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Behaviour of composite members subjected to earthquake loading

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ABSTRACT: This paper describes an advanced analysis model developed to represent the behaviour of composite steel/concrete members and frames. The formulation consists of beam-column finite elements accounting for geometric nonlinearities and material inelasticity. The nonlinear cyclic concrete model considers confinement effects and the inelastic cyclic constitutive relationships for steel include the effect of local buckling. The models are calibrated and compared with experimental data from cyclic and pseudo-dynamic tests conducted by the authors and their co-workers on a new ductile configuration of composite beam-columns. A parametric study carried out using the analytical tool is briefly described. Finally, the significance of findings from the analytical and experimental results on the seismic design process according to modern code recommendations is presented.

1 INTRODUCTION

Economical design of structures to resist strong earthquakes mandates the use of structural components responding well into the inelastic range. During severe earthquake ground motion, a structure should be able to sustain large deformations without collapse, and be capable of dissipating substantial energy through inelastic deformations of its structural members. In 'Capacity Design', key members are dimensioned to develop a carefully chosen ductile failure mode, and to demonstrate plastic deformational capability sufficient for the development of the full capacity of the whole structural system. It is therefore imperative that the full deformational and strength capacities of members are accurately evaluated and assessed through realistic testing techniques and reliable analytical investigations.

In Europe, whilst many investigations were carried out to study the behaviour of steel and reinforced concrete structures, very few were directed towards the study of composite steel/concrete structures known, particularly in Japan, to exhibit earthquake-resistant properties superior to both its components. Consequently, a research project was initiated at Imperial College to study the behaviour of partially-encased composite beam-columns, typically used in Europe for static design, subjected to cyclic and earthquake loading. This was motivated by the success of composite beam-columns in dealing with fire-resistance problems coupled with a recognised need for seismic design criteria for composite construction, to be included in the new Eurocode 8 (1988), 'Structures in Seismic Regions'.

In this paper, the nonlinear models developed to simulate the behaviour of composite members are briefly presented. Analytical simulations are compared with some results from a series of cyclic and pseudo-dynamic experiments conducted by the authors and their co-workers (Elghazouli, 1991; Elnashai et al, 1990; Elghazouli et al, 1990). In these investigations, a novel configuration of partially encased members providing enhanced ductility and energy dissipation capacity was developed and tested. The significance of the main findings of a parametric analytical study and of the experimental results are briefly discussed.

2 NONLINEAR MODELS

Use was made of the advanced nonlinear dynamic frame analysis program 'ADAPTIC' (Izzuddin and Elnashai, 1989). The program was originally developed to provide an efficient tool for the nonlinear static and dynamic analysis of two and three dimensional steel frames. Several options are available regarding the choice of elements, type of analysis and loading, and comprehensive dynamic analysis capabilities are included. The accuracy of 'ADAPTIC' has been extensively verified elsewhere (Izzuddin, 1990; Izzuddin and Elnashai, 1989; Elnashai et al, 1989).

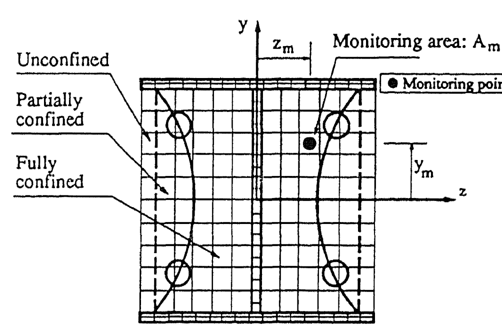

Figure 1. Monitoring areas and confinement zones for composite sections

The elasto-plastic cubic element was chosen for the analysis of composite steel/concrete frames. This element assumes a cubic shape function in the chord system, and monitors stresses and strains at various points across two Gaussian sections, allowing the spread of plasticity throughout the cross-section. As shown in Figure 1, the cross-section of any composite section is divided into a number of monitoring areas. For each area, an appropriate cyclic constitutive relationship for the material under consideration is used.

For concrete, the cyclic constitutive relationship implemented by Madas and Elnashai (1989) was used. The model is adapted from the relationships given by Mander et al (1988), and is shown schematically in Figure 2. The effect of confinement on the peak stress, peak strain and the inclination of the post-peak stress-strain relationship of the envelope curve is calculated taking into account the maximum transverse pressure from confining steel. Concrete confinement zones were suggested for several types of composite section, as shown in Figure 1 for a partially encased configuration. The confinement factor, i.e. strength enhancement factor, associated with each zone is estimated from a modification of the procedure given in Appendix A of Eurocode 8 (1988) for reinforced concrete members. The latter was shown to give results in good agreement with experiments on composite members (Elghazouli, 1991) compared to other available approaches.

