Seismic behaviour of welded beam-to-column connections in steel frames

Comportamento sismico di connessioni trave-colonna saldate in telai di acciaio

Within the "Steelquake" research program, the behaviour of welded beam-to-column connections in steel frames has been investigated under the effect of seismic actions. The interest for the problem originated from the analysis of buildings damaged by the Northridge and Kobe earthquakes, which gave evidence to a series of unexpected brittle fractures in welded joints. A wide number of both experimental and numerical studies has then been performed to the purpose of clarifying the conditions under which the brittle collapse of steel welded connections can occur. On the basis of the many results made available by the research, two specific problems are discussed in the present work, namely the real stress distribution in the welding zone and the plastic response of the panel zone in the column web; the interest for this last point is due to the positive contribution which may come from the panel zone plastic behaviour to frame ductility.

Nell'ambito del progetto di ricerca "Steelquake" è stato studiato il comportamento di connessioni saldate trave-colonna in telai di acciaio soggetti ad azioni di tipo sismico. L'interesse per il problema è stato sollevato dall'analisi dei danni prodotti negli edifici dai terremoti di Northridge e Kobe, che ha posto in evidenza una serie di impreviste rotture fragili in giunti saldati. E' stato quindi svolto un grande numero di studi sperimentali e numerici con lo scopo di chiarire sotto quali condizioni si possa manifestare il collasso fragile di connessioni saldate fra elementi di acciaio. Sulla base dei numerosi risultati prodotti dalle ricerche svolte, l'articolo discute due problemi specifici: la reale distribuzione di sforzi presente nella zona interessata dalla saldatura e la risposta plastica della zona di pannello nell'anima della colonna; l'interesse per quest'ultimo punto è legato al positivo apporto della plasticizzazione del nodo alla duttilità globale del telaio.

Prof. dr. ing. Claudio Chesi, dr.ing. Marco Valente
Dipartimento di Ingegneria Strutturale, Politecnico di Milano

1. INTRODUCTION

In the last few years the seismic performance of welded beam-to-column joints in moment resisting frames has been widely debated by many researchers; the well known cases of brittle failures reported after the 1994 Northridge and 1995 Kobe earthquakes, indeed, have contradicted the assumption of a ductile behaviour, which is commonly adopted for this structural typology in the presence of repeated cycles of inelastic deformations. Field reports from the Northridge earthquake, in particular, documented that failure was generally due to the fracture of butt welds at the bottom flange, with little or no evidence of plastic engagement of the beam; also, it was highlighted that in some connections fractures started with the stress level in the beam clearly below first yield, while in some panel zone yielding had already taken place (NIST, 1997). Although the complete joint collapse was not reached in many cases, these failures pointed out for the need to clarify in which conditions they are likely to be induced by seismic actions. As a consequence, several research programs on this topic have been developed around the world, which are briefly summarized in the following.

Extensive experimental investigation on the subject has been promoted at Sacramento and Berkely Universities under the SAC Program (Mahin et al., 1997; Popov et al., 1998), at S. Louis University (Warmka et al., 2000), at Hiroshima University (Matsuo et al., 2000). Detailed finite element analyses have also been used to examine fracture toughness requirements in welded beam-to-column connections; toughness demands have been evaluated in terms of the elastic stress intensity factor (KI) and inelastic Crack Tip Opening Displacement (CTOD). The analyses quantified the effects of high localized stresses and strains in the proximity of weld root defects in beam flange welds of standard connections (representative of pre-Northridge construction); observations collected from full-scale experimental tests on connections have thus been confirmed (Wei-Ming Chi et al., 2000; Ostertag, 2001).

A wide research program, the "Steelquake" project, has also been promoted by the European Community (Plumier et al., 1998; Bernuzzi et al., 1997; Castellani et. al., 1997), under which both laboratory tests and numerical activities have been performed. Experimental investigations included: a) full-scale tests of 78 beam-to-column joints, representative of typical connections; b) shaking table tests of a reduced scale (1:2) moment resisting frame; c) pseudo-dynamic cyclic testing of a full scale frame.

