0498

COMPOSITE SUBSTRUCTURES WITH PARTIAL SHEAR CONNECTION: LOW CYCLE FATIGUE BEHAVIOUR AND ANALYSIS ISSUES

Oreste S BURSI[1] And Roberto CALDARA[2]

SUMMARY

Eleven pull-push specimens with commonly used headed stud shear connectors and six full-scale steel-concrete composite substructures with full and partial shear connection were tested. Inelastic responses to different sets of variable, constant and random reversed displacements were analysed and the importance of cyclic loading on the strength softening, on the ductility deterioration and on the structural damage of components and members was scrutinized. Thereby, some design conclusions have been drawn for members with composite dissipative zones and different fatigue life models have been investigated to establish suitable indices of structural damage. Moreover, finite element simulations set both to reproduce composite beam behaviour and to evaluate different modelling assumptions as well as local effects were carried out. In detail, analytical issues related to the effective width of concrete slab, to the force-slip relations of shear connectors and to the bond-slip phenomenon relevant to longitudinal reinforcing bars were investigated.

INTRODUCTION

The behaviour of composite structures represents nowadays a many-faceted subject. Indeed, many books and a vast number of journal articles, see Viest et al. (1997) and Leon (1998) among others, have dealt with that topic. Nonetheless, the earthquake resistant design of steel-concrete composite structures is still hampered by inadequate design code provisions. Indeed, the part of Eurocode 8 (EC8) (1994) regarding steel-concrete composite systems appears yet as an informative Annex owing to the lack of data. Thereby, extensive experimental and numerical research into the seismic resistance of composite members and structures under simulated-earthquake conditions is carried on (Bouwkamp et al., 1998). On the North-American side, a major step taken in recent years was the development of the U.S. code on seismic design of composite structures, viz. the NEHRP provisions (1997). With regard to composite beams, equations are provided to guarantee ductility in composite dissipative zones for beams that are intended as part of the primary lateral-force-resisting system. In the above-mentioned codes, the Eurocode 4 (EC4) (1992) and the LRFD (1993), the design may rely on the partial shear connection notion, and, thereby, the slip between the concrete slab and the steel beam can not be overlooked. Moreover, connectors become inelastic more quickly than their rigid counterpart under moderate or strong earthquakes and, thereby, also considerations regarding their absorption capacity become remarkable. In these conditions, the connector ability to dissipate energy depends on its capability to withstand low-cycle fatigue during the seismic event. Thereby, a better understanding of the low-cycle fatigue behaviour of shear connection is needed if members have to be designed embodying composite dissipative zones. Consideration of the low-cycle fatigue behaviour requires the availability of data for connectors loaded cyclically to failure. The bulk of available useful data, however, is for connectors loaded monotonically with only few exceptions. See the work of Astaneh et al. (1993) with regard to push-type specimens and of Bouwkamp et al. (1998) with reference to beam specimens among others. However, no analytical models were developed in the authors' knowledge for the prediction of the low-cycle fatigue behaviour of shear connectors. The adoption of partial shear connection increases also the complexity of mathematical models for the analysis as well as the formulation of structural finite elements. So far, one-dimensional (1D) beam models based on fibre concepts and on the Navier-Bernoulli beam theory provide the best compromise between accuracy and computational expense. Along this line, beam

[1] *Department of Structural and Mechanical Engineering, University of Trento, Trento, Italy Email Oreste.Bursi@ing.unitn.it*
[2] *Department of Structural and Mechanical Engineering, University of Trento, Trento, Italy*

finite elements were developed by Ayoub and Filippou (1997) and Salari et al. (1998) among others, in which distributed shear forces were embodied at the steel beam-concrete slab interface. Conversely, Manfredi et al. (1999) modelled continuous composite beams by means of discrete connector shear force-slip properties and solved the resulting system of differential equations in finite-difference form. Nevertheless, all the aforementioned studies miss the clarification of practical modelling and analysis issues as well as the consequences of cyclic loading. The study presented in this paper extends the analysis initiated by Bursi and Ballerini (1996) toward the assessment of the seismic vulnerability of composite members endowed with dissipative zones and partial shear connection. The main focus of this study is the revision of the cyclic behaviour of push-type and composite beam specimens under constant, variable and random amplitude cycles. Experimentally obtained data are then exploited to provide some design indications for composite dissipative zones and to validate cumulative damage models. From a modelling standpoint, it is apparent that the effective width of the concrete slab, the force-slip relations of shear connectors and the bond-slip phenomenon relevant to longitudinal reinforcing represent basic analytical issues. To ascertain their effects on the performances of 1D beam elements endowed with non-linear behaviour, numerical simulations were conducted with more complex and accurate three-dimensional (3D) and two-dimensional (2D) beam models.