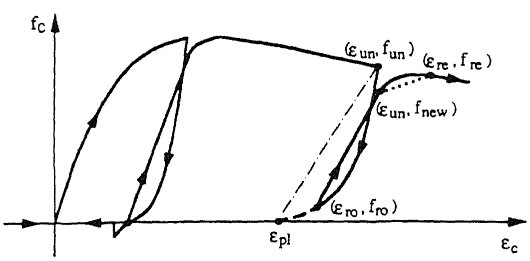

Figure 2. Cyclic constitutive model for concrete

Two cyclic models for mild steel were implemented within the elasto-plastic cubic formulation of the program. The first is the bilinear kinematic strain-hardening model, while the second is the uniaxial case of the multi-surface model originally proposed by Popov and Petersson (1978), representing exact material behaviour with cyclic degradation and mean stress relaxation. Although the former model is simple to implement and computationally more efficient, and its parameters can be more easily identified, the second model captures more realistically the behaviour of mild steel under elasto-plastic cycling (Izzuddin, 1990).

In the case of partially encased composite members, ductility is adversely affected by inelastic local buckling of the steel flanges. Consequently, a constitutive relationship for steel accounting for local buckling under cyclic loading was developed and implemented within the program (Elghazouli, 1991). This model was coupled with the more simplified bilinear stress-strain relationship for steel. The basic phenomena of local flange buckling are represented through a simple approach that can be implemented within frame analysis programs. It accounts for realistic cyclic behaviour in compression and recovery of stresses in tension following local buckling.

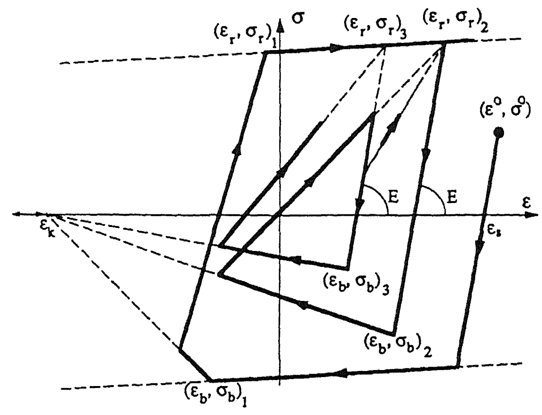

Figure 3. Cyclic material model for mild steel including the effect of local buckling

For each monitoring area of the flange sub-divisions, A_m in Figure 1, local buckling is initiated if a critical strain associated with the Euler buckling strain is exceeded. In estimating the buckling coefficient k the web is assumed to be effectively restrained by the surrounding concrete. Also, the effect of lateral bars welded to the flanges in delaying local buckling and on the post-local buckling behaviour is appropriately accounted for and calibrated with the experimental results. The stress-strain relationship of each flange monitoring area in subsequent loading and unloading following the onset of local buckling is based on the observed behaviour of steel axially loaded members (Popov and Maison, 1980; Ballio and Perotti, 1987), and is shown diagrammatically in Figure 3. Details of the formulation of the model are given elsewhere (Elghazouli, 1991).

3 COMPARISON WITH EXPERIMENTS

The analytical models were calibrated and compared with the results of a series of experiments on partially encased composite members subjected to cyclic and pseudo-dynamic loading conditions. Members with two different cross-sectional details were tested. The first, referred to as EM section, is typical of European practice with two longitudinal bars on either side of the web and confinement stirrups. The second, referred to as IC section, is a novel configuration designed to provide improved performance for earthquake-resistance. In the latter, additional transverse bars were welded to the flanges within the potential plastic hinge zone in order to inhibit or delay local buckling at large strain levels. In both cases, reduced stirrup spacing was used in the expected plastic hinge zone to provide sufficient concrete confinement. The testing programme comprised a total of fourteen EM and IC models. The members were tested as cantilevers under different values of constant compressive axial load and either lateral cyclic displacements or pseudo-dynamic earthquake loading. Typical member details are shown in Figure 4. A full account of the experimental and analytical results is given elsewhere (Elghazouli, 1991).