Different failure modes were observed in the tested connections depending on welding procedure, local details, and cyclic load amplitude; a few modes could be classified of a brittle nature. In particular, evidence was collected of brittle fractures in welded joints in proximity of the weld toe, which are representative of typical cases observed both at Northridge and Kobe. Non-ductile failure modes also occurred in long spanbeams, due to the bottom flange buckling.

Experimental activities on welded joints have shown that brittle fractures are influenced by welding defects, stress concentration, poor local detailing, and low toughness weld metal, but have not provided full comprehension of the phenomenon. A better understanding has come from studies based on finite element models; from these, it appears that the stress propagation through the connection does not follow the classical beam theory, which assumes the bending moment transferred by the beam flanges and the shear force carried by the web. In particular, it has been shown that: 1) approaching the connection, the beam stress resultant tends to flow towards the flanges, where concentration of normal and shear stresses takes place; 2) the stress distribution is not uniform across the beam flange, with the maximum occurring at the center of the beam. As a consequence, the demand on the full penetration groove welds is not adequately predicted. Similar results have been announced by (Warmka, 2000).

The research reported here has been suggested by specific results coming from the pseudo-dynamic cyclic testing of the full scale frame, performed at the ELSA laboratory (Taucer et al., 1999). Attention has been focused on the role of the panel plate, at the beam-column intersection. Both UBC and Eurocode give rules for the seismic design of panel nodes; according to Eurocode, the panel zone must be designed in the elastic range, while UBC allows to take benefit from the contribution of the column flanges in the evaluation of the shear resistance of the panel zone. This last effect has been widely confirmed by finite element analyses. Notwithstanding this contribution, design calculations often result in the requirement of doubler plates; apart from economical considerations, doubler plates may require heavy welding, thus provoking distortion and residual stresses. Large welds also create a heat-affected zone that increases the risk of brittle behaviour in the joint region.

On the basis of numerical analyses, it has been shown that, when plastic engagement of the panel plate is possible, stable hysteretic cycles with a controlled hardening take place; a similar result was reported in the seventies on an experimental basis (Popov and Bertero, 1973). Stiffer panel zones, therefore, reducing the panel zone contribution to plastic dissipation, are of doubtful advantage. On the other hand, numerical analyses have proven that yielding of large panel zones may cause early fracture in connections; balance conditions must then be developed, in order to assure a compromise between desirable and undesirable effects.

In the present paper, a contribution to this problem is offered through two numerical models, which were developed within the Steelquake project with the purpose of interpreting some experimental results obtained at the ELSA laboratory. Reliability of the models has been asseverated by validation tests; it has been shown that, for cyclic loading in the non-linear range, the displacement history is thoroughly reproduced when plastic distortion of the elements only is engaged; satisfactory replica of the experimental tests, however, have been obtained even when buckling of the beam flanges is present.

2. RESULTS FROM PSEUDO-DYNAMIC TESTS

Pseudo-dynamic testing of a real scale steel moment resisting frame prototype was performed at the ELSA Laboratory JRC (Joint Research Centre, Ispra). The experimental program included also cyclic tests up to failure.

The specimen was a two-storey, one bay, moment resisting steel frame, as shown in fig. 1. The beams were connected to the column flanges by means of complete joint penetration groove welds of the beam flanges and by fillet welding of the beam webs.

An access hole, with a 40 mm radius, was provided at the bottom of the web, in order to allow for the weld root passes. The connection continuity through the column was ensured by horizontal plate stiffeners on the planes of the beam flanges, fillet welded to the column web and column flanges.

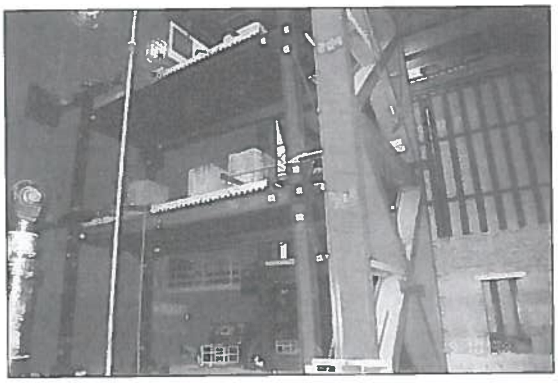

Fig. 1 – General view of the test specimen.