PULL-PUSH AND BEAM SPECIMEN BEHAVIOUR

The first step of this investigation intended the shear strength and displacement ductility of headed stud shear connectors to be determined. Thereby, eleven elemental push-type specimens divided into two series were fabricated. Beam stub and concrete slab characteristics were similar to those of companion steel-concrete composite beams (Bursi and Gramola, 1999b), according to the EC4 requirements for specific push tests (1992). The geometry is depicted in Fig. 1a whereby the ribs of the steel sheeting are parallel to the beam axis in order to enhance the connector ductility. Details on the profiled steel sheeting, slab reinforcement, material properties, test set-up and instrumentation can be found in Bursi and Gramola (1999a). Suffice it to emphasize that the testing equipment should provide symmetric boundary conditions when specimens are subjected to cyclic loading. Nelson studs were endowed with a shank diameter of 15.9 mm and a mean height of 101.7 mm. Specimens of both series were loaded in a quasi-static fashion both monotonically and cyclically by a suite of sequential-phased displacement histories. The so-called complete testing procedure proposed by the ECCS (1986) was adopted as well as a cumulative damage testing programme. Whilst the first type of procedure is exploited to acquire data on the specimen capacity, the second type of programme permits, by means of a cumulative damage model, component performances under arbitrary loading histories to be predicted. For the sake of brevity, only some of the specimen responses are commented upon. The reaction force-slip response relevant both to the NPM-02 and to the NPC-02 specimen is depicted in Fig. 2. The inelastic behaviour of the NPM-02 specimen in the monotonic regime is governed by stud shearing and concrete crushing. With regard to the NPC-02 specimen, both stiffness and strength of stud connectors reduce at all stages owing to cyclic yielding and fatigue cracking in the studs as well as to propagation and coalescence of microcracks in concrete. At present, design codes do not predict the shear strength of stud connectors in a low-cycle high displacement regime and, thereby, it is worthwhile to quantify their accuracy in such conditions. The ideal shear strength predicted by the code has been defined as the one provided by using measured rather than nominal material properties. The corresponding values obtained with EC4 (1992) as well as with AISC (1993) without partial safety factors are depicted in Fig. 3. The overestimation provided by the non-seismic code EC4 (1992) is evident as a result of reversed displacement effects and amounts to 11 per cent for the NPM-02 specimen and to 56 per cent for the NPC-02 specimen, respectively. By accounting for the partial safety factor, these and additional tests conducted by Aribert et al. (1998) draw the conclusion that the design resistance of headed stud shear connectors in dissipative zones can be obtained from the design resistance provided by EC4 (1992) applying a reduction factor equal to 0.75. Moreover, the cyclic slip capacity can be derived from the one relevant to a monotonic push-type test applying a reduction factor equal to 0.5.

Composite substructures were designed with hinges located at the midspan and midheight of beams and columns, viz. the inflection points. In detail, these boundary conditions are drawn in Fig. 1b. Since the aim of the research was the investigation of the composite action in the cyclic regime, a conventional degree N/N_f of shear connection equal to 1.36, 0.68 and 0.41 was exploited for the composite beams with full shear connection (FC), intermediate partial connection (IPC) and low partial connection (LPC), respectively. For each degree of shear connection two specimens were tested (Bursi and Zandonini, 1998). The FC and IPC specimens exhibited similar seismic performances whilst the LPC substructures were designed and tested in a second phase in order to cover the range of shear connection of practical interest. The geometrical characteristics of composite substructures are also illustrated in Fig. 1b, schematically. They represent a typical European design in which there is no encasement of steel beams. Both the IPE 330 beam and the concrete slab characteristics are similar to those of the companion pull-push specimens described above. #8 12-mm longitudinal reinforcements were adopted and provided a reinforcement ratio of 0.81 per cent whilst around the node the ratio was increased to 1.78 per cent to be able to transfer to the column the overall bending moment. Other details can be found in Bursi and Gramola (1999b). Again TRW Nelson studs were adopted in this test programme with a shank diameter of 15.9 mm and a mean height of 101.7 mm. Moreover, both strength and ductility of connectors were enhanced by using large stud spacing and ribs parallel to the direction of shear flow. The instrumentation that forms integral part of the test set-up is illustrated in Fig. 1b, too. In detail, the interface slip between the steel beam and the concrete slab was detected by means of coupled LVDTs located at Secs. 1, 2, 3 and 4 whilst the uplift was measured at Secs 1, 2, 3, 3', 4 and 5. To estimate internal forces in the steel beams, flange strains were recorded by means of linear strain gauges located at Secs. 2 and 4. At these sections also axial deformations of reinforcing bars were monitored too. Thereby, the effective width of the reinforcing bars was scrutinized at each loading stage. With regard to the displacement test procedures, the so-called short testing procedure proposed by the ECCS (1986) and the pseudo-dynamic testing procedure were exploited. The former is adopted to acquire data on the specimen capacity, such as the maximum strength, ultimate displacement ductility, maximum absorbed energy, etc. The latter entails strength, ductility and energy absorption capabilities of specimens for a given random excitation to be assessed. Other details regarding the composite beam tests can be found in Bursi and Zandonini (1998) and Bursi and Gramola (1999b).