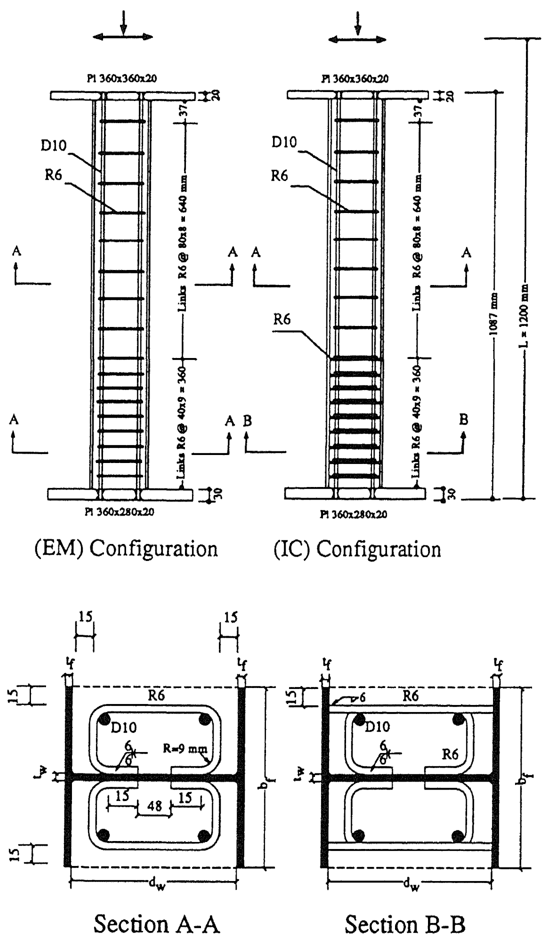

Figure 4. Details of typical EM and IC specimen

The main observation of the experimental investigation was the substantially higher ductility consistently achieved by the IC models compared to identically loaded EM models. In several cases, the curvature and rotational ductilities of the IC configuration exceeded four-fold that of the corresponding EM specimens. On the other hand, the difference in stiffness and yield capacity in both configurations was negligible. In other words, the same seismic forces would be initially attracted by both types of member, yet with a much higher ductility and energy dissipation capacity achieved by the IC configuration.

Examples of the comparison between the experimental and analytical results are given in Figures 5 and 6 for models EM02 and IC02, and Figures 7 and 8 for models EM03 and IC03. All four models were subjected to a constant compressive axial load representing 30% of axial plastic capacity. Identical lateral cyclic displacements of increasing amplitudes were applied on specimens EM02 and IC02, whereas EM03 and IC03 were subjected to the same scaled acceleration time history of the El Centro earthquake using the pseudo-dynamic technique.

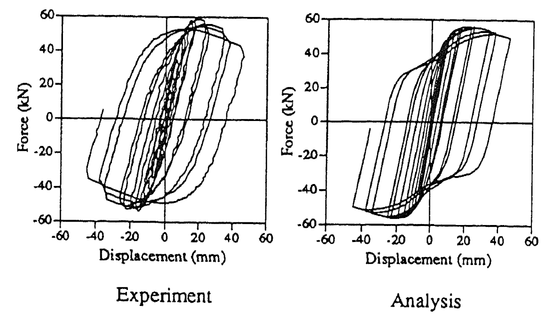

Figure 5. Experimental and Analytical results for EM02

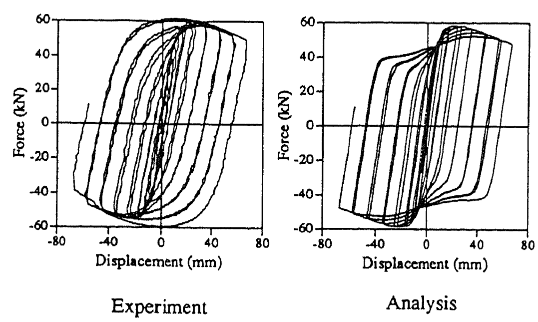

Figure 6. Experimental and Analytical results for IC02

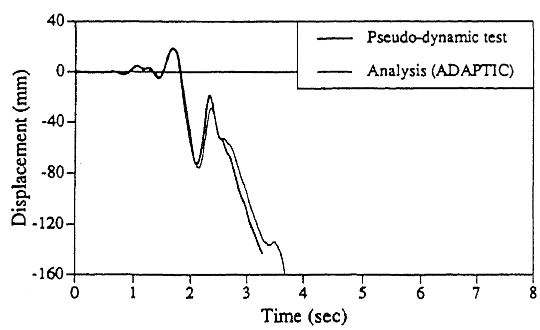

Figure 7. Experimental and Analytical Displacement response for EM03

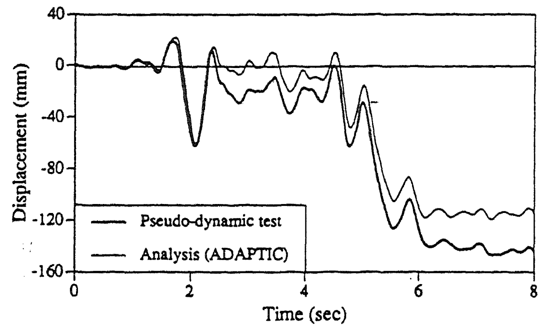

Figure 8. Experimental and Analytical Displacement response for IC03

The results clearly indicate the superior behaviour of the IC models. For EM02 and IC02, the analysis accurately predicts the stiffness, capacity and ductility. Nevertheless, discrepancies observed in the shape of the hysteresis loops are attributed to the perfect cracking mechanism assumed in the concrete model. The pseudo-dynamic tests are accurately simulated by the analysis. In general, very good correlation was achieved between the experimental and analytical results for all models.

