve to underline the potential role of semi-rigid connections in earthquake resistant design of steel structures, the dearth of information on i) expected flexibility, ii) period estimation and iii) effective length of members, was also highlighted. It is therefore of immediate importance that comprehensive studies are undertaken to arrive at quantitative design guidelines for this type of connection and for their effect on the frame static and dynamic characteristics.

Whereas, the role of experimental testing in final confirmation of behavioural patterns cannot be over-emphasized, it is highly desirable to conduct extensive analytical studies to both enrich and steer experimental testing programmes. Having established the case for analytical modelling of semi-rigid frames subjected to earthquake loading, it remains to be mentioned that developments in hysteretic models for the connections clearly lag behind beam and beam-column modelling techniques as discussed below.

2 EXISTING MODELS

Experimental and analytical studies have been carried out, particularly in the last two decades, in order to define the moment-rotation (M-θ) relationship for different types of semi-rigid connections. Analytical models range from purely empirical curve fitting of test data (e.g. Moncarz and Gerstle, 1981 and Poggi and Zandonini, 1985), to extensive 3D finite element analysis (Kukreti et al., 1987). Although mathematical expressions fit closely virtually any shape of M-θ curve (Mazzolani, 1988), they suffer from the disadvantage that they can neither be extended outside the range of the calibration data nor can they be used for a different type of connection. On the other hand, finite element

methods provide accurate representation of the behaviour for the considered connection type, but their modelling is a time-consuming and costly procedure, especially under cyclic and dynamic loading.

Recognition of the above limitations instigated the development of the so-called mechanical models. This approach comprises simulation of the real behaviour of semi-rigid connections by an assembly of component contributions. The model accuracy and versatility is increased as the number of components taken into account increases. The advantage of such models is that they are comparatively easy to be implemented in frame analysis programs. The actual shape of the M-θ curve is obtained without need to constrain it to follow predetermined patterns, thus no calibration to experimental results is needed. However, although mechanical models for the behaviour of semi-rigid connections under monotonic loading have been developed in recent years (Wales and Rossow, 1983), the complex cyclic response has not been addressed. In this paper a mechanical model is presented for the assessment of the complete moment-rotation relationship of the semi-rigid connection types shown in Figure 1, under monotonic and cyclic loading conditions.

3 MODEL DESCRIPTION

The studied beam-column connection types (Figure 1) consist of the following (bolted or welded) elements: a) double web angles b) seat and/or top angles c) any combination of the above and d) partially welded flush end-plate. The following are the most significant deformable components that influence the moment-rotation behaviour: i) connecting elements (angles, cleats, end-plate), ii) bolts and fasteners, iii) column flange and iv) column web panel zone.
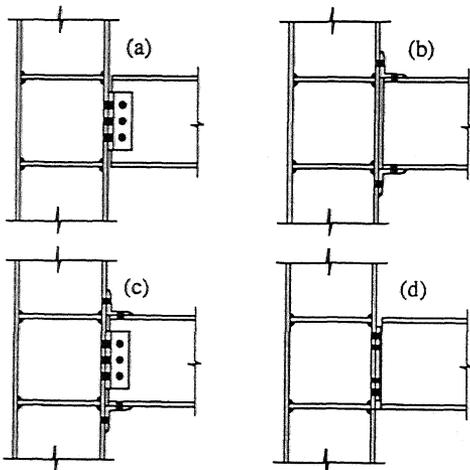

Figure 1. Considered connection types.

An idealization of the proposed model is shown in Figure 2, where points A and B represent the end of the

beam and the centroid of the column, respectively. The nonlinear spring sets C and D simulate the deformable connecting elements, while the column shear panel deformation is accounted for by nonlinear springs E.
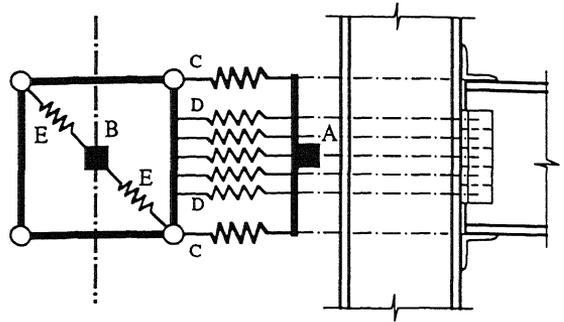

Figure 2. Idealization of beam-to-column connection.

