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DUCTILITY OF FRAMES WITH SEMI-RIGID CONNECTIONS

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ABSTRACT

In this paper, the seismic behaviour of frames with semi-rigid partial-strength connections is examined. Selected results from recent experimental investigations are presented and discussed. The experiments include monotonic, cyclic and pseudo-dynamic tests on two-storey steel frames with rigid and semi-rigid beam-to-column connections. Analytical simulations obtained from an advanced nonlinear analysis program are compared with experimental results. The computer program which incorporates detailed connection models is then used to investigate the effect of the type of connection on frame behaviour. It is shown from both the experimental and analytical studies that semi-rigid frames exhibit ductile and stable hysteretic behaviour and may provide an effective and reliable earthquake-resistant design solution.

KEYWORDS

Steel frames; connection behaviour; semi-rigid frames; beam-to-column connections; semi-rigid connections; pseudo-dynamic tests.

INTRODUCTION

For design purposes, beam-to-column connections are conventionally considered to be either pinned or fully rigid. This assumption simplifies significantly the design process on the expense of the realism and accuracy of the analysis particularly for connections of intermediate stiffness, referred to as semi-rigid. Modern codes of practice for static design such as Eurocode 3 (1990) and AISC (1986) now include some guidance on the design of frames incorporating this type of connection. The advantages of using semi-rigid connections, in terms of constructional economy and improved control on the structural behaviour, are being increasingly recognised. For seismic design, semi-rigid frames are not utilised mainly due to their relative flexibility. However, recent work by several researchers such as Astaneh *et al* (1989) indicated that the rigid frame is not necessarily the optimum solution since more flexible frames may attract lower inertial forces. Moreover, the performance of frames with fully-welded beam-to-column connections during recent earthquakes have cast doubts over the reliability of this type of rigid frame construction and directed attention towards other alternatives.

This paper presents selected results from recent experimental and analytical studies carried out to investigate the feasibility of semi-rigid frames in comparison with fully-welded frames. The studies include monotonic, cyclic and pseudo-dynamic tests on two-storey rigid and semi-rigid frames (Takanashi *et al*, 1993; Elnashai and Elghazouli, 1993). Comparison between the experimental results and analytical simulations are also presented. Use was made of the nonlinear dynamic analysis program ADAPTIC (Izzuddin and Elnashai, 1989) which includes a non-linear cyclic component-based model for the connection (Madas and Elnashai, 1992). The analytical model is also used to investigate, through simple examples, the effect of connection type on the seismic response of steel members and frames.

EXPERIMENTAL INVESTIGATIONS

The testing arrangement, member details and cross-sectional dimensions are shown in Fig. 1 (Takanashi *et al*, 1993). One half of two storey single-bay frames was tested horizontally, with symmetry accounted for in the boundary conditions. The frames were fixed at the base, whilst hydraulic actuators were used to apply displacements and/or loads at the two floor levels. The frames are referred to as either rigid (RGD) or semi-rigid (SRB). Rigid connections were fully welded, whilst the semi-rigid connections consisted of top and seat angles, and two side angles. In all frames, rolled steel profiles were used for the columns whereas the beams were built-up from welded steel plates; stiffener plates of 9 mm thickness were welded onto the shear panel. Average material properties obtained from test samples are given in Table 1, where σ_y, σ_u and ε_{ult} are the yield stress, ultimate stress and ultimate strain, respectively.