For the sake of brevity, only the most significant results relevant to the FC and the IPC substructure exposed to the short testing procedure are commented upon. The hysteresis loops of the reaction force developed by the FC substructure vs. the controlled displacement are plotted in Fig. 4a. Cycles are stable to a certain extent. Moreover, the inelastic hysteretic behaviour exhibited by the specimen is governed by steel beam yielding for positive (pull) loads and reinforcing bar yielding as well as concrete fracturing for negative (push) loads. In line with the EC4 (1992) predictions, based on the limiting breadth over thickness ratios for Class 1 sections, web and then flange buckling occurred at the first negative hemicycle characterised by a partial displacement ductility ratio e/e_y^- equal to 4. Both web and flange local buckling happened at 250 mm from the column flange beyond the reinforcing plates (see Fig. 1b). However, the buckling in alternate cycles did not reduce substantially the substructure load-carrying capacity. As a result, positive as well as negative ductility levels equal to 6 were reached. Under the assumption of a storey height of 3.5 m, one can consider the allowable interstorey drift limits suggested by some codes, viz. 2.0% or 2.5%, respectively. These limits are important to serviceability in order both to not impair the stability of the structure and to provide added strength and stiffness in moment frames. The substructure behaviour was satisfactory with respect to drift (Bursi and Zandonini, 1998). The corresponding reaction force-controlled displacement loops of the IPC substructure are reported in Fig. 4b. In this test hysteresis cycles are stable too. However, web and flange buckling revealed at the first negative hemicycle with partial ductility ratio equal to 4. In addition, at the third positive hemicycle of the same set, the beam-to-column joint fractured near the weld beads. The resistance of composite sections was analysed by using a rigid plastic (stress-block) approach in accordance with the conventional limit states collected in Table 1. The mean reaction forces without safety factors as predicted by EC4 (1992) and corresponding to the maximum resistance provided by composite substructures in the pulling and pushing regime were computed, respectively. These values are indicated in Fig. 4 by using an effective width equal to 820 mm averaged between EC4 (1992) and LRFD (1993) indications. The ratio between the aforementioned resistances without partial safety factors to the plastic failure strength of the specimens is about 1.2. This ratio is about 1.0 for the LPC specimen. The shortcoming of EC4 (1992) predictions depends on different combined phenomena. On one hand, the shear connectors do not reach the ultimate strength at the plastic failure load as implied by the code whilst, on the other hand, the actual strength of specimens decays through an energy release process which results from concrete crushing. The main results provided by the three companion substructures subjected to pseudo-dynamic tests (Bursi and Zandonini, 1998) are commented upon hereinafter. Due to the peak ground acceleration that caused specimen collapse the energy absorption properties of specimens under pull loading were not fully exploited. Conversely, web and flange buckling determined failure and reduced the specimen energy consumption characteristics. Local buckling phenomena were very akin to those observed in the corresponding quasi-static cyclic tests. Nevertheless, a satisfactory behaviour of the specimens with no resistance loss up to the interstorey drift limits was observed. The design resistances without safety factors predicted by EC4 (1992) and corresponding to the maximum resistances of composite beams were computed too. The overestimation with respect to the plastic failure resistance reached about 30 per cent for FC and IPC specimens and about 40 per cent for the LPC specimen. It is evident that predictions are unsafe when random-amplitude displacements determine the structural response. Indeed, the substructures experienced a large plastic excursion in the pulling regime owing to the N69W component of the 1952 Taft earthquake. Thereby, a lowering of the yield stress upon reloading owing to the Bauschinger effect happened in the hogging moment region. From cyclic and pseudo-dynamic test data, one may draw the conclusion that FC and IPC specimens exhibited similar performances.

These results confirm the statements of Richard Yen at al. (1997) based on low-cycle fatigue experimental data. They stated that no practical difference in terms of strength and displacement ductility exists between composite beams with full and 80 per cent of the shear connection degree. Thereby, from a design standpoint one may conclude that if connectors are ductile according to EC4 (1992) rules, partial shear connection may be adopted safely in composite dissipative zones with a minimum connection degree of 80 per cent. Conversely, full shear connection is required when non ductile connectors are exploited. Moreover, the results of Cosenza et al. (1997) have shown clearly that only high ductile slab reinforcement steel, like the Ductility class M or H defined in EC8 (1994), must be exploited in hogging moment regions to improve both bearing and plastic rotation capacity of steel-concrete composite beams.

