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COMPARISON OF THE SEISMIC DESIGN AND RESPONSE OF STEEL AND COMPOSITE FRAMES

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ABSTRACT

Despite large differences in many aspects of their behaviour, the seismic design of composite frames is normally performed using the same rules and techniques that are applied to the design of steel frames. These differences are most emphasised in the case of moment-resisting frames where frame behaviour is governed by the flexural response of beam and column members. To identify where the most important of these differences lie, nine moment-resisting frames have been designed according to the rules of the structural eurocodes. Three of these frames possess steel beams and columns, another three possess composite columns and beams while the remaining three frames possess steel columns and composite beams. The response of each frame to a variety of earthquake ground motions is obtained through nonlinear time-history analysis. By predefining a number of local and global failure criteria, the performance of each frame is compared and ground motion intensities at yield and collapse are obtained, from which actual response modification (or behaviour) factors are evaluated. These are compared with the provisions of Eurocode 8. The controlling response criteria, both in the design process and in evaluating the seismic performance of the frames, are also identified.

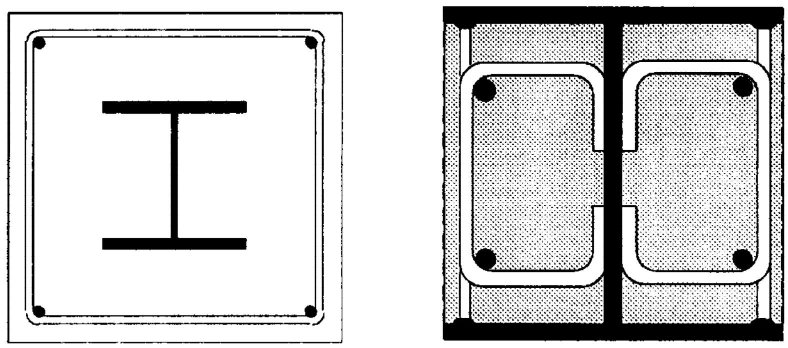
KEYWORDS

Composite construction, steel frames, seismic design, inelastic response, time-history analysis

