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Seismic Resistance of Semi-rigid Steel Frames

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Summary

The paper deals with aspects of seismic resistance of steel frames with bolted connections, as an alternative to fully welded and hybrid configurations which have indicated inferior behaviour during recent earthquakes. Experimental and analytical studies undertaken to verify the performance of bolted steel frames in high seismicity areas are briefly outlined. The investigations indicate that, compared with fully welded counterparts, bolted frames may demonstrate favourable seismic behaviour and may be used as a more reliable and cost-effective form for earthquake resistance.

1. Introduction

Modern seismic design relies on two fundamental aspects of structural engineering. The first is the realisation of a pre-defined, favourable, failure mode, whilst the second is the provision of deformation capacity sufficient to absorb the earthquake input energy. The former is referred to as 'capacity design' whilst the latter, which is much less developed, is referred to as 'displacement based design'. Modern code provisions, such as Eurocode 8, provide a complete framework for capacity design. However, this is still force-based, with a check on displacement. Recent publications (e.g. Kowalski and Priestley, 1995; Calvi and Kingsley, 1995) give proposals for seismic design based on displacement, with a check on force, for buildings and bridges, respectively. The application of both concepts hinges on a precise knowledge, a priori, of the strength and deformation characteristics of the structure and its constituent components. An alternative to this precision is the provision of a fuse; a structural component which can be rigorously designed to yield prior to the over-stressing of other components. Both approaches; precision in force and deformation capacity estimates and provision of fuses, lend weight to the use of steel structures in seismic design. This is somewhat balanced by the cost and accessible technology advantage afforded by concrete structures.

Steel moment frames subjected to earthquake loading are traditionally designed with fully welded connections. This is justified by noting that the static stiffness of economical bolted connections is significantly lower than their welded counterparts, hence they are supposed to violate code drift limitations. By imposing drift limits on the inherently flexible bolted connections frame, the cost advantage of eliminating welding is partially or completely lost.

Experimental and analytical work undertaken in recent years (Nander and Astaneh, 1989; Elnashai and Elghazouli, 1994) has highlighted the fallacy of this treatment, which is based on static response. Due to the period elongation and energy dissipation in the connection, bolted frames may indeed displace less than welded structures, since they attract less load and posses higher damping. These studies, amongst others, opened the door to further work on the development of a complete design procedure for such frames. This effort was given an added impetus by the reported failures of welded connections during the Northridge (USA) earthquake of 17 January 1994, followed by further evidence from the Hyogo-ken Nanbu (Japan) earthquake of 17 January 1995. The development of seismic design regulations for bolted connections became not only an issue of economy but also one of safety.

In this paper, a brief history of the effects of earthquakes on steel structures is given alongside comments on the advantages of bolted versus welded connections. This is followed by an expose of experimental investigations undertaken at Imperial College and the University of Tokyo, and supporting analyses. The paper gives a clear indication that many of the drawbacks of steel structures in seismic design are alleviated by bolted partial strength connections.

2. Response of Steel Frames in Previous Earthquakes

For many years, steel structures enjoyed the reputation of being the most suitable form of construction for earthquake resistance. Publications sponsored by the steel industry in the United States (AISI, 1991) found a dearth of cases of steel failure to report, hence concentrated on reinforced concrete damage instead. A review of eleven earthquakes world-wide from 1964 (Alaska) to 1990 (Philippines) indicated that only minor damage is sustained by steel structures, in contrast with the extensive damage and collapse suffered by reinforced concrete structures. In particular the Mexican earthquake of 1985 produced striking statistics (Table 1; Yanev et al, 1991). It is, however, important to note that RC structures are in the overwhelming majority, hence the exposed sample is much larger than for steel. Moreover, land-mark projects are more likely to be in steel, which is better suited to high rise structures. Such projects typically are subject to stringent quality control procedures, in contrast with residential owner-builder structures.