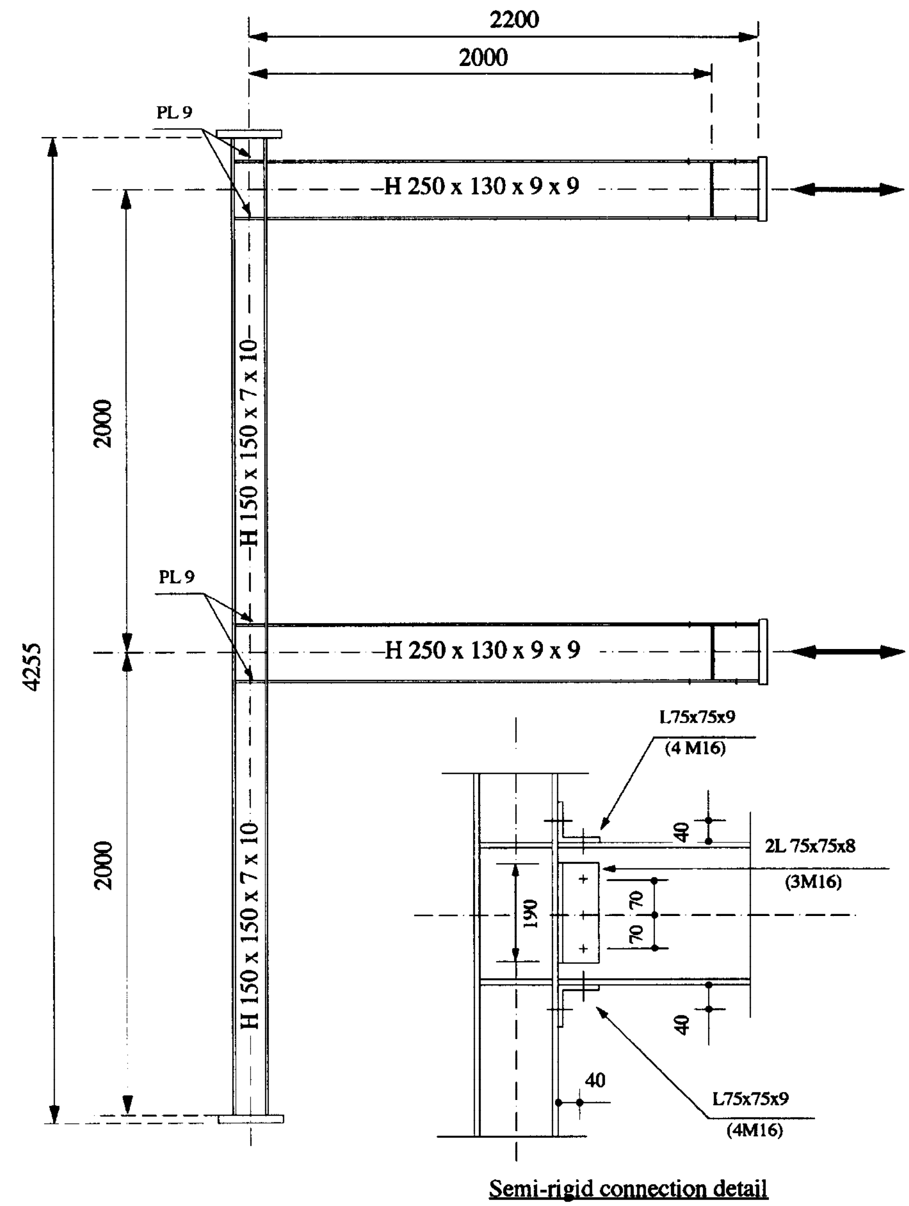


Fig. 1. Testing arrangement and frame details

For the monotonic and cyclic tests, the second floor actuator was used in a displacement-control mode, where the displacement was incremented in one direction. The second floor actuator force was then measured and used to drive the first floor actuator, which was operating on a load-control basis. The ratio between the second and first floor loads was maintained at a 2:1 ratio. Table 2 gives a summary of the testing procedures for the frame specimens, referred to as either semi-rigid (SRB) or rigid (RGD); m_1 and m_2 are the masses assumed at first and second floor levels, respectively; T_1 and T_2 are the first and second natural periods; A_{max} is the maximum acceleration amplitude used.