LOW-CYCLE FATIGUE BEHAVIOUR

Theories of plasticity are able to characterize the material behaviour in the inelastic regime owing to cyclic loading. However, these models do not typically include the notion of failure. Yet it is well known that a dissipative structural component subjected to cyclic loading will often fail due to low-cycle fatigue, viz. a damage process which results from a limited number of excursion, typically less than 1000, well into the inelastic range. Thereby, a considerable amount of effort has been expended in defining suitable damage parameters to obtain the best correlation between the damage parameter and the life to failure. In view of damage model validation, it is deemed to be necessary to define failure, viz. to evaluate the cycle number N_f that entails failure. Due to the large uncertainty in the failure definition, three different criteria are adopted in this study and are collected in Table 1. The first two of them are based on an energy approach. The importance of the energy approach relies on its ability to unify microscopic and macroscopic testing data and to formulate multiaxial failure criteria. In order to define a damage limit state Calado e Castiglioni (1996) proposed an energy ratio. In detail, the ratio between the absorbed energy at the last cycle before failure and the energy that might be absorbed in the same cycle if the component would exhibit an elastic-perfectly-plastic behaviour over the same ratio with reference to the first cycle in the inelastic range. With reference to the total displacement range of each cycle, Bernuzzi et al. (1997) defined failure by means of the so-called relative energy drop. Lastly, the criterion of Chai et al. (1995) was exploited. It relies on the design assumption that failure happens when the degraded resistance approaches the plastic failure resistance. All the above-mentioned criteria were applied to the specimens under investigation. With regard to the response provided by the ECCS procedure (1986), a cycle number N_f of about 50 for the pull-push specimens and of about 16 for the composite beam specimens was evaluated, respectively. Once failure is defined, it is appropriate to associate the final state of the specimen under cyclic loading to a unique point of an equivalent monotonic response. In detail, it is deemed to be necessary the definition of a damage indicator, viz. a state variable that enables a one-to-one correspondence from any damaged state caused by cyclic or dynamic loads to a unique point of a monotonic specimen response. In these conditions, the safety assessment is straightforward being unique the distance from failure. In detail, a damage index is introduced, viz. a normalized damage indicator, for which zero corresponds to the virgin state and one to the achievement of failure in agreement with the assumed criterion. Different choices relevant to damage indicators are available. For the sake of brevity only the approach proposed by Bernuzzi et al. (1997) is commented hereinafter. Wöhler’s approach is the basis of the aforementioned approach with the adoption of a global parameter like the total displacement range. In the log-log scale, the ensued damage relation exhibits a slope of –1/3. As the approach relies on the Miner’s rule for cumulative damage, the interrelation between displacement and force (path dependency) is overlooked with such a model. Thereby, it is deemed to be necessary the application of a cycle counting method like the rainflow or the reservoir technique. The adoption of the damage model proposed by Bernuzzi et al. (1997) is illustrated in Fig. 5. Damage lines are relevant to the second test series of pull-push specimens (Bursi and Gramola, 1999a) and are endowed with a slope of – 0.40 for D = 1.0. This value is very close to the theoretical value –1/3 adopted for steel components. To evaluate specimen substructures in terms of low-cycle fatigue damage as well as to establish a correlation between pull-push and composite substructure specimens, composite beam data were re-evaluated according to the above-mentioned approach too. The degree of correlation between pull-push and composite substructures is provided in Fig. 5 in terms of interface slip *s*. One may observe that the damage state of LPC specimens which has been caused by stud connector failure is very close to the pull-push damage domain. This trend implies a correlation between pull-push specimen and composite substructure damage. Thereby, damage domains calibrated by means of pull-push tests could be employed for connector design in dissipative composite members under cyclic loading. Clearly, such approach needs a validation for different connector sizes.