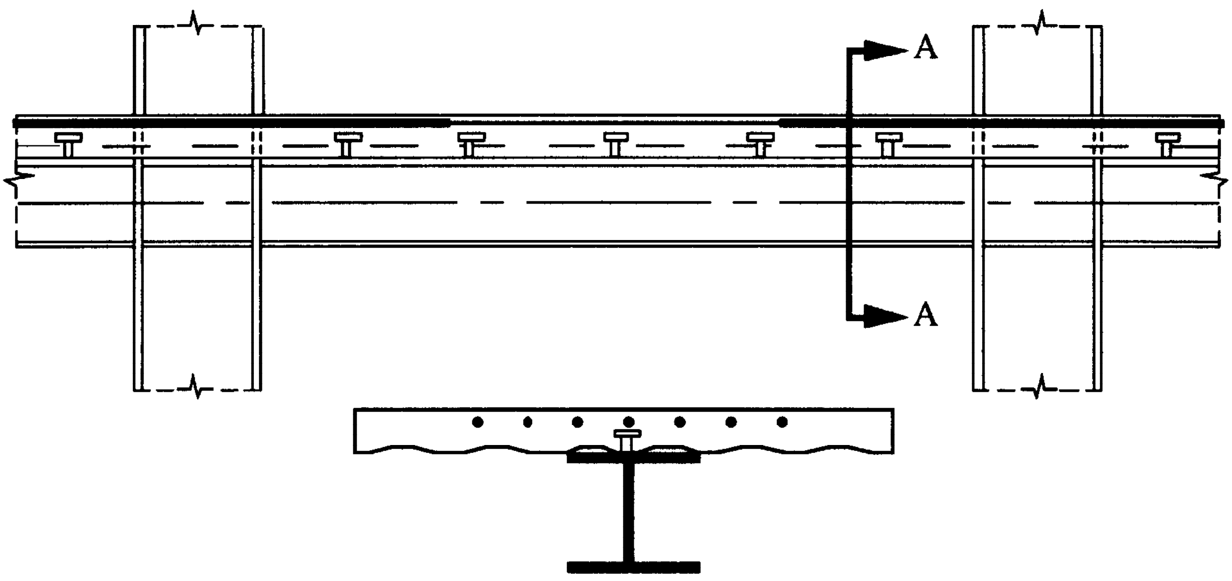
INTRODUCTION

The use of composite construction techniques whereby a concrete component is used to supplement steel members offers significant benefits in terms of improved insulation properties, higher construction speeds and lower material cost requirements. This has led to the widespread application of composite columns and beams in modern building frames where resistance requirements under vertical gravity loads dominate. Where significant lateral loads exist, such as when large wind or seismic design loads apply, the current level of knowledge and confidence in the applicability of composite beams has lead to a much lower incidence of application (Griffis, 1992). In contrast, there has been significant experience of the application of composite columns in seismic resistant designs, especially in Japan where their use has been long established (Wakabayashai, 1987). Elsewhere, the application of particular configurations of both encased and partially-encased columns to the design practices of North America and Europe has been the subject of much study (Elnashai *et al.*, 1991; Plumier and Schleich, 1993).

These studies have confirmed that composite columns of the type shown in Figure 1(a) possess sufficient rotational ductility to perform as dissipative members in moment-resisting frames. Indeed, the presence of the concrete component allows both fully- and partially-encased columns to achieve a higher level of rotation capacity, for any given section slenderness, than would be possible with their plain steel equivalents. While the application of composite columns has therefore been shown to represent an attractive option in the design of earthquake resistant frames, considerably less information is available on the response characteristics of composite beams. In particular, there is a lack of experimental verification of the capabilities of these members when exposed to large amplitude cyclic oscillations. The benefits of beam members of the type illustrated in Figure 1(b) stem from the utilisation of the reinforced concrete floor slab within the flexural resistance of the frame system, in addition to its conventional functions of load transfer, insulation and service. This is achieved through the use of the shear connectors shown in Figure 1(a) which combine the slab and its supporting steel element to form a single composite member. Additional benefit may be derived during the construction phase, where the steel beam may be used to support the formwork required for the floor slab. This formwork, which often takes the form of profiled steel sheeting, may be sacrificial, or alternatively be retained as part of the resisting element. The complexity of this arrangement however, while being of particular benefit in simply-supported beams, presents a number of difficulties in confirming the suitability of such members when a reliable response in which both large excursions into the inelastic range and successive reversals of loading are to be sustained.



(a) fully and partially-encased composite column sections



A-A

(b) composite beam member

Figure 1. Composite members for moment-resisting frames

Two additional factors are encountered in the design of these members. The first of these is the vastly different response characteristics in positive (sagging) and negative (hogging) bending. The second, and more important, factor however is the difficulty in utilising the full bending resistance of the members under lateral loading conditions. Specifically, the width of floor slab contributing to this resistance will vary depending upon the relative proportions of lateral and vertical loading, while the ability of individual members to transfer moment to their supporting columns will depend on both the direction of loading and the detailing of longitudinal and transverse slab reinforcement (Ansourian, 1976).

These issues are addressed in more detail in later sections of this paper, in which a number of case studies in the design and response analysis of a number of moment-resisting building frames are described. These frames, which employ various combinations of bare steel and composite beams and columns, are designed according to the requirements of the structural Eurocodes. Their response is then determined using both static and nonlinear time-history analysis under a range of ground motion records and the comparative performance of the frames is assessed in terms of a number of performance criteria defined in terms of both global and local failure. This assessment allows the most critical aspect of the response of each frame to be determined and, where this relates to the type of frame members employed, differences in the response of steel and composite frames to be identified. The ground accelerations necessary to cause such failures to occur may be further used to identify the behaviour factors (Eurocode 8, 1994) displayed by each frame under a earthquake record, facilitating further comparison of frame types.

CASE STUDIES

In total, nine frames were investigated as described above. The three different structural configurations shown in Figure 2 were each designed with three combinations of steel and composite beams and columns, as described in Table 1. Thus, for example, Frame CS-35 is a three storey, five bay frame with composite beams and steel columns. Where composite columns are referred to, the use of partially-encased columns of the type shown in Figure 1 is implied.