Table 1. Material Properties

	σ_y (N/mm^2)	σ_u (N/mm^2)	ε_{ult} (%)
Column flange	288	436	28
Column web	332	455	24
Beam plates	276	424	30
Angles 75x75x9	301	450	29

Table 2. Testing Sequence and Procedure

Model Reference	Testing Method	m_1 (Kg)	m_2 (Kg)	T_1 (sec)	T_2 (sec)	A_{max} (g)
SRB01	Monotonic	-	-	-	-	-
RGD02	Cyclic	–	–	–	–	-
SRB02	Cyclic	-	-	-	-	-
RGD03	Pseudo-dynamic	8,400	8,400	0.62	0.19	0.54
SRB03	Pseudo-dynamic	8,000	8,000	0.60	0.20	0.45

Based on predictive analysis, part of the 1940 Imperial Valley, El Centro, NS component acceleration record, of 15 second duration, was chosen to conduct the pseudo-dynamic tests. The masses and peak ground acceleration were appropriately selected in order to satisfy a fixed capacity-to-testing ratio in both frames, i.e. the same scaling factor to the frame capacity, while maintaining a similar fundamental period as shown in Table 2. Equal concentrated masses at both storeys were assumed in the pseudo-dynamic algorithm; 8400 Kg for RGD03 and 8000 Kg for SRB03. Peak ground acceleration of 0.54g was chosen for RGD03, whereas the corresponding value for SRB03 was 0.45g. More detailed description of the testing programme is given elsewhere (Takanashi *et al*, 1993).

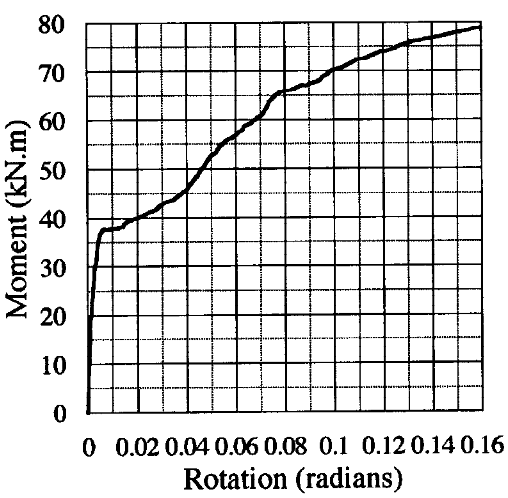


Fig. 2. Monotonic moment-rotation relationship

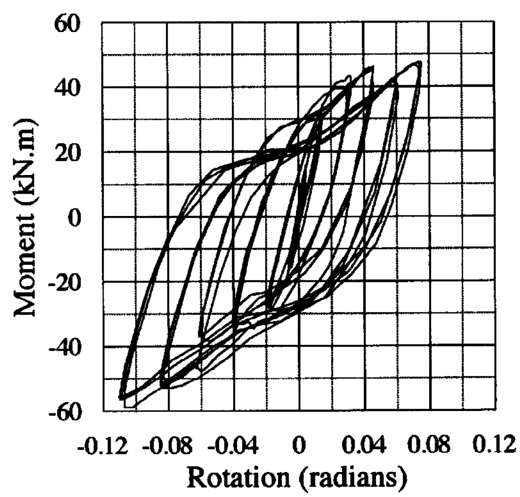
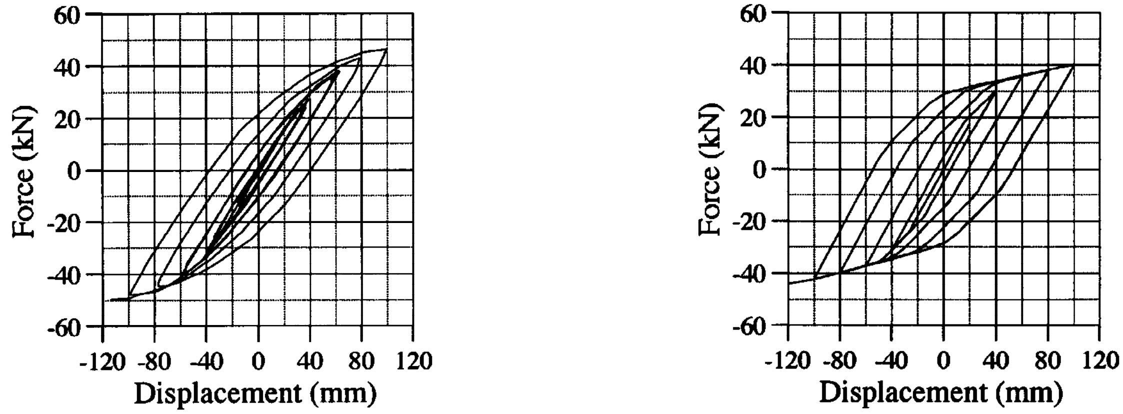