FINITE ELEMENT MODELLING ISSUES

The behaviour of composite beams with partial shear connection exposed to large alternate actions should not be scrutinized only through experiments. Indeed, the underlying state of stress among structural steel, reinforcing bars and concrete at the interface is very difficult to capture. Thus far, the finite element models devoted to the analysis of the non-linear behaviour of composite members are either based on 3D and 2D elements, which represent in great detail the component parts of the structural elements, or 1D beam elements that capture only the salient feature of the non-linear behaviour. The validation phase was based on measured material data. However, owing to the presence of different materials in the LPC specimens, all the simulations commented upon hereinafter adopt a reference material that corresponds to the one relevant to the FC specimens. This represents a clear advantage of the finite element analysis. The main issues commented here are the effective width of concrete slab, the constitutive shear law of headed stud connectors and the bond stress-slip phenomenon relevant to reinforcing bars. To perform 3D analyses, a model assembled with the ABAQUS code (Hibbitt et al., 1998) and endowed with shell elements S4 both for the steel beam and concrete slab was adopted. Beam elements B31 were used to model reinforcing bars whilst non-linear spring elements SPRING2 were employed to reproduce headed stud shear connectors as well as the interaction between the concrete slab and the reinforcing bars. Moreover, linear truss elements T3D2 were employed to model uplift. The 2D model exploits plane stress elements CPS4I to reproduce the steel beam and the concrete slab whilst reinforcing bars are modelled with truss T2D2 elements. Connectors are reproduced with SPRING2 elements. 1D analyses were performed with the DRAIN-3DX code (Campbell et al., 1994). Inelastic fibre beam-column elements with distributed plasticity and with five fibre each were used to model the steel beam and the concrete slab. The heavy steel column was reproduced with an elastic beam-column element whilst the reinforcing bars are modelled by means of a distributed representation. Headed stud shear connectors as well as the uplift were simulated by means of joint elements. In 3D and 2D models a macrolevel approach was adopted for concrete fracture whilst the plain concrete was assumed to be an equivalent isotropic continuum with smeared cracks (Hibbitt et al., 1998). The material model exploited for 1D elements was the one available in the DRAIN-3DX code (Campbell et al., 1994).

To perform 2D and 1D analyses an effective width has to be provided to the models. Thereby, 3D simulations allowed the effective width of the concrete slab to be estimated. The distribution of the compressive stresses in the slab relevant to a displacement ductility ratio e/e_y^+ of 4.3 is plotted in Fig. 6 relevant to the pull regime. Clearly, the effective width reduces toward the simply supported beam end (see Fig. 1b). Given the midspan L equal to 4000 mm, the average effective width is about 0.16L. Conversely, it is about 0.3L, viz. the actual slab width, in the push regime. As expected, the effective widths exhibit different trends in the two inelastic regimes, implying that this effect must be accounted for when 2D and 1D simulations are performed. Moreover, effective width values result to be smaller than the ones adopted to predict the beam plastic failure resistance. A 1D simulation is depicted in Fig. 7, whereby the actual effective width evaluated with the slab in compression has been used. The FC specimen is presented here as the interaction between the steel beam and the concrete slab is very large. One may observe that experimental data and numerical predictions are in a good agreement with each other. Clearly, the 1D model is unable to capture the strain-softening behaviour of the substructure in the push regime caused by the local buckling of the steel beam web and of the flange, respectively. Conversely, an overstrength effect can be observed when the effective width resulting from the slab in the tensile regime is adopted. Thereby, cyclic simulations conducted with 1D and 2D models require the exploitation of the effective width evaluated from the concrete slab in compression, being the influence of the concrete in the tensile regime much less important. When simulations of composite beams under cyclic loading are performed, the choice of the force-slip relation to be adopted for stud shear connectors is not obvious. Indeed, a large difference can be observed in Fig. 2 in terms of strength and displacement ductility between the shear response of connectors subjected to monotonic and cyclic loading, respectively. If the skeleton of the cyclic force-slip response is adopted in the 3D model, a reduction of the response in the inelastic regime can be observed. However, the predictions relying on the monotonic force-slip response confirm the reliability of results as highlighted in Fig. 7. Indeed, the aforementioned behaviour can be explained observing that the FC substructure approached failure with about 16 cycles. This number results to be much smaller than the number of cycles at collapse, about 50, experienced by connectors in the pull-push test. Thereby, because the correlation between the monotonic response of a connector and the cyclic response of the substructure is stronger in terms of low-cycle fatigue prediction, observe Fig. 5, it is evident that the simulations are more accurate when monotonic force-slip laws for connectors are exploited. As far as the bond-slip phenomenon between concrete and reinforcing bars is concerned, simulations with the 3D model embodying the bond stress-slip behaviour were conducted. It was observed that the influence of the bond stress-slip phenomenon on the force-displacement response is not remarkable when the concrete slab is subjected to tensile stresses. Indeed, the significant amount of 12-mm transversal reinforcing bars located at 200 mm, needed to avoid the longitudinal shear failure or the longitudinal splitting of the slab (EC4, 1992) prevented the bond stress-slip phenomenon. Thereby, though the bond-slip behaviour was not accounted for in the 1D model owing to the distributed representation of reinforcing bars, apart from the local buckling, the simulation highlighted in Fig. 7 results to be satisfactory in the pushing regime.