Table 1. Frame layout and member types

Frame	No. Storeys	No. Bays	Beam Type	Column Type
SS-22	2	2	Steel	Steel
CS-22	2	2	Composite	Steel
CC-22	2	2	Composite	Composite
SS-35	3	5	Steel	Steel
CS-35	3	5	Composite	Steel
CC-35	3	5	Composite	Composite
SS-63	6	3	Steel	Steel
CS-63	6	3	Composite	Steel
CC-63	6	3	Composite	Composite

While the above range of frames allows possible variations in the principal design features of the type of beam and column members employed to be considered, possible variation in a number of other aspects of the structure were not included. These included details of the beam to column connections (assumed to be of the full-strength, fully rigid moment-resisting type capable of transferring complete member bending capacities between connected beam and column members and details of the shear connection in composite beams (also assumed to be full strength), while the possible use of profiled sheeting in construction and in contributing to flexural resistance was neglected. In addition, a number of design features such are material strengths and imposed loading were kept constant for all frame designs. These design invariants included characteristic material strengths of 30 N/mm^2 for concrete, 400 N/mm^2 for reinforcing steel and 275 N/mm^2 for structural steel. An inter-frame distance of 4m was assumed in each structure, as were floor slab depths of 140mm and roof slab depths of 120mm. Characteristic imposed loads of 3.5 kN/m^2 were applied at each floor level except at the roof were a smaller value of 1.5 kN/m^2 was applied. A design acceleration of 0.25g was also assumed in each case and the use of Class 1 steel sections throughout each frame allowed a uniform behaviour factor of 6 to be applied.

Once designed, the response of each frame was determined using the plane frame nonlinear dynamic analysis program ADAPTIC. The formulation of the cross-sections of each frame element in the ADAPTIC model employed a number of monitoring areas or points which allowed the spread of plasticity and the reduction in stiffness with increasing rotation to be captured. Figure 2 illustrates these cross-section models for composite beams, partially encased columns and bare steel members.