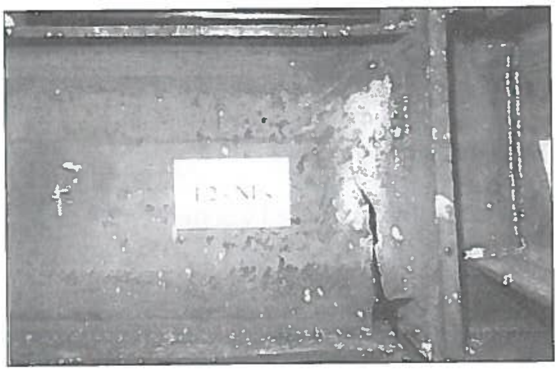

Fig. 2 – Typical failure of a beam-to-column connection.

In addition, the column web panel plate was partially stiffened and strengthened by means of K stiffeners, 12 mm thick, welded to the outer side of the column web panel zone.

The slab did not reach the nodes, but was interrupted at 0.8 m from the outer edges both in the longitudinal and transverse direction, in order to avoid composite action of the slab in the vicinity of the connection.

The cyclic test was performed by imposing a displacement time history at the second storey of the structure; displacements were amplified up to failure of the specimen. The loading function consisted of a set of cycles at the reference ductility level, alternated with cycles of increasing ductility.

Three of the eight beam-to-column connections failed during the cyclic tests; similar failure modes were observed, with fractures in the plane of the beam flange weld; no evidence was found of tearing of the column flange material, cracking or substantial buckling of the beam flange; the fracture propagated through the web of the beam, fig. 2.

The panel plate contributed in some measure to the plastic deformation of the connection, especially at the second storey, where the panel shear forces were higher, fig. 3. The early yielding of the panel plate was considered beneficial, because the plastic rotation demand to the beam was reduced in consequence. This may explain why only one beam out of three failed at the top flange, even though bending moments at the beam ends due to dead load should have induced failure of the top flange welds first: yielding of the panel zone may have absorbed the initial dead load deformations, thus equilibrating the demands between the beam top and bottom flanges.

The panel plate rotations at the first storey were biased towards negative values. This can be explained by the presence of bending moments due to dead load, and by the high restraint provided by the column. The column shear at the first storey is of opposite sign with respect to the panel shear resulting from the beam bending moment, thus explaining the smaller rotations of the panel zone at this storey.

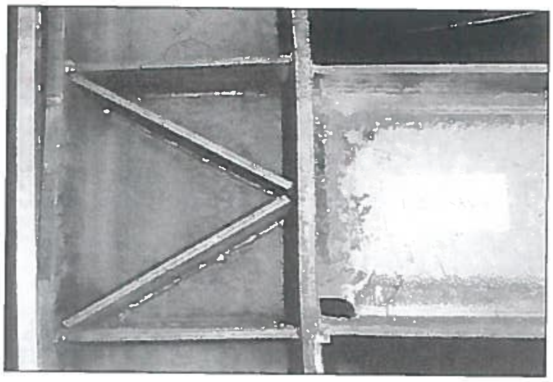

Fig. 3 – Deformation of the panel zone.