CONCLUSIONS AND OUTLOOK

This paper has summarized the experimental and numerical activity conducted at the University of Trento over the last three years, on the field of members with partial shear connection under cyclic loading. Experimental results pertaining to elemental pull-push and composite substructure specimens imply a satisfactory behaviour of dissipative composite beams with partial shear connection. In detail, pull-push test data confirm that the design resistance of ductile connectors in dissipative zones can be obtained from the design resistance provided by Eurocode 4 (1992) applying a reduction factor equal to 0.75. Moreover, the cyclic slip capacity of ductile stud shear connectors can be derived from the one relevant to monotonic push-type tests applying a reduction factor equal to 0.5. Composite beam tests confirm that a shear connection degree of 0.8 entails beam performances similar to the full shear connection specimens. Thereby, from a design standpoint the partial shear connection concept can be adopted safely also in beam elements endowed with dissipative zones.

Quite new is the treatment of experimental data relevant to pull-push and beam specimens in terms of damage. It appears that standard damage models can be applied to composite members too. Indeed, a correlation in terms of damage between pull-push and composite beam specimens has been determined when the damage limit state governs the behaviour of dissipative composite beams. Moreover, numerical analyses have emphasized important modelling issues such as the effective width relevant to the concrete slab, the proper modelling of the shear connection in terms of constitutive law and the bond stress-slip relation between reinforcing bars and slab. Indeed, a large amount of transversal reinforcing bars needed to avoid the longitudinal shear failure or the longitudinal splitting of the slab prevents the bond-slip phenomenon of the longitudinal reinforcing bars. Insofar as future activities are concerned, an experimental campaign encompassing push-type and dissipative substructure specimens embodying 22 mm headed stud shear connectors is carried on in order to support the above-mentioned conclusions.

REFERENCES

American Institute for Steel Construction, (1993), *Load and Resistance Factor Design - Specifications for Structural Steel Building*, 1.

Aribert, J.M., Lachal, A. and Tahoszki, Z.S. (1998), “Tests on welded and nailed Hilti connectors. Test procedure. Main results. Cyclic behaviour.”, Report Lab. of Struct. and App. Mech. INSA Rennes, France.

Astaneh-Asl, A., McMullin, K.M., Fenves, G.L. and Fukuzava, E. (1993), “Innovative semi-rigid steel frames for control of the seismic response of buildings”, Report UCB/EERC-93/03.

Ayoub, A. and Filippou, F.C. (1997), “A model for composite steel-concrete girders under cyclic loading”, *Build. to Last- Proc. of Struct. Congr. XV,* Portland, 1, 721-725, ASCE, New York.

Bernuzzi, C., Calado, L. and Castiglioni C. A. (1997), “Ductility and load carrying capacity prediction of steel beam-to-column connection under cyclic reversal loading”, *J. of Earthquake Engng*, 1(2), 401-432.

Bouwkamp, J.G., Parung, H. and Plumier, A. (1998), “Bi-directional cyclic response study of 3-D composite frame”, in “Eleventh European Conf. on Earthquake Engineering”, *Proc. Europ. Conf.*, Paris, September.

Bursi, O. and Ballerini, M. (1996), “Behaviour of a steel-concrete composite substructure with full and partial shear connection”, *Proc. 11th World Conf. on Earthquake Engng.* Pergamon, Paper No. 771, Elsevier, Oxford.

Bursi, O.S. and Zandonini, R. (1998), “Quasi-Static Cyclic and Pseudo-Dynamic Tests on Composite Substructures with Softening Behaviour”, selected paper in *Stability and Ductility of Steel Structures,* edited by Usami and Itoh, Elsevier, 119-130.

Bursi, O. S., and Gramola, G. (1999a), “Behaviour of headed shear connectors under low-cycle high amplitude displacements”, *Materials and Structures*, 32, 218, 290-297.

Bursi, O. S., and Gramola, G. (1999b), “Behaviour of composite substructures with full and partial shear connection under quasi-static cyclic and pseudo-dynamic displacements”, *Materials and Structures*, (in print).

Calado, L. and Castiglioni, C.A. (1996), “Steel beam-to-column connections under low cycle fatigue: experimental and numerical research”, *11th WCCE,* Acapulco, Paper No. 1227.

Campbell, S., Powell, G. H. and Prakash, V. (1994), *DRAIN-3DX Base program description and user guide - Version 1.10*, Report No. UCB/SEMM-94/07, University of California, Berkeley.

Chai, Y. H., Romstad, K. M. and Bird, S. M. (1995), “Energy-based linear damage model for high-intensity seismic loading”, *J. Struct. Engrg.*, ASCE, 121(5), 857-864.

Cosenza, E., Fabbrocino, G. and Manfredi, G. (1997), “The influence of the rebar ductility on the rotational capacity of composite beams”, *5th Int. Colloquium on Stability and Ductility of Steel Struct.*, Nagoya, 879-886.

ECCS (1986), *Recommended Testing Procedures for Assessing the Behaviour of Structural Steel Elements Under Cyclic Loads,* Technical Committee 13, No. 45.