Type of Structure	Extent of Damage	Number
RC Frame	severe	45
	collapse	82
Steel Frame	severe	2
	collapse	10

Table 1. Damage Comparisons from the Mexico Earthquake, 1985

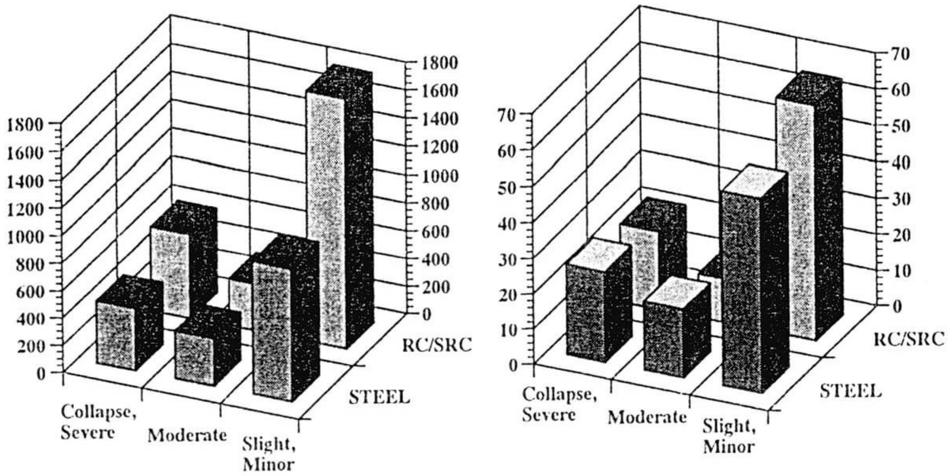
Taking into account the distinct characteristics of the exposed building stock, the conclusion drawn in the AISI publication (Yanev et al, 1991) that steel exhibits all the favourable seismic resistance characteristics is not substantiated.

Over the years, isolated cases of steel damage were reported, such as a heavily loaded x-braced warehouse (Miyagi-ken Oki, Japan, 1978) which suffered bracing distress, an eleven storey moment frame with first floor connection failure and the spectacular failure of two of the three structures in PiËo Suarez complex (Mexico City, 1985). Damage was reported to a four storey braced steel frame in the area affected by the Whittier Narrows earthquake of 1987, with a few buckled bracing members. The damage was blamed though (Yanev et al, 1991) on an attached RC structure which is reported to have caused high torsional forces on the steel frame. With almost complete devastation of the area hit by the Spitak (Armenia) earthquake of 1988, all types of structure, with the exception of steel frames, suffered extensive damage and collapse. In the case of the latter, only a few cases of weld failure were reported. Further confirmation of the high seismic resistance of steel structures was furnished by the Loma Prieta damage assessment (EERI, 1995 amongst others). Only consequential damage to a very small number of steel frame buildings was reported, alongside buckling of a bracing member used to retrofit a reinforced concrete structure in San Francisco (Elnashai et al, 1989).

A few days after the Northridge earthquake of 17 January 1994, reports emerged that several cases of steel beam-column connections have failed in a brittle manner. The extent of this damage unfolded gradually and has now emerged as a most serious concern about the safety of steel frames in seismic areas. Out of a sample of 89 buildings selected as representative of 300 to 400 steel buildings in the area affected by the earthquake (NYA, 1995), 26% of all connections were damaged. If the number of connections inspected (2342) is taken into account, this percentage gives 615 damaged connections in the sample chosen. This further implies that there could be a few thousand failed connections in the area affected by the Northridge earthquake.

Repair of these connections often necessitates evacuation of the building and loss of use, in addition to the expense of repair and recladding. The effect of this earthquake on the reputation of steel structures as the ideal earthquake resistant construction material has been devastating. This prompted the industry, in liaison with Government agencies, professional institutions and academic establishment to create a joint venture (SAC; Structural Engineering Association of California, Applied Technology Council and California Universities Research into Earthquake Engineering) to respond to the questions raised by the extensive damage observed. It is beyond the scope of this paper to discuss possible causes of the observed damage, but it is sufficient to state that laboratory tests have confirmed that connections designed and manufactured strictly to code requirements and best shop practice failed to provide the necessary levels of ductility.