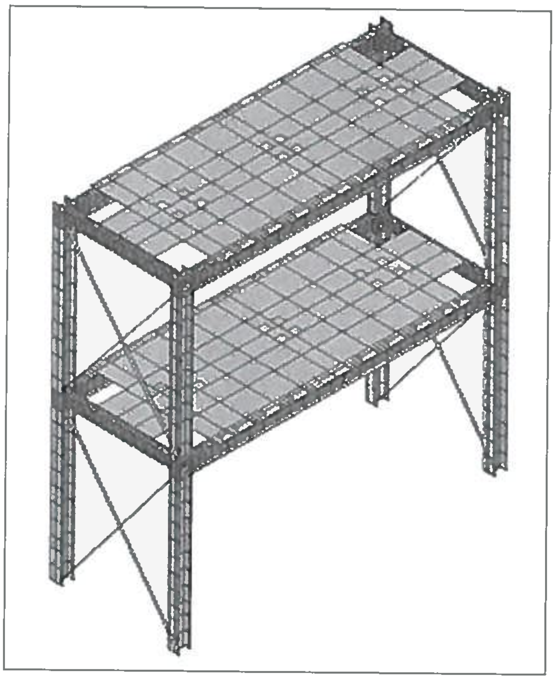

Fig. 4 – Three dimensional finite element model for the moment resisting frame.

3. NUMERICAL MODELS

A three-dimensional model of the complete specimen was developed using the general-purpose finite element analysis program ABAQUS. The three-dimensional finite element model is shown in fig. 4. The finite element analysis results are sensitive to the type of elements and the mesh size and orientation used in the model. Mesh convergence studies were therefore performed. The results indicate that a four-node shell element with reduced integration produces consistent results. The model includes, approximately, 9000 nodes, 7000 shell elements and 40000 degrees of freedom. Slide line elements are used to connect the slab to the steel girders, using a high stiffness to achieve fully composite action.

The analyses account for material nonlinearities through classical metal plasticity theory based on the Von Mises yield criterion; the effects of strain hardening are also considered. Isotropic hardening is assumed for monotonic analyses, whereas kinematic hardening is assumed for cyclic analyses. Geometric nonlinearities are accounted for through a small strain, large displacement formulation.

Weld geometry is neglected in the finite element models and the weld material properties are assumed to be identical to those assumed for the base metal. Although the effects of such an assumption are critical when considering local stress and plastic strain demands, they are insignificant with regard to the system global modelling.

Analyses were performed by imposing a history of sinusoidal displacements at the second storey of the structure. The displacement history is the same as the one applied during experimental testing at JRC; sets of cycles at the reference ductility level (150 mm) were alternated to cycles of increasing ductility, fig. 5. The numerical model was calibrated on the experimental data. A satisfactory agreement between numerical and experimental data was observed for all the analyses performed, fig. 6.

The research was then focused on the behaviour of a single joint, for which a three-dimensional redu-

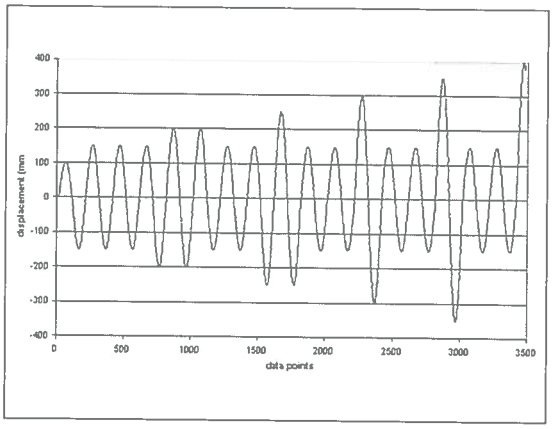

Fig. 5 – Imposed cyclic displacement hystory.

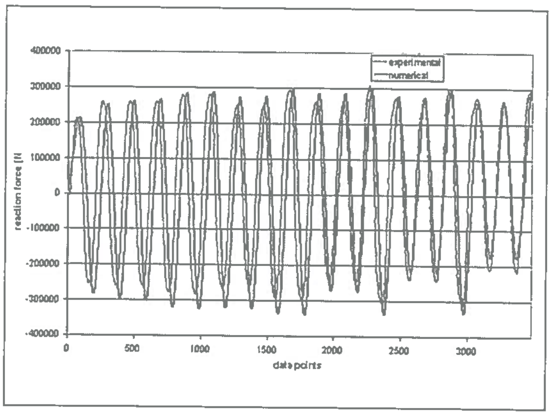

Fig. 6 – Comparison between experimental and numerical results.