Fig. 3. Cyclic moment-rotation relationship

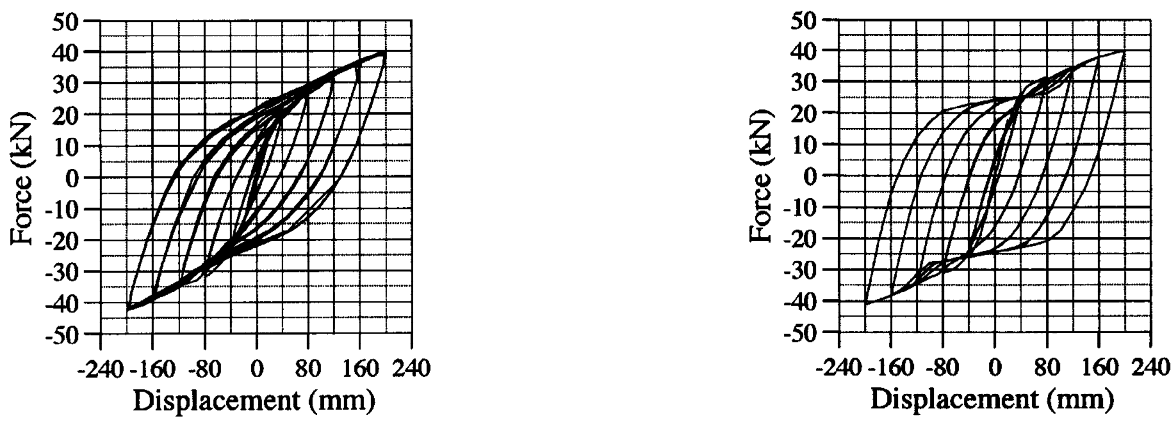
Figures 2 and 3 show the experimental moment-rotation curves for the semi-rigid frame extracted from the monotonic and cyclic tests, respectively. Figures 4 and 5 depict the load-displacement relationships at the top of the frame for models RGD02 and SRB02, respectively. The results clearly indicate the ductile and stable behaviour of the connection. The displacement response histories at both floors from the two pseudo-dynamic tests on models RGD03 and SRB03 are given in Fig. 6. Both frames exhibited stable hysteretic behaviour. Model RGD02 gave maximum displacements of 70 mm and 120 mm at the first and second storey levels, respectively. The corresponding values for the semi-rigid model were 40 mm and 85 mm, respectively, which are considerably lower than the rigid frame. In other words, tested under a consistent capacity-to-testing peak ground acceleration ratio, the semi-rigid frame response displacements may be lower than the fully-welded case as demonstrated in the above-described tests.