The standing of steel in seismic design was dealt another blow in the Hyogo-ken Nanbu earthquake of 17 January 1995 (EERI, 1995, Elnashai et al, 1995 amongst many others). Apart from the devastation inflicted on old open lattice column structures, which was neither surveyed nor reported, a very large number of steel frames suffered distress, especially in the beam-column connection. It is noteworthy though that connection configuration in California and in Japan are distinct. Whereas connections in Japan are usually fully welded, hybrid connections are in wide use in California. These comprise welded beam flanges to column flange (intended for moment transfer) and plates welded to the column flange and bolted to the beam web (intended for shear transfer; termed shear tabs). The causes of damage are therefore not strictly related.



Overall Damage Statistics by Number for All Districts (20 February 1995)

Overall Damage Statistics by Percentage for All Districts (20 February 1995)

Fig.1. Damage Statistics from the Hyogo-ken Nanbu Earthquake (Elnashai et al, 1995)

In contrast with observations from previous earthquakes (with the exception of Northridge) steel structures suffered as much as reinforced concrete structures, especially when considering the right hand side of Figure 1 above. Moreover, if composite steel/concrete structures (referred to in Japan as SRC) are added to steel and not to concrete, the picture becomes even more bleak.

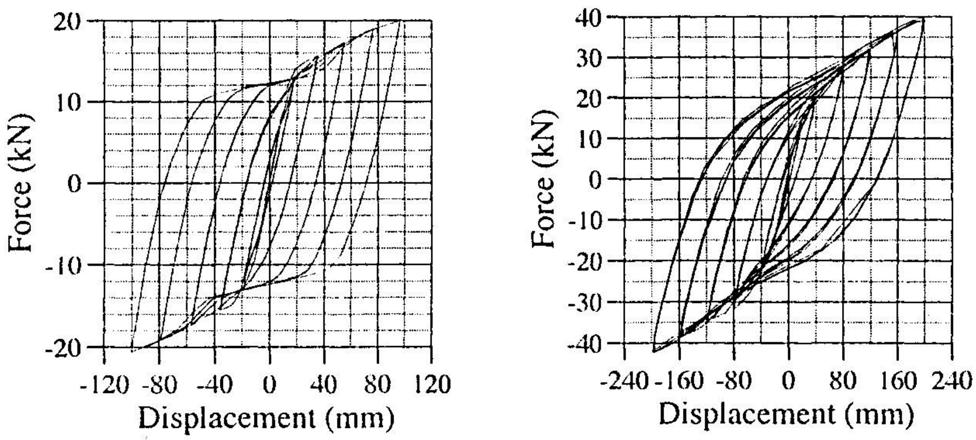
In the light of the above, it is the writers' opinion that steel structures remain a most suitable solution to earthquake design problems. However, there is no room for complacency in their design, since it has been shown that they are no less vulnerable to earthquake damage than reinforced concrete structures. Moreover, the repeated observation of failure of welded connections lends further weight to the effort dedicated to the development of seismic design rules for bolted connections, as discussed further in this paper.

As a consequence of the damage inflicted on welded and hybrid connections, re-assessment of steel seismic design procedures is underway. The reliability of welded connections, which has been subject to scrutiny in Japan for several years, is being examined fundamentally and new connection configurations are under development. The half way mark has been reached via the publication of guidance notes on the seismic design of rigid bolted connections (Astaneh, 1996). The next step forward is the use of semi-rigid bolted connections, with full or partial strength, which is the subject of the sections below.

In addition to the concerted effort dedicated to improving seismic design regulations for new construction (e.g. SAC Advisory Notes and the Interim Guide), several proposals have been forwarded for the upgrading of existing connections. This may take one of two forms, (I) strengthening of the connection by cover plates or other means or (ii) weakening of the beam by trimming or perforating the flanges.