Under the combined action of moment and axial force, the connection top and seat angles deform uniformly along their length (spring set C). However, web angles and partially welded flush end-plates undergo non-uniform deformations in the vertical direction. Therefore, series of springs D are included, each of which exhibits an independent force-deformation relationship.
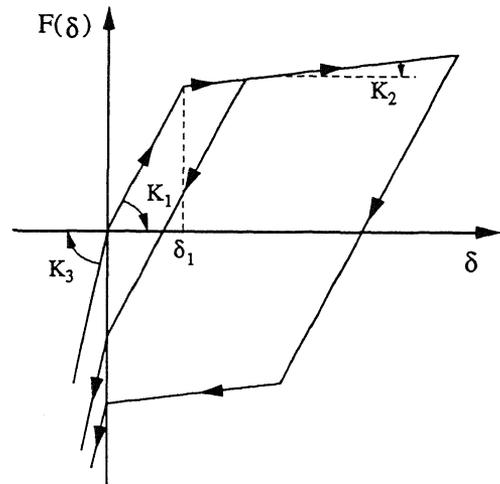

Figure 3. Component model for cyclic loading.

The nonlinear behaviour of spring sets C and D is idealized by a trilinear load-deformation relationship and further generalized for cyclic loading, as shown in Figure 3. The initial stiffness K_1 derived from elastic analysis of the deformable components (Figure 4), is given by the following expression:

$$K_1 = \cfrac{1}{\cfrac{1}{K_c} + \cfrac{1}{K_b} + \cfrac{1}{K_f}} \qquad (1)$$

where: K_c is the elastic stiffness of the connecting element (e.g. angle), K_b is the bolt stiffness and K_f is the stiffness of the column flange. The deformation δ_1 corresponds to the formation of a plastic hinge at the critical section. After plasticity, a hardening stiffness K_2 is employed. The above parameters define the behaviour of the components when deformation δ is tensile. However, when deformations become compressive, due to the presence of the comparatively rigid column, the stiffness is increased to K_3, as proposed by Wales and Rossow (1983).
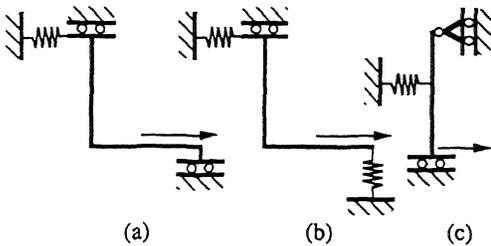

Figure 4. Idealized deformable components: a) double web angles, b) top/seat angles, and c) partially welded flush end-plate.
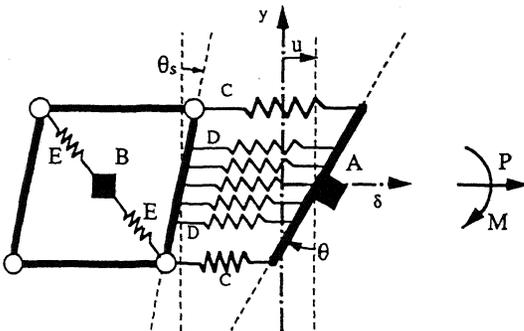

Figure 5. Deformed beam-to-column connection model.