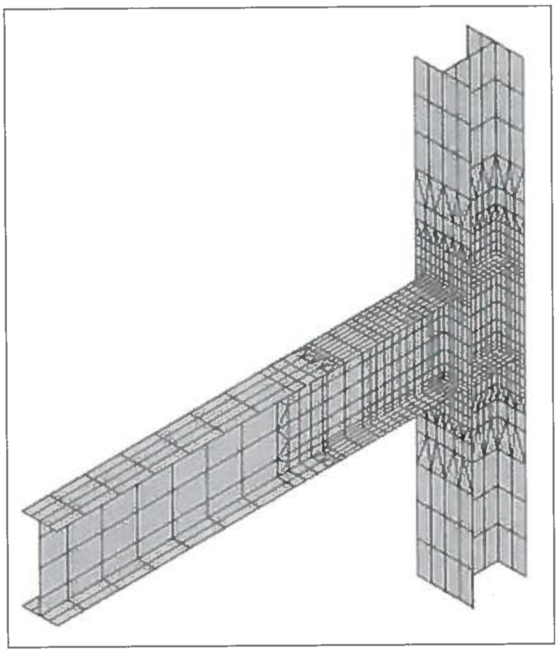

Fig. 7 – Three dimensional finite element model for the joint.

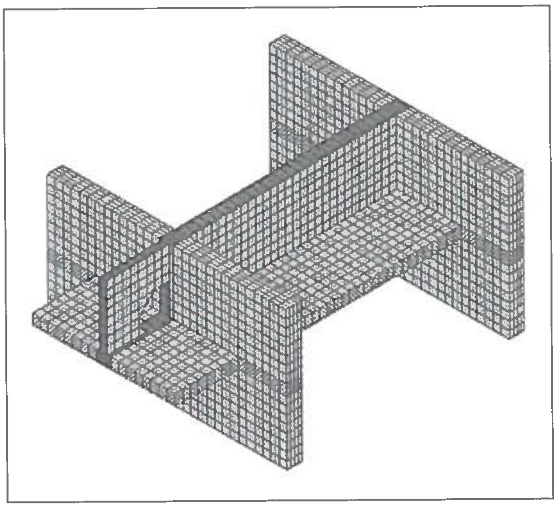

Fig. 8 – Three dimensional finite element sub-model.

ced model was developed. Actions to be applied on this sub-model were taken from the results provided by the frame global analysis.
The reduced model includes 1600 nodes, 1600 shell elements and 9000 degrees of freedom, fig. 7. In the nodal zone, i.e., in the beam and column portions in the vicinity of the joint, the mesh was refined to allow for a more accurate appraisal of the transfer force mechanism. In order to trigger local buckling, an initial out of plane geometrical imperfection was considered of a sinusoidal shape, with an amplitude of 0.02 mm.
A sub-modelling technique was applied to represent the connection between the beam lower flange and the column, including the continuity plates; sub-modelling had the purpose to provide results accurate enough to allow close examination of the ductile fracture potential of the different configurations analyzed. The submodel had 24 elements through the width of the beam flange and 4 through the thickness; four elements were used through the thickness of the beam web. The mesh included approximately 10000 brick elements,13000 nodes and 40000 degrees of freedom, fig. 8. Elastic convergence studies were performed on the sub-structure.
Hydrostatic stress σ_{hyd}, effective plastic strain PEEQ, Von Mises stress σ_{mis}, and Rupture index RI are computed at the critical locations in the region between the column flange and the beam flange:

$$\sigma_{hyd} = -\frac{1}{3}\,\mathrm{trace}\,(\sigma_{ij}) \tag{1}$$

$$\mathrm{PEEQ} = \sqrt{\frac{2}{3}\varepsilon^p_{ij}\varepsilon^p_{ij}} \tag{2}$$

$$\sigma_{mis} = \sqrt{\frac{3}{2}S_{ij}S_{ij}} \tag{3}$$

where $S_{ij}=\sigma_{ij}+\sigma_m\delta_{ij}$ are the deviatoric stress components.
The rupture index was based on the proposal from Hancock and Mackenzie for ductile fracture in steel under a multiaxial state of stresses:

$$RI = \frac{PEEQ}{\exp(-1.5\frac{\sigma_{hyd}}{\sigma_{mis}})} \tag{4}$$

where $\sigma_{hyd}/\sigma_{mis}$ is the stress triaxiality ratio.
The beam yield stress and strain are used for normalization of these quantities. The normalized indices have been computed at different locations and along different lines through the connection in order to allow for comparisons among the different configurations which were investigated, see table 1. Three lines are of particular importance and deserve a detailed description. A first one is at the interface between the beam and column flanges, i.e., the region where many of the observed cracks initiated. A second one is at the edge of the weld zone and is located at mid height of the beam flange; this line is of interest because a number of fractures were observed in the beam HAZ region. Finally, a line is at mid height of the beam flange, and underneath the access hole. This region has been shown to have significant strain concentration, which could lead to low cycle fatigue in the event of ductile behaviour.

4. RESULTS FROM THE FINITE ELEMENT ANALYSES

The response to both monotonic and cyclic imposed displacements has been analyzed. The maximum imposed displacement level corresponds to that observed at failure in the experimental analyses. Computations confirmed the role of the panel zone in reducing the rotation demands on the beams at the second storey; at the same time, smaller rotations of the panel zone and larger demands on the beam rotation were found at the first storey. Moreover, the analysis results show a concentration of tensile stresses at the center of the beam

Hydrostatic stress (MPa)	Von Mises stress (MPa)	PEEQ Index	Stress Triaxiality Ratio	Rupture Index	
FLANGE CENTER - INTERFACE LINE					
237	325	28.5	0.73	0.171	Without doubler plate
223	304	22.3	0.73	0.133	With doubler plate
FLANGE EDGE - INTERFACE LINE					
211	299	40.8	0.70	0.233	Without doubler plate
200	302	42	0.66	0.226	With doubler plate
FLANGE CENTER - BEAM AFFECTED ZONE					
220	317	41	0.69	0.231	Without doubler plate
206	308	39	0.67	0.213	With doubler plate
FLANGE CENTER - ACCESS HOLE CENTER					
223	344	51	0.65	0.269	Without doubler plate
211	332	45	0.63	0.231	With doubler plate

Table 1 – Stress and strain indices at the beam flange.

flange weld, see fig. 9. This is due to the action of the column web, which provides higher stiffness at the center of the weld while, at all other locations, the column flange is less effective. Transverse strains in the beam and column flanges are limited by the surrounding steel; as a result, hydrostatic tensile stresses develop at the center of the flange weld. Due to this state of stress, the center of the flange is a prime location for cracking. The hydrostatic tensile stress state is more severe after panel zone yielding, which indicates that the panel zone yielding increases the potential for cracking.

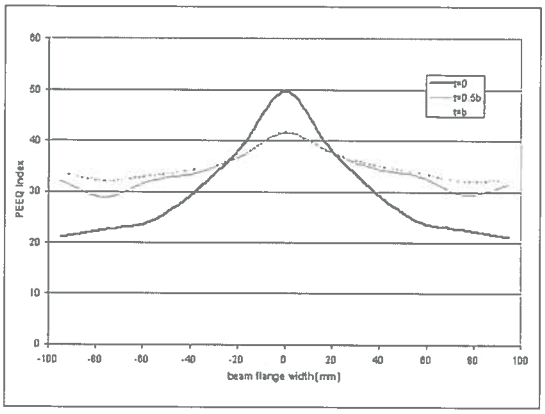

Fig. 9 – PEEQ index distribution across the beam flange at the root of the groove weld.

4.1. Distribution of stresses

The stress state has been studied at a stage close to failure. The distribution of the Von Mises stress in the plane of the beam and column webs has been analyzed; it appears that the stress distribution agrees well with that predicted by the classical beam theory at some distance from the column face, while drastical changes occur in the vicinity of the connection. At the center of the web stresses are extremely low; in the flanges and flange welds, instead, high stress concentration is present. The distribution of stresses is governed not only by the loading function, but by the deformation constraints imposed by the connection details as well. Traditional details of welded moment connections prevent free deformation of the top and bottom fibers of the beam. Such boundary conditions restrain the beam cross section from warping, due to the shear deformation and the Poisson effect. The deformed shape of the column panel zone, which is opposite to the warping shape of the beam web, and the local bending of the column flange increase stresses in the top and bottom regions of the connection.