4. Experimental Investigations

An experimental programme carried out as a joint activity between the Institute of Industrial Science, Tokyo, and Imperial College, London, investigated the feasibility of semi-rigid frames in comparison with rigid alternatives. Fully-welded connections were used for the rigid frames, whereas the semi-rigid frames comprised bolted connections with top and seat angles, and two web cleats. Full details of the models and loading regimes are given elsewhere (Takanashi et al, 1993; Elnashai and Elghazouli, 1994).



First Floor

Second Floor

Fig. 2. Hysteretic load-displacement relationships from a cyclic test on a semi-rigid frame

Monotonic, cyclic and earthquake tests were performed on two-storey rigid and semi-rigid steel frames. The monotonic and cyclic tests were carried out under a hybrid displacement/load control procedure whereas the earthquake tests were performed using the pseudo-dynamic technique. In the monotonic and cyclic tests, the semi-rigid connections sustained the expected moment capacity with high rotational ductility and largely stable hysteretic behaviour, as shown in Figure 2.

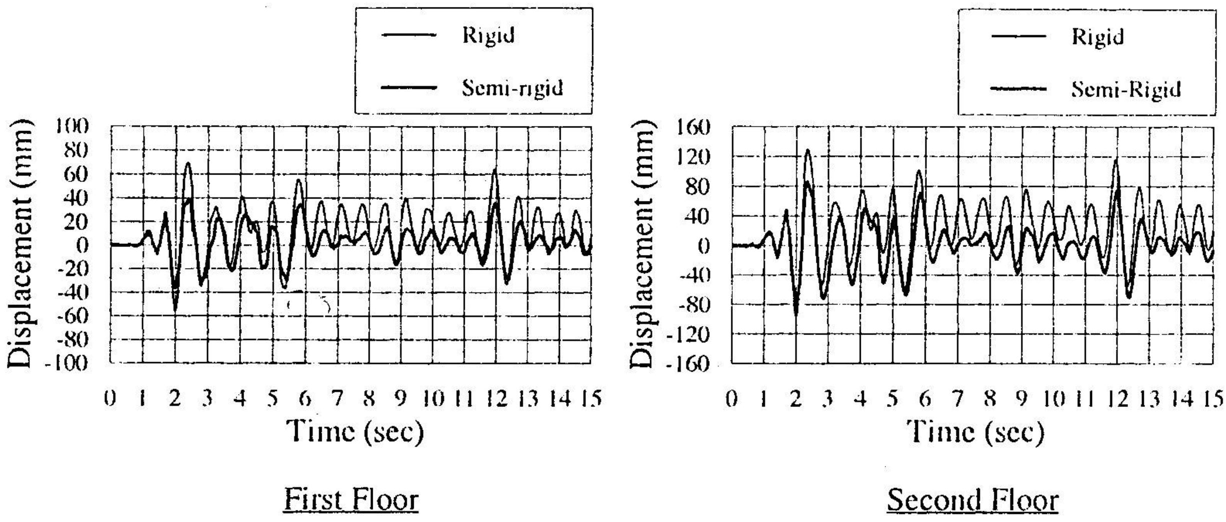


Fig. 3. Displacement response rigid semi-rigid frames under fixed capacity/load ratio

In the pseudo-dynamic tests, the same earthquake records were applied on similar rigid and semi-rigid frames. However, the masses and peak accelerations were appropriately scaled to satisfy a fixed strength-to-load ratio in both frames whilst maintaining a similar fundamental period. As shown in Figure 3, the peak displacement response was considerably lower in the semi-rigid case. This indicates that, despite their relative flexibility semi-rigid frames may, depending on the characteristics of both the structure and the applied load, cause smaller storey deformations as compared to rigid frames.

5. Analytical Studies

Analytical investigations were undertaken to study the effect of different connection types on the frame response. Full results from this investigation together with a detailed description of the analytical models used are given elsewhere (Elnashai and Elghazouli, 1994). Two storey frames similar to those used in the experiments, with beam-to-column connection properties ranging from fully rigid to flexible, were analysed under monotonic and earthquake loads.

The four frames were first analysed under monotonic loading. The monotonic moment-rotation relationships for the connections are shown in Figure 4. The same hybrid displacement-load control used in the experiments was adopted. Displacements were applied at the top floor, whereas the load at the first floor level was kept at half the value of the top-floor restoring force.