Experiment

Analysis

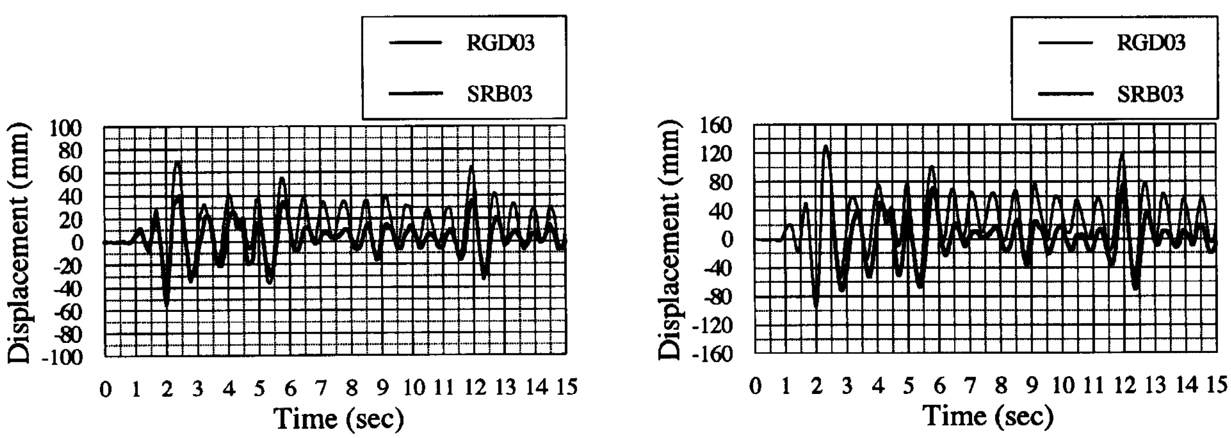
Fig. 4. Experimental and analytical load-displacement relationships for RGD02 at the top of the frame



Experiment

Analysis

Fig. 5. Experimental and analytical load-displacement relationships for SRB02 at the top of the frame



First Floor

Second Floor

Fig. 6. Experimental displacement response history for RGD03 and SRB03 at both floor levels

ANALYTICAL STUDIES

The advanced nonlinear dynamic analysis computer code ADAPTIC (Izzuddin and Elnashai, 1989) was used for all the analytical studies. A new component-based cyclic model has also been incorporated in the program (Madas and Elnashai, 1992) to account for the detailed behaviour of different types of welded and bolted beam-to-column connections. Within the model, the connection is discretised into a number of deformable springs each represented by a cyclic trilinear relationship. The model also accounts for the effect of bolt slippage in the connection. The nonlinear behaviour of the shear panel is represented by a trilinear cyclic moment-rotation relationship. Coupling between the components relationships are determined according to equilibrium and compatibility requirements. Detailed description of the model is given elsewhere (Madas and Elnashai, 1992).

Analytical simulations using the above-described models were undertaken and compared with the experimental results. The cyclic bilinear kinematic model was used to represent the behaviour of mild steel, for which the value of the strain hardening parameter was assumed as 1.0 %. For the semi-rigid connections, the radial bolt clearance used was 1.0 mm, and the prestressing torque was 200 kN.m. In order to demonstrate the accuracy of the analytical models, few comparisons are presented herein. Comparison for the cyclic tests on the rigid and semi-rigid frames RGD02 and SRB02, are shown in Fig.s 4 and 5, respectively. Figure 7 shows the comparison for the pseudo-dynamic tests on models RGD03 and SRB03. As observed in the figures, very good correlation is obtained between the analytical and experimental results.