4.2. Doubler plate thickness

A few different values of the panel thickness have been considered, in order to analyze the effect on the connection response and the potential for fracture. In the case of an unreinforced panel, excessive distortion of the panel zone can occur and conditions are favourable for fracture at the welds connecting the beam and column flanges. Although the plastic rotation demand on the beam is smaller, the stress and strain conditions are more severe; the stress and strain indices, indeed, decrease when the panel zone is strengthened by addition of doubler plates, see table 1. In this situation, the potential for cracking in the flange weld might be reduced. The contribution of the panel zone deformation to the overall plastic rotation of the connection is limited. However the level of the indices are not very high; brittle fractures at the weld-column interface, therefore, were probably due also to large pre-existing flaws at the welding. Moreover, the recourse to notch-tough weld metal and improvements in welding details are essential for increasing the fracture resistance of the beam flange welds. From these preliminary results it is quite clear that appropriate design measures can exploit energy dissipation either in the panel or in the beam and it seems convenient to have both elements dissipating at the same degree.

The above results show that a correct modelling of the structure is essential to the purpose of predicting local damage in the beam and the panel zone. Moreover, analyses point out that a limited yielding in shear of the panel zone preserves the beam from large damage. From the point of view of codes, however, the reinforcing of the panel zones is mandatory.

5. FINAL REMARKS

Several cases of fractures propagating from either the heat affected zone around the weld or the weld itself have been reported in Northridge and Kobe and constitute a possible failure mode for welded connections. Fracture of the beam flange in the region of the cope has also been observed in a number of experimental tests on connections. Shear yielding of the panel zone and flexural yielding of the beam are the primary yield mechanisms for welded connections. The experimental research has shown that connections with either flexural yielding or panel zone yielding can develop significant plastic rotations.

Although panel zone yielding is a good energy dissipation mechanism, excessive column deformation can cause negative effects, such as column flange kink or permanent distortion of the column. Excessive shear deformation of the panel zone seldom results into the collapse of the panel itself; large deformations, however, are frequently very demanding for the welds and contribute to local fractures in the connection. Furthermore, large plastic deformation of the panel zone may produce inelastic shear buckling, with significant loss of both resistance and stiffness. The results of this study suggest that it is important to keep under control the extent of the panel zone deformation in order to avoid early occurrence of fracture. A tentative recommendation is to limit the contribution of the panel zone to the connection rotation to 50% of total. Yielding of the panel zone, indeed, was observed in the experimental and numerical results, with panel rotations comparable to those recorded for the beam, especially at the second storey; both the panel zone and the beam, therefore, contributed to the connection plastic deformation. Properly detailed panel zones possess good inelastic deformation capacity. Further studies of this issue are required and research, indeed, is on–going. However, it is believed that panel zone yielding might be acknowledged by design codes as a positive mechanism, acting in favour of ductility; at the same time, it must be clear that inelastic panel zone deformations need to be kept under control, as overly large deformations may result into an increased potential for connection fracture. Moreover, the use of notch-tough weld metal and the improvements in welding details are essential for increasing the fracture resistance of beam flange welds.