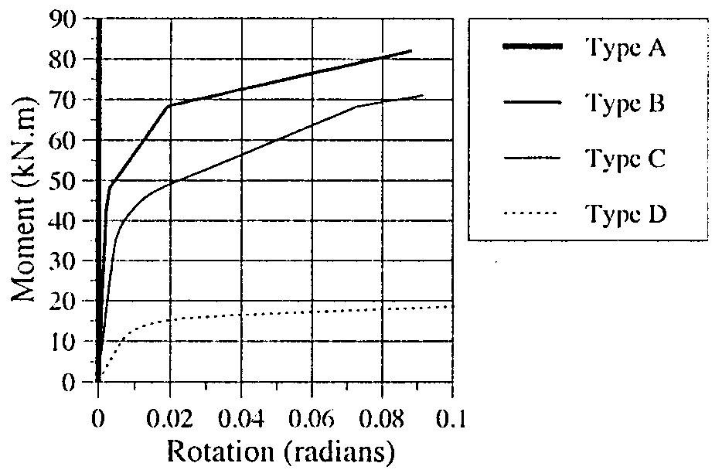


Figure 4. Moment-rotation relationships for different types of connections

Load-displacement relationships for the four frames are shown in Figure 5 at the first and top floor levels, which shows the effect of the connection properties on the frame stiffness and capacity. The reduction in yield and ultimate capacities for semi-rigid and flexible frames is accompanied however by an increase in the deformation at yield.

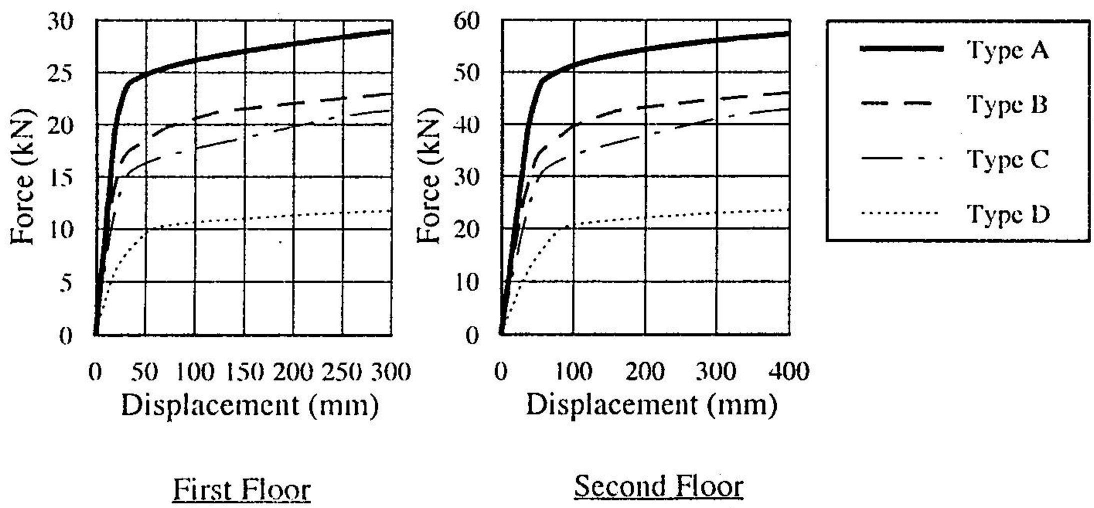


Figure 5. Load displacement relationships for the different frames

The connection properties were also shown to have a significant influence on several important response parameters such as the number, length and location of plastic hinges, and in turn on the overall ductility of the frame. In addition, the influence of the connection stiffness on the dynamic characteristics of the frame was also studied. The four frames were subjected in turn to the full scale El Centro N-S component acceleration time history. Table 2 gives the natural periods of vibration of the frames as well as the peak displacement response at both floor level in each frame. Due to the 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 example, as shown in Table 2, 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.