4 DUCTILITY-BASED DESIGN

The nonlinear analytical models described above may be used in conducting reliable dynamic analyses, which are required by modern codes of practice for structures of special importance or those classified as being irregular. For regular structures, however, simplified approaches are used, whereby idealized elastic response spectra are appropriately scaled using a structural behaviour factor to account for inelastic response. This requires not only an evaluation of the available local ductility in critical members but also the relationship between the local member ductilities and the overall structural behaviour factors.

In order to assess the local ductility of partially encased members and to quantify the effect of several important parameters, an analytical parametric study was carried out using the developed analytical models. The geometry of the member was retained as a cantilever since the constituent members of a frame undergo double curvature bending which can be simulated by an assembly of cantilevers. The yield point was assumed to correspond to the yielding of both extreme fibres of the critical section, whilst failure was associated with the onset of local flange buckling. The parameters studied included the axial load, steel yield stress and strain hardening, concrete confinement, flange slenderness and member slenderness. The effect of these parameters on the yield and ultimate capacity, curvature and rotational ductilities and the length of the plastic hinge zone were extensively investigated (Elghazouli, 1991).

It was shown in both the experimental and analytical investigations that ductility decreases with the increase of compressive axial load, yield stress of steel, flange slenderness and the spacing of the bars inhibiting local buckling in IC members. Conversely, an increase in concrete confinement has a positive influence on ductility. On the other hand, the length of the plastic hinge zone was shown to decrease with the increase of compressive axial loads, steel yield stress, flange slenderness and spacing of local buckling inhibitors, whereas it increases with the increase of concrete confinement and steel strain hardening. If special details are included to enhance the ductility of critical sections, such as local buckling inhibitors or closely spaced confinement reinforcement, it should be provided within the entire length of this zone. The extent of the plastic hinge may be estimated from the yield and ultimate moment given by reliable section analysis programs. Otherwise, for detailing purposes, 40% of the height from the point of fixity to the point of contraflexure was shown to be a safe upper bound for the extent of the plastic hinge in most practical cases.

According to the general procedure adopted in Eurocode 8 rules for concrete structures, the behaviour factor is first estimated on the overall structural level. This is followed by an estimation of the displacement or rotational ductility demand of critical members. In the third stage, this rotational ductility is converted into a curvature ductility demand. The final step involves providing adequate member detailing to achieve this requirement or to check the existing curvature ductility.

A similar approach for the last two stages of the above-mentioned procedure can be outlined by which the available rotational ductility of a partially encased member can be assessed or, more importantly, the required curvature ductility is ensured for a given rotational ductility demand. The moment and curvature at yield can be easily calculated from simple equations ignoring the effect of concrete confinement and post yield steel characteristics. The ultimate moment can also be estimated using detailed section analysis programs or elaborate relationships including accurate material characteristics. The values of both moments would be needed if the extent of the plastic hinge zone is to be precisely calculated.

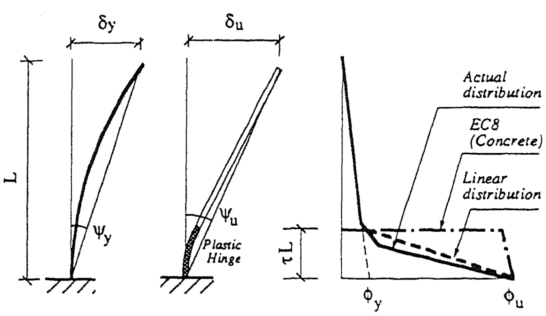

Figure 9. Curvature distribution in the plastic hinge zone

In the rules for concrete structures given in Eurocode 8, a constant curvature distribution within the plastic hinge zone is assumed, as shown in Figure 9. This approximation would tend to overestimate the rotational ductility of reinforced concrete flexural members (Pilakoutas, 1990). Using this distribution and assuming that deflection angles are relatively small, it can be shown that the relationship between the curvature ductility, D_ϕ, and the rotational ductility, R_ψ, is given by:

$$R_\psi = 3\tau\left(1 - \frac{\tau}{2}\right)D_\phi \quad (1)$$

in which:

$$R_\psi = \frac{\psi_u}{\psi_y} - 1 \qquad \text{and} \qquad D_\phi = \frac{\phi_u}{\phi_y} - 1 \quad (2)$$

where ψ_y and ψ_u are the deflection angles at yield and ultimate, respectively; ϕ_y and ϕ_u are the curvatures at yield and ultimate, respectively; τ is the plastic hinge length normalized by the height of the cantilever member.