Eurocode 4 - ENV 1994-1-1 (1992), *Design of composite steel and concrete structures - Part 1-1: General rules and rules for buildings*, European Committee for Standardisation.

Eurocode 8 - ENV 1998-1-3 (1994), *Design provisions for earthquake resistance of structures - Part 1-3: Specific rules for various materials and elements*, European Committee for Standardisation.

Hibbitt, Karlsson & Sorensen Inc. (1998), *ABAQUS – User’s manual, version 5.7,* Vol. 1-2.

Leon, R. T. (1998), “Analysis and design problems for PR composite frames subjected to seismic loads”, *Engineering Structures*, 20(4-6), 364-371.

Manfredi, G., Fabbrocino, G. and Cosenza, E. (1999), “Modeling of steel-concrete composite beams under negative bending”, *J. of Engrg. Mech.* ASCE, 125(6), 654-662.

NEHRP (1997), *Recommended Provisions for Seismic Regulations for New Buildings*, Building Seismic Safety Council, Washington D.C.

Richard Yen, J. Y., Lin, Y. and Lai, M. T. (1997), “Composite beams subjected to static and fatigue loads”, *J. Struct. Engrg.*, ASCE, 123(6), 765-771.

Salari, M.R., Spacone, E. Shing, P.B. and Frangopol, D.M. (1998), “Nonlinear analysis of composite beams with deformable shear connectors”, *J. Struct. Engrg.*, ASCE, 124(10), 1148-1158.

Viest, I.M., Colaco, J.P., Furlong, R.W., Griffis, L.G., Leon, R.T. and Wyllie, L.A. (1997*), Composite construction design for buildings*, ASCE, McGraw-Hill.

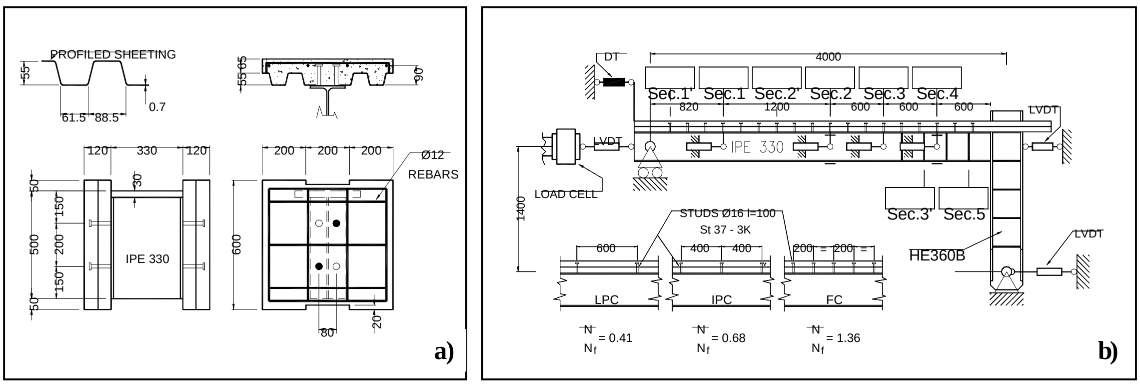

Fig. 1 - Geometrical characteristics of: a) push-type specimens; b) composite beam specimens