REFERENCES

1. Bernuzzi C., Calado L., Castiglioni C.: Behaviour of steel beam-to-column joints under cyclic reversal loading: an experimental study. Stability and ductility of steel structures, Elsevier Science, 1998.
2. Bertero V., Anderson J., Krawinkler H.: Performance of Steel Building Structures During the Northridge Earthquake. Report N. UCB/EERC- 94/09, 1994.
3. Castellani A., Castiglioni C., Chesi C., Plumier A.: A European Research Program on the Cyclic Behaviour of Welded Beam-to-Column Connections. Los Angeles, Northridge Earthquake Engineering Conference, Proceedings, August 1997.
4. Civjan S.A., Engelhardt M. D., Gross J. L.: Experimental program and proposed design method for the retrofit of steel moment connections. New Zealand, 12th World Conference on Earthquake Engineering, Proceedings, paper N° 0257, 2000.
5. Hiroshi M., Tanaka A.: Statical characteristics of the improved rigid beam-to-column connections of steel structures. New Zealand, 12th World Conference on Earthquake Engineering, Proceedings, paper N° 0213, 2000.
6. Lozza S., Peano A., Pellegrini R., Castellani A., Castiglioni C., Chesi C., Colombo A., Negro P., Taucer F., Plumier A., Carydis P.: Improved assessment of steel buildings performance during earthquakes – Steelquake. Contract ENV4-CT96-0278, 1998.
7. Mahin S.A., Hamburger R.O., Malley J.O.: National Program to Improve Seismic Performance of Steel Frame Buildings. Journal Perf. of Const. Facilities, ASCE, 12(24), pp. 172-179, 1998.
8. Matsuo A., Nakamura Y., Takamatsu T., Matsui Y.: Fracture and plastic deformation capacity of the welded joint of composite beam-to-steel column connections. New Zealand, 12th World Conference on Earthquake Engineering, Proceedings, paper N° 0181, 2000.
9. Mazzolani F., Piluso V.: Theory and design of seismic resistant steel frames. Chapman and Hall, E&FN SPON, 1996.
10. Naeim F., Skliros K., Reinhorn A. M.: Influence of hysteretic deteriorations in seismic response of multistory steel frame buildings. New Zealand, 12th World Conference on Earthquake Engineering, Proceedings, paper N° 0216, 2000.
11. NIST: Failure Analysis of Welded Steel Moment Frames Damaged in the Northridge Earthquake. National Institute of Standards and Technology, Report NISTIR 5944, January, 1997.
12. Ostertag C.P.: Microstructural Characterization of Fractured Steel Beam Column Connections. Forthcoming in Journal of Materials in Civil Engineering.
13. Plumier A., Agatino M., Castellani A., Castiglioni C., Chesi C.: Resistance of steel connections to low-cycle fatigue. Paris, 11th European Conference on Earthquake Engineering, Proceedings, 1998.
14. Popov E., Bertero V.: Cyclic loading of steel beams and connections. ASCE, Journal of the Structural Division, 1973.
15. Popov E.P., Blondet M., Stepanov L., Stojadinovic B.: Full Scale Steel Beam-Column Connection Tests. Sacramento (CA), SAC Technical Report 96-01, Part 2, SAC Joint Venture, pp. 4-1 to 4-129, 1996.
16. Roeder C., Foutch D.: Experimental results for seismic resistant steel moment frame connections. ASCE, Journal of Structural Engineering, pp. 581-588, 1996.
17. SAC: Connection Test Summaries. Sacramento (CA), Report No. SAC-96-02, SAC Joint Venture, 1996.
18. Taucer F., Negro P., Colombo A.: Cyclic testing of the steel frame - Final Report. Elsa Laboratory, Joint Research Centre, Ispra, Italy, Special Publication N. I.98.160, 1999.
19. Tsai K., Popov E.: Steel beam column joints in seismic moment resisting frames. University of California at Berkeley, California, USA, Report n. UCB 99/19, 1999.
20. Warmka G. R., Truman K. Z. 2000: Improving the performance of steel beam-column moment-resistant connections. New Zealand, 12th World Conference on Earthquake Engineering, Proceedings, paper N° 0009, 2000.
21. Wei-Ming Chi, Deierlein G. G.: Fracture toughness demands in standard and reduced beam section welded moment connections. New Zealand, 12th World Conference on Earthquake Engineering, Proceedings, paper N° 0181, 2000.
22. Zandonini R., Bernuzzi C., Bursi O.: Steel and steel-concrete composite joints subjected to seismic actions – general report. Kyoto, Japan, STESSA 97 - Behaviour of Steel Structures in Seismic Areas, 1997.