On the other hand, if a linear distribution of curvature in the plastic hinge zone is assumed, equation (1) would be modified as follows:

$$R_\psi = \frac{\tau}{2}(3 - \tau)D_\phi \quad (3)$$

Figure 10 depicts the relationship between the actual rotational ductility obtained in the parametric study and that obtained using equations (1) and (3) above. It is clear that the linear approximation is much closer to the actual results than the constant curvature assumption. Moreover, the rotational ductilities calculated using (3) are consistently on the lower side of the actual results, and hence conservative values are always estimated. This condition, however, is not satisfied by using equation (1) and the rotational ductility may be significantly overestimated.

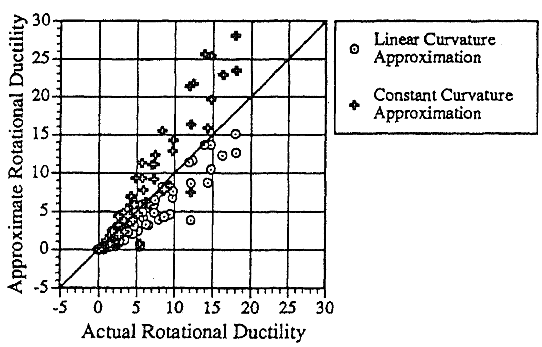

Figure 10. Relationship between actual and approximate rotational ductilities

For simplicity, if the plastic hinge length is not accurately estimated, a conservative value of τ may be assumed as 0.2 based on the results of the parametric study. By substituting this value in equation (1), the rotational ductility for the case of constant curvature approximation becomes:

$$R_\psi = 0.54\ D_\phi \quad (4)$$

and for the case of linear distribution of curvature, the rotational ductility from equation (3) is given by:

$$R_\psi = 0.28\ D_\phi \quad (5)$$

In Figure 11, the rotational ductility from the parametric studies is shown versus the curvature ductility. The rotational ductilities estimated from (4) and (5) are also indicated. Whereas the values given by equation (4) are unconservative, equation (5) represents a lower bound for the actual rotational ductility, except for very few cases where the confinement factor is close to unity. However, such low confinement factors should not be allowed for design since the concrete may deteriorate before yielding of steel.

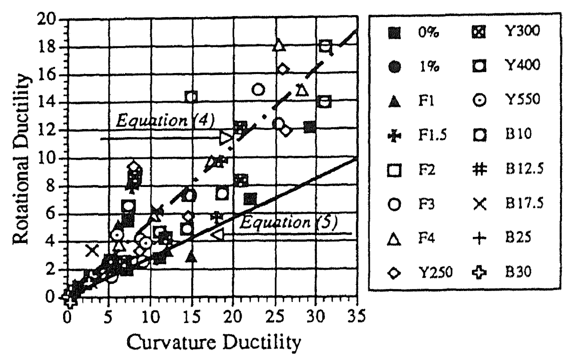

Figure 11. Rotational versus displacement ductility from the results of the parametric study

The final stage of relating the curvature ductility to the section properties is a more arduous task. As observed in the experimental and analytical investigations, many parameters can influence the attained curvature ductility. In order to simplify the design procedure, simple relationships or design charts can be derived between the curvature ductility and the section properties. This may be used to determine the required section details or to assess the existing configuration. For example, the combination of flange slenderness and spacing of local buckling inhibitors required to satisfy a certain curvature ductility demand for a given yield stress and confinement factor may be defined. Also, since the change of the spacing of the local buckling inhibitors in the IC members will have virtually no effect on neither the stiffness nor the yield capacity, it might be possible to choose a spacing that satisfies the curvature ductility demand while retaining the originally assumed flange slenderness.

5 CONCLUDING REMARKS

An analytical model for the analysis of composite frames was developed and compared with test results on a novel configuration of partially encased composite beam-columns. The new type of member was shown to provide substantial enhancement in ductility and energy dissipation capacity. A procedure for the design of the composite members according to modern seismic codes was outlined based on the results of the experimental and analytical investigations.

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