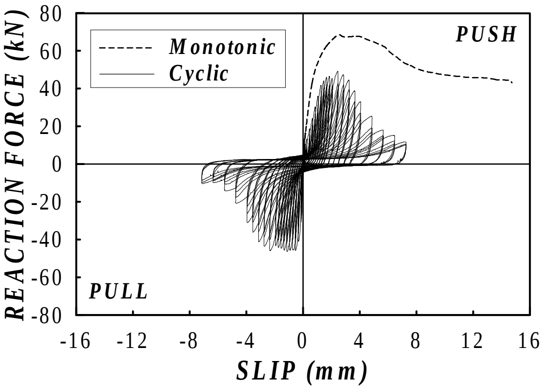

Fig. 2 - Push-off and pull-push reaction force versus controlled slip

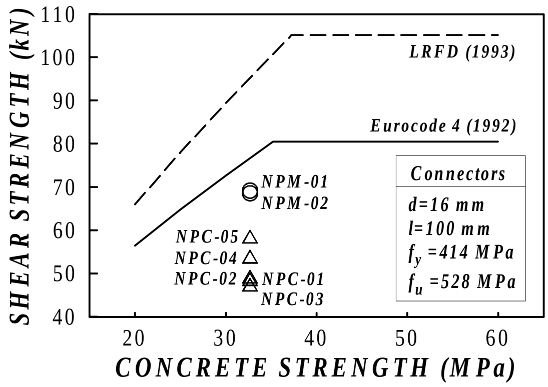

Fig. 3 - Comparison between specimen maximum strength and code prediction

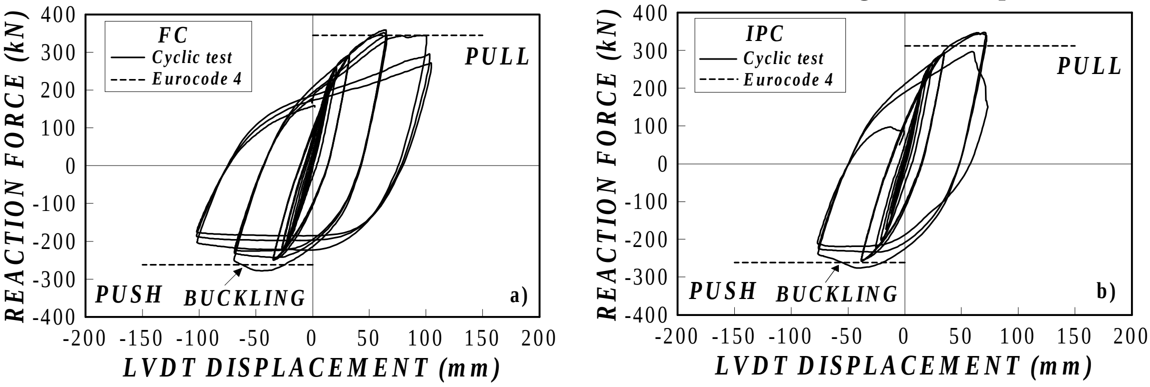

Fig. 4 - Quasi-static hysteresis loops of reaction force vs. horizontal displacement of substructures: a) FC specimen; b) IPC specimen

CONVENTIONAL LIMIT STATES	
$\varepsilon_{u,\ tens.}$	*: bottom flange of steel beam*
$\varepsilon_{u,\ compr.}$	*: top flange of steel beam*
$\varepsilon_{u,\ tens.}$	*: longitudinal rebars (concrete deck)*
Local buckling of steel beam	
Interstorey drift = 2,5%	
Node displacement = e_u	
DAMAGE LIMIT STATES	
Criterion of Calado et al. (1996)	
Criterion of Bernuzzi et al. (1997)	
Criterion of Chai et al. (1995)	

Tab. 1 - Definition of conventional and damage limit states

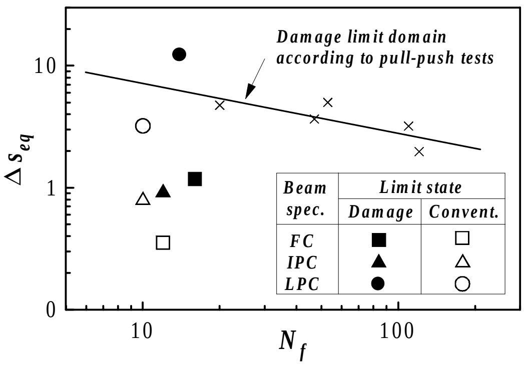

Fig. 5 - Damage correlation between pull-push and beam specimens

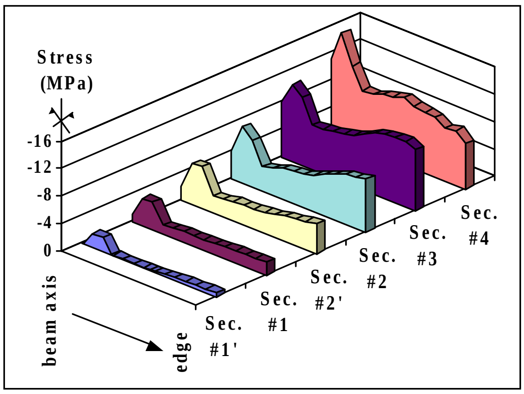

Fig. 6 - Compressive stresses in the concrete slab relevant to the FC substructure

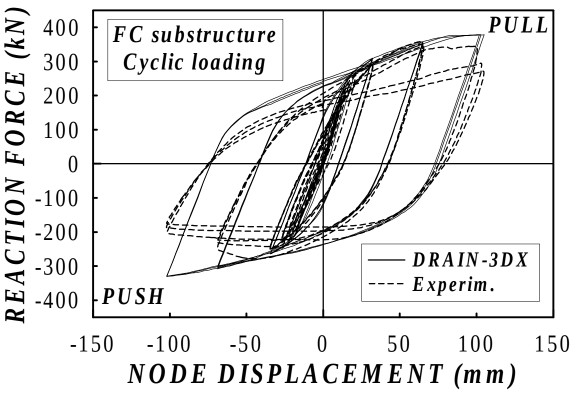

Fig. 7 - Experimental and predicted hysteresis loops with the effective width from the pull regime