Frame Type	T1 (sec)	T2 (sec)	Peak Displ (mm) *First Floor*	Peak Displ (mm) *Second Floor*
A	0.596	0.193	48	68
B	0.664	0.201	37	71
C	0.696	0.204	39	77
D	0.736	0.208	36	90

Table 2. Natural periods and peak displacements for the analysed frames

The results of the dynamic analyses confirmed the conclusions of the experimental work that, even when all types of frame are subjected to the same ground motion, frames classified as semi-rigid may still exhibit favourable response compared to similar rigid types.

Further analyses were undertaken (Danesh, 1996), with funding from the UK Engineering and Physical Sciences Research Council (EPSRC) to quantify the limits of applicability of semi-rigidly connected frames in areas of medium to high seismicity. Several structural configurations were studies, with different numbers of bays and stories. The main parameters studies where:

- Connection capacity as a percentage of beam capacity
- Column design actions

The objective of studying the former parameter is to give an indication of the minimum connection capacity consistent with seismic integrity, with the consequence of saving in materials and construction costs. The aim from studying the latter parameter is to investigate the possibility of releasing the requirement of column over-strength for capacity design purposes. This would lead to further savings in column sections. A sample of results for two storey two bay and four storey two bay frames is given in Tables 3 and 4.

Frame	Drift %	Beam Rotation	Connection Rotation
Rigid	1.5	0.019	-
22C3R	2.4	0.030	0.021
22C5R	2.8	0.028	0.020
22C7R	2.4	0.026	0.017
22C3C	2.8	0.022	0.029
22C5C	2.3	0.022	0.021
22C7C	2.2	0.024	0.019

Table 3. Response Parameters for 2 Storey 2 Bay Frames

In the Tables 3 and 4, C3, C5 and C7 stands for connection strength 30%, 50% and 70% of the beam strength, respectively. Suffix R or C indicate whether the column design actions are based on magnified beam actions (R) or actions from analysis of the frames under earthquake and static loads (C). Maximum drift, beam and connection rotations are obtained by dynamic analysis of the frames using an artificial accelerogram compatible with the EC8 elastic spectrum for soil class B, scaled to a ground acceleration of 0.3g.

Frame	Drift %	Beam Rotation	Connection Rotation
Rigid	1.8	0.020	-
42C3R	2.5	0.017	0.024
42C5R	2.0	0.019	0.017
42C7R	2.0	0.020	0.018
42C3C	2.6	0.016	0.023
42C5C	2.0	0.018	0.017
42C7C	1.9	0.019	0.017

Table 4. Response Parameters for 4 Storey 2 Bay Frames

Collectively, the results indicate that semi-rigid frames are indeed a feasible solution even with design accelerations of 0.3g. It is however noted that a connection with 30% of the beam capacity leads to drift limits at odds with code recommendations. It is also indicated that imposing over-strength requirements on columns in the presence of partial strength connections is pointless. Further analysis is underway at Imperial College, but the sample results presented above are quite conclusive in confirming the viability of use of partial strength bolted connections in seismic design.

6. Concluding Remarks

The paper discussed the feasibility of frames with bolted connections for seismic resistance as an alternative to welded frames. The alarmingly inferior behaviour of welded connections in recent earthquakes was described. In addition, the potential advantages, in terms of reliability and economy, of using bolted as opposed to welded connections were highlighted. The experimental and analytical studies presented show that semi-rigidly connected frames provide adequate and, in some cases, favourable earthquake-resistant qualities. It was shown that semi-rigid frames do exhibit 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 motion. The results demonstrated that bolted frames may displace less than their welded counterparts, contrary to common belief. In general, it was shown that the response of semi-rigidly connected frames may be superior to rigid frames, provided that stable hysteretic behaviour is ensured. Further examination of the effect of connection strength and column design force was carried out. It was shown that provided the connection strength is higher than about 50% of the beam strength, bolted frames satisfy existing seismic code requirements even for high design ground accelerations. Moreover, it is confirmed that capacity design regulations expressed as column over-design factors need not apply. This leads to on the whole further economy in materials.

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