Under a moment M and a force P applied to the connection (Figure 5), the end of the beam centreline (point A) undergoes a rotation θ and a displacement u, relative to point B. The deformational state of the spring sets C and D can be defined by two variables only, the axial deformation u and rotation $\theta_e = \theta - \theta_s$ (where θ_s is the rotation due to shear panel deformation). Thus, the deformation δ_i of a spring element i is given by the following relationship:

$$\delta_i = u + \theta_e \, y_i \qquad (2)$$

The force $F(\delta_i)$, which the ith element is subjected to, is given by the cyclic load-deformation curve of Figure 3. Finally, the total axial force P and moment M transmitted by n connecting elements is given by:

$$P = \sum_{i=0}^{i=n} F(\delta_i), \quad M = \sum_{i=0}^{i=n} F(\delta_i) \, y_i \qquad (3)$$

The nonlinear behaviour of the shear panel is represented by the trilinear cyclic model shown in Figure 6. Stiffnesses K_e, K_t and K_s alongside the rotations θ_1 and θ_2 are determined through the model of Krawinkler et al. (1971).
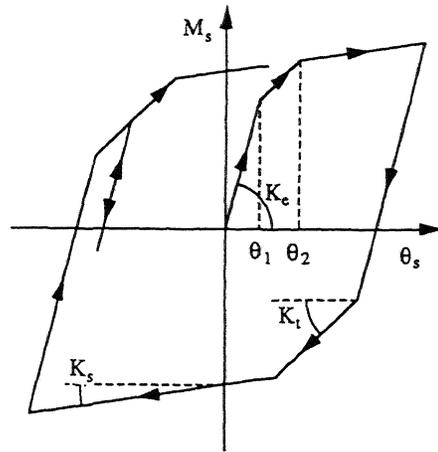

Figure 6. Shear panel model for cyclic loading.

The total rotation θ is the sum of the shear panel rotation θ_s and that of the connecting elements θ_e ; the ratio θ_e/θ_s is determined by equilibrium requirements. Whereas the shear panel rotational stiffness $K_{\theta s}$ is known a priori, the connecting elements' overall rotational stiffness $K_{\theta e}$ can only be defined after θ_e is determined. Therefore, an iterative procedure is used for the evaluation of θ_s and θ_e to satisfy rotation compatibility ($\theta = \theta_s + \theta_e$) and moment equilibrium ($M_s = M_e$), as shown in Figure 7. Finally, the rotational stiffness of the connection as a whole is given by:

$$K_\theta = \cfrac{1}{\cfrac{1}{K_{\theta s}} + \cfrac{1}{K_{\theta e}}} \qquad (4)$$

The cyclic model described herein is validated by comparison with experimental results (Madas and Elnashai, 1992). Hereafter, a sub-set of the examples considered is described.
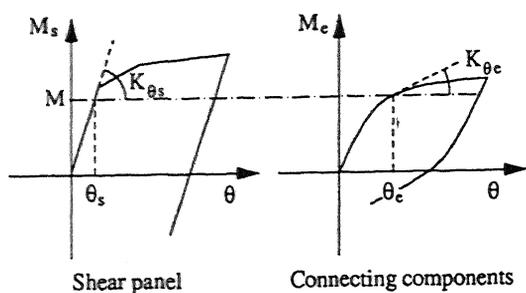
Figure 7. Coupling between shear panel and connecting components.

4 VERIFICATION ANALYSES

4.1 *Monotonic loading conditions*

Under monotonic loading the model is compared to experimental investigations of Radziminski and Azizinamini (1985) and Davison et al. (1987).

Figure 8 shows the comparison with three tests by Radziminski and Azizinamini (1985) on full scale connections consisting of bolted top and seat flange angles and double web angles. All three specimens were identical in terms of geometry and material properties, except for the thickness of the top and seat flange angles (15.9mm for 14S8, 12.7mm for 14S6 and 9.5mm for 14S5).
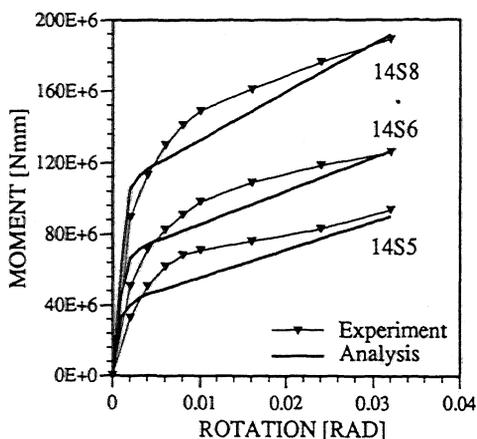

Figure 8. Comparison between experimental results by Radziminski et al. (1985) and predictions by the proposed model .