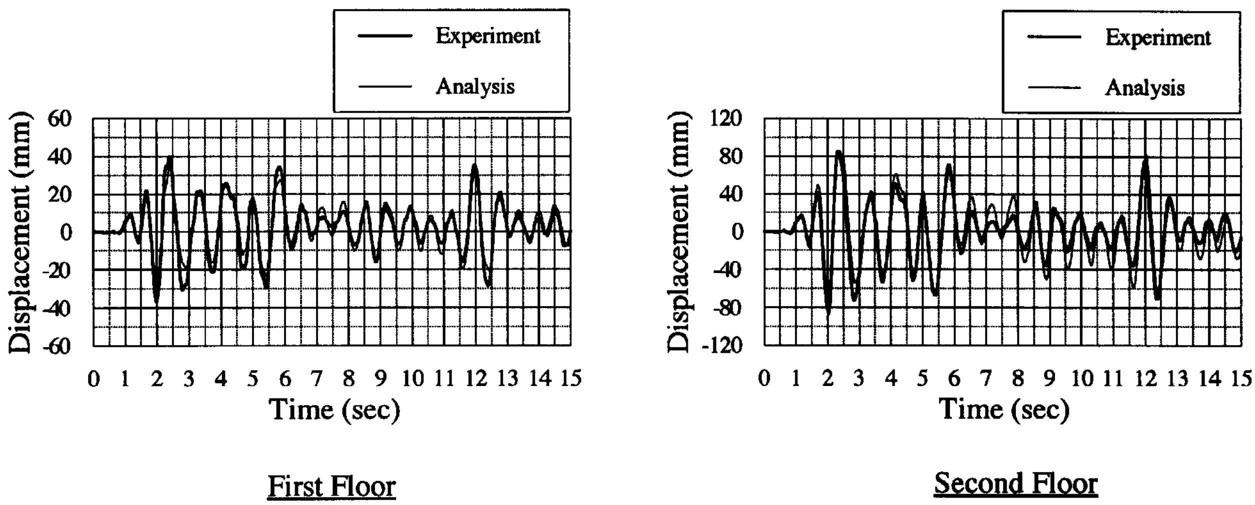


Fig. 7. Experimental and analytical displacement response histories for SRB03 at both floor levels

In order to highlight the effect of different connection types on the overall response of the frame, a simple analytical study was undertaken. Four types of connections covering a wide range of stiffness from fully rigid to flexible, were considered:

Type A: fully-rigid connection, fully welded ignoring shear panel deformation
Type B: as Type A, but accounting for shear panel deformation (as in RGD models)
Type C: semi-rigid connection, top, seat and two web angles (as in SRB models)
Type D: flexible connection, two web angles only

The same geometry and material properties of the test frames, shown in Fig. 1, were used for this study (Elnashai and Elghazouli, 1993). The moment-rotation relationships for the connections are depicted in Fig. 8. The normalised moment-rotation relationships for the connections according to the classifications given in Eurocode 3 (1990) and that suggested by Bjorhovde *et al* (1990) are depicted in Fig.s 9 and 10, respectively. It should be noted that, due to the different reference beam length adopted, the two classifications result in different normalised stiffnesses.