The experimental results by Davison et al. (1987) on four connections are compared to the prediction by the proposed model in Figure 9. All four specimens have the same beam and column sizes with a variable connection type. The connecting elements for JT/06 are double web angles, for JT/08 top and seat angles, for JT/10 seat and double web angles and for JT/14 partially welded flush end-plate (end-plate welded to beam web only).
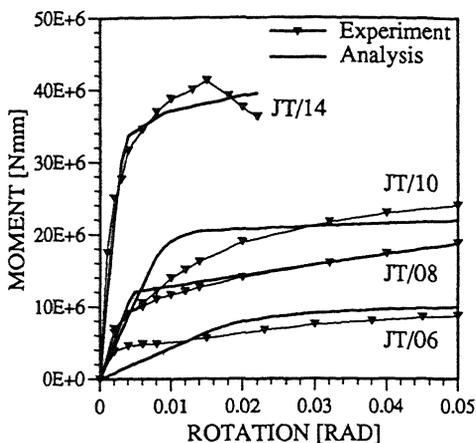

Figure 9. Comparison between experimental results by Davison et al. (1987) and predictions by the proposed model.

Both Figures 8 and 9 indicate good agreement between the analytical and experimental results for a variety of semi-rigid connections that are commonly utilized in practice. It is particularly noteworthy that for the first series of tests (Figure 8), although the analysis shows a definite transition from elasticity to plasticity, the initial rotational stiffness is closely followed and the overall moment capacity of the connection is underestimated by only a small margin.

Regarding the second set of experiments (Figure 9), despite the presence of a limited amount of slip during testing, the corresponding analysis prediction gives satisfactory results.

4.2 *Cyclic loading conditions*

The experimental work by Ballio et al. (1987) was used as a comparison basis for the validation of the model under cyclic loading. The experimental investigation comprised a number of tests on both welded and bolted connections between a cantilever beam and an axially loaded column. Herein, the behaviour of specimen B1, consisting of bolted top and seat flange angles and double web angles, is presented.

Figures 10 and 11 depict the experimental results and analytically-predicted force-deflection curves, respectively, when the free beam end is subjected to load reversals. It is observed that the analytical results obtained by the proposed model are close to the experiment. In particular, the overall behaviour is in both cases characterized by a stiffening effect (region corresponding to displacements 1.5 to 6 cm, as shown in Figure 11). This occurs during the closure of the gap between the column flange and the top (or seat) angle, when the latter is under compression. Moreover, the
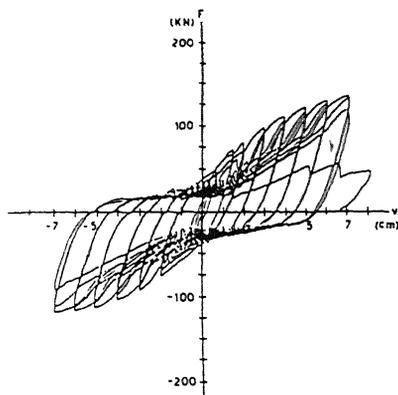
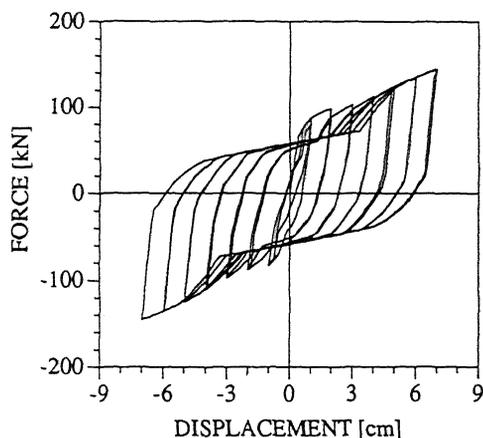Figure 10. Experimental results by Ballio et al. (1987) for test B1.