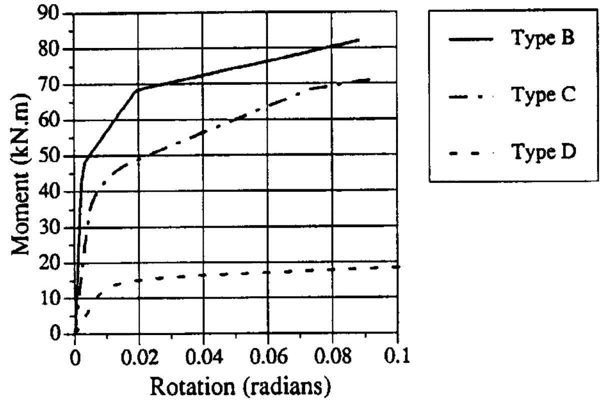


Fig. 8. Moment-rotation relationships of the connections

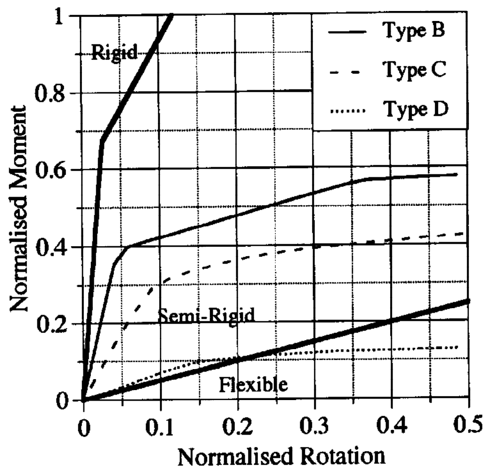


Fig. 9. Classification of connections according to EC3

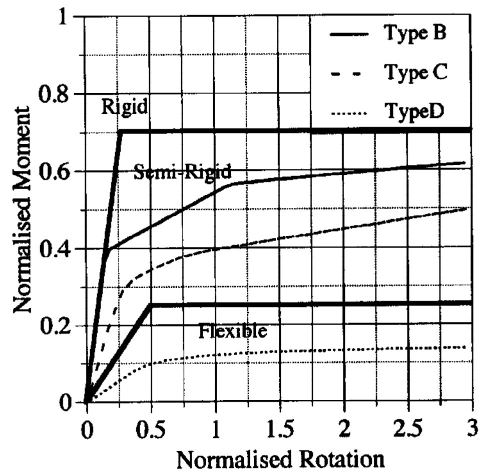


Fig. 10. Classification of connections according to Bjorhovde *et al*

The four frames were first analysed under monotonic loading. The same hybrid displacement-load control used in the tests was adopted. Load-displacement relationships for the four frames at the second floor level are shown in Fig. 11. The stiffness of frames type B, C, and D are approximately 80%, 60% and 30% of the stiffness of the fully rigid frame. Both the yield and ultimate capacities of frames type B, C and D are shown to be approximately 80%, 70% and 40% of the respective values for frame type A; this is accompanied, however, by an increase in the deformation at yield.

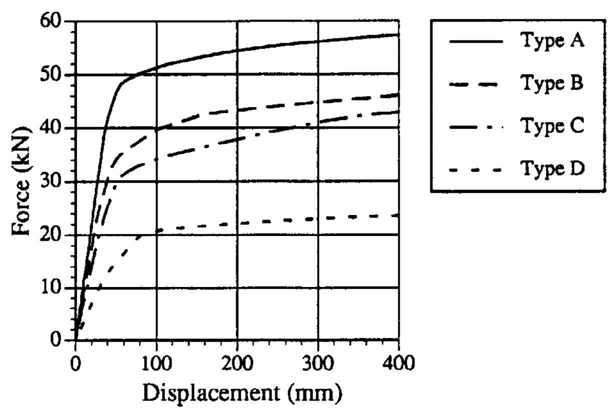


Fig. 11. Load versus displacement at the top of the frames under monotonic loading

Figure 12 shows the migration of the point of contraflexure within the first storey column of each of the four frames. In the figure, the distance from the base of the frame to the point of contraflexure in the column is normalised by the height of the first storey column, and presented against the control displacement at the second floor level. As expected, the height of the point of contraflexure increases with increased flexibility of the connection; in the inelastic range, the normalised position is approximately 0.5, 0.6, 0.65 and 1.15 for types A, B, C and D, respectively. In this example, for the flexible frame, type D, the point of contraflexure lies outside the column.

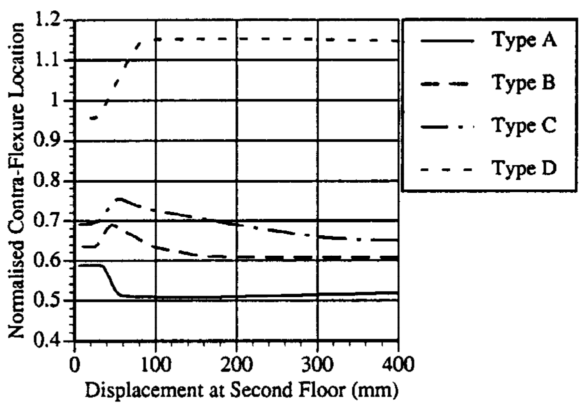


Fig. 12. Normalised position of the contra-flexure point versus the control displacement

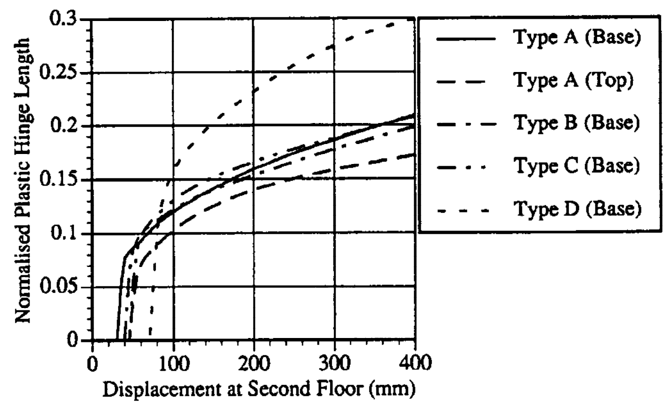


Fig. 13. Normalised plastic hinge length versus the control displacement

The plastic hinges occurring within the first floor column were examined for the four frames. Figure 13 depicts the plastic hinge length normalised by the column height for the different frame types. The results are plotted against the control displacement at the top of the frame. Plastic hinges occur only at the base of the column for all types except the fully rigid type where plastic hinges are formed at both ends. The base plastic hinge is formed firstly in Type A, at a displacement less than 30 mm; the appearance of the hinge is delayed gradually with increased flexibility up to about 70 mm for Type D. With increasing displacements, Type D shows rapid increase in the length of the plastic hinge length. This, however, has to be considered in accordance with the yield deformation if ductility was to be accurately assessed. The continuous increase in plastic hinge length observed in the analysis is due to the assumed constant increase in strain hardening. It should be noted as well that at such large strain levels, local failure criteria at the connection and section levels have to be considered and may govern the response.

Table 3. Dynamic characteristics of the analysed frames

Frame Type	ω_1 (rad/sec)	ω_2 (rad/sec)	T_1 (sec)	T_2 (sec)
A	10.55	32.54	0.596	0.193
B	9.46	31.32	0.664	0.201
C	9.03	30.80	0.696	0.204
D	8.54	30.21	0.736	0.208

The four frames were then subjected to a 100% El Centro N-S component acceleration time history of 15 seconds duration. Masses were assumed at the two floor levels to be 10,000 Kgms each. Eigen value analyses were carried out for the four frames to determine the natural periods of vibration. The results are given in Table 3.

Table 4. Peak displacements for different frames

Frame Type	Peak displacement (mm) First floor	Peak displacement (mm) Second floor
A	48	68
B	37	71
C	39	77
D	36	90

Due to the varying dynamic characteristics of the frames, the relative response of the four frames showed significant differences in terms of both the response frequency and the displacement amplitudes. For brevity, only the peak displacements from the dynamic analyses are given in Table 4. At the first floor level, the largest displacement amplitude of approximately 49.0 mm is observed in the response of the fully rigid frame, Type A, whereas the peak displacement does not exceed 40 mm in any of the other three frames. This shows that even when all types of frame are subjected to the same ground motion, semi-rigidly connected frames may still demonstrate favourable response.

CONCLUSIONS

Several observations and conclusions were made above. The main conclusion of the experimental and analytical studies presented in this paper is that semi-rigidly connected frames demonstrate adequate and, in some cases, favourable earthquake-resistant qualities. It was shown that semi-rigid frames do exhibit a ductile and stable hysteretic behaviour. Although the stiffness and capacity of semi-rigid frames are lower than similar rigid frames under monotonic and cyclic loading, the response under earthquake loading largely depends on the dynamic characteristics of the both the frames and the input ground motion. In many cases, the response of semi-rigidly connected frames may be superior to rigid frames, provided that stable hysteretic behaviour is ensured.

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