Figure 11. Predictions by the proposed cyclic model for test of Figure 10.

presence of a flat region (reduced moment transferred through the connection), that is observed predominantly during unloading, is attributed to the fact that neither the top nor the seat angle are in contact with the column face. However, the development of slip during testing causes the capacity of the connection to be 135 kN, whereas the corresponding analytical value is 145 kN (difference of 6.8%).

5 CONCLUSIONS

The objective of the present study was to develop an analytical model capable of predicting the behaviour of semi-rigid connections under monotonic and cyclic loading, without a priori knowledge of their moment-rotation skeleton curve. The adopted methodology is based on assembling the overall connection model from component contributions, such as angles, end-plate, bolts, column flanges and panel zone response. It is demonstrated that although the component models are

derived from simple mechanics principles, the complex overall behaviour of the connection is represented satisfactorily.

The developed techniques are currently the subject of further refinement and validation, with a view to investigating the comparative merits of various types of semi-rigid connections in seismic design of steel and composite frames.

REFERENCES

Ballio, G., Calado, L., De Martino, A., Faella, C. and Mazzolani, F.M. 1987. Cyclic behaviour of steel beam-to-column joints experimental research. *Estratto dalla Rivista Costruzioni Metalliche* 2: 3-24.

Davison, J.B., Kirby, P.A. and Nethercot, D.A. 1987. Rotational stiffness characteristics of steel beam-to-column connections. *Journal of Constructional Steel Research* 8: 17-54.

Izzuddin, B.A. and Elnashai, A.S., 1989. *ADAPTIC, a program for static and dynamic analysis of structures by adaptive mesh refinement; user manual.* Engineering Seismology and Earthquake Engineering Report ESEE/89-7.

Krawinkler, H., Bertero, V.V. and Popov, E.P. 1971. *Inelastic behaviour of steel beam-to-column subassemblages.* University of California, Berkeley, California, Report EERC/71-7.

Kukreti, A.R., Murray, T.M. and Abolmaali, A. 1987. End-plate connection moment-rotation relationship. *Journal of Constructional Steel Research* 8: 137-157.

Madas, P. and Elnashai, A.S. 1992. *A component-based model for cyclic analysis of steel and composite beam-column connections.* Imperial College, London, Report ESEE/92-3.

Mazzolani, F.M. 1988. Mathematical model for semi-rigid joints under cyclic loads. In *Connections in Steel Structures: Behaviour, Strength and Design.*(ed. R. Bjorhovde et al.). Elsevier Applied Science Publishers, London: 112-120.

Moncarz, P.D. and Gerstle, K.H. 1981. Steel frames with nonlinear connections. *Journal of Structural Division*, ASCE 107 (ST8): 1427-1441.

Nader, M.N. and Astaneh-Asl, A. 1989. *Experimental studies of a single storey steel structure with fixed, semi-rigid and flexible connections.* University of California, Berkeley, California, Report EERC/89-15.

Parra Rosales, J.G. 1991. *Seismic resistance of steel frames with semi-rigid connections.* MSc Dissertation, Engineering Seismology and Earthquake Engineering, Imperial College, London.

Poggi, C. and Zandonini, R. 1985. Behaviour and strength of steel frames with semi-rigid connections. In *Connection Flexibility and Steel Frame Behaviour* (ed. W.F. Chen). ASCE : 57-76.

Radziminski, J.B. and Azizinamini, A. 1985. Prediction of moment-rotation behaviour of semi-rigid beam-to-column connections. In *Connections in Steel Structures: Behaviour, Strength and Design.*(ed. R. Bjorhovde et al.). Elsevier Applied Science Publishers, London: 33-40.

Wales, M.W. and Rossow, E.C. 1983. Coupled moment-axial force behaviour in bolted joints. *Journal of Structural Division*, ASCE 109 (ST5): 1250-1266.