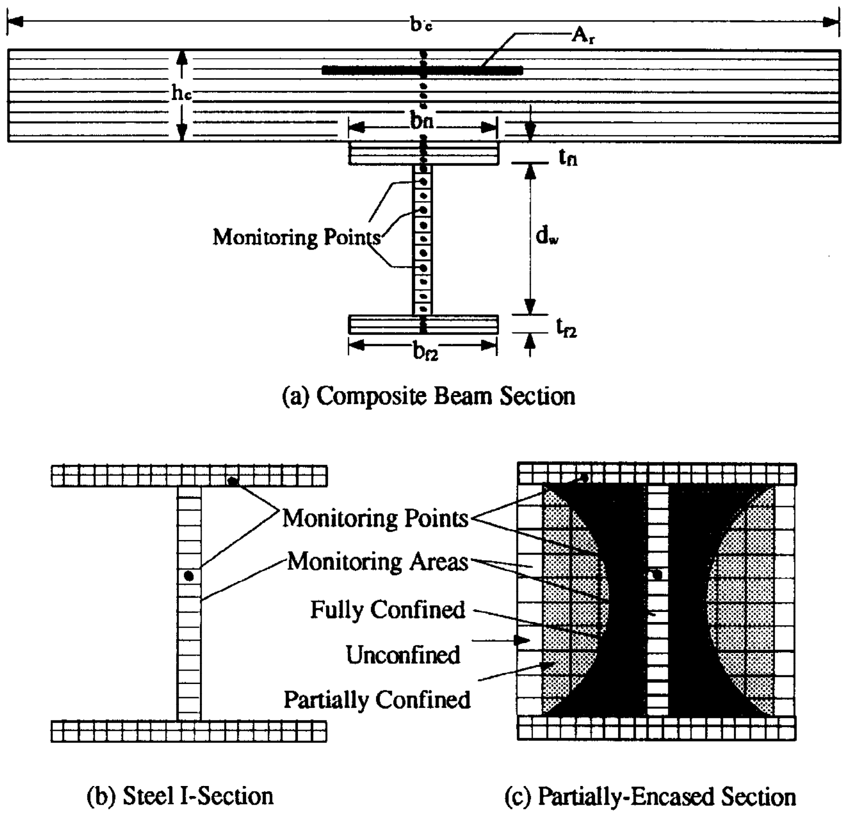


Figure 2: Finite-element cross-sections of steel and composite members

Table 2. Ground motions employed in the analysis of steel and composite frames

Earthquake	Recording Station and Component	Peak Ground Acceleration (g)	a/v ratio[1] (g/ms^{-1})	Spectrum Intensity[2] (m)	Acceleration Scaling Factor
Friuli 1976	Tolmezzo EW	0.159	1.99	143.7	1.89
Gazli 1976	Karakyr Pt. EW	0.724	1.20	71.3	0.84
L.Prieta 1989	Emeryville S80W	0.213	0.99	116.3	1.74
El Centro 1940	El Centro S00E	0.344	0.94	100.5	1.25
Spitak 1988	Gukasyan Trans.	0.182	0.77	95.6	2.47
L.Prieta 1989	Emeryville N10W	0.250	0.58	192.3	0.90

Notes
1: Ratio of peak ground accelerations and velocities in corrected records
2: Eurocode 8 design spectrum intensity (a_g = 0.25g) = 172.4m

The response of each frame to six different earthquake records was determined using time-history analysis with the Hilber-Hughes-Taylor time-stepping scheme (Hilber *et al.*, 1977). Details of the accelerograms employed in the analysis are described in Table 2. These were selected on the basis of their peak ground acceleration to velocity ratios; the variation in a/v ratio serving to ensure sufficient dissimilarity in frequency characteristic and member ductility demands (Tso *et al.*, 1992). Each of these accelerograms was scaled to possess a spectrum intensity equal to that of the Eurocode 8 design acceleration with a ground acceleration of 0.25g. Further scaling relative to this value was employed to evaluate the effects of more severe seismic loading (Broderick and Elnashai, 1996).

DESIGN FEATURES

In the design of both steel and composite frames, the restriction on interstorey drift imposed by Part 2 of Eurocode 8 had a major influence on the selection of suitable beam and column members. This restriction, although intended to control response at the servicability limit state, effectively limits allowable drift due to the elastic design forces to less than 1.5% of storey height. In this respect, it is noteworthy that while the ability of composite beams to effectively resist lateral loads requires further confirmation in the inelastic range, their response at the much lower amplitudes implied at the servicability state is more assured. For taller frames, such as the six-storey frames considered here, the maximum interstorey displacement occurs in the second storey of the structure. As construction considerations imply that column sections will normally remain uniform over the height of two or three storeys, the effect of imposing a control on interstorey drift in the lower storeys will imply a drop in storey shear strength above the second or third floor. In this situation the requirements of vertical regularity become significant. While the provisions of Eurocode 8 do not, however, provide explicit guidelines on how to determine whether this has been achieved, this piecewise reduction in column size has implications on the locations in which plastic hinges form during high amplitude response.

In moment-resisting frames, the lateral stiffness required to limit design interstorey drift is provided solely through the flexural resistance of the frame members. Although the material benefits of providing greater stiffness in beam rather than column members are a function of the relative values of bay width and storey height, the need to provide sufficient overstrength to prevent column hinging tends to dominate member selection instead. For composite beams, some interpretation of the code provisions are required when determining how great this overstrength should be, not least on account of the different plastic bending resistances of these members in positive and negative bending. Despite the resistance in negative bending being typically 20-30% lower than that in positive bending, conservative application of the code provisions require column overstrengths at internal nodes to be based upon the larger value only.

For composite beams, the selection of member dimensions is commonly governed by bending resistance in negative moment regions due to the persistent design (or non-seismic) load case. For the seismic design case, the application of moment redistribution is not allowed in the design analysis, resulting in the need to employ larger quantities of beam reinforcement. In either case, the bending capacity of these members, in positive moment regions, will be as much as twice the bending moment applied under the seismic design load case. This implies a reserve of resistance which is readily exploitable when the response of the frame enters its inelastic range. For steel beams in which lateral restraint from the floor slab is not assured, resistance to lateral torsional buckling may control member design. This was assumed to be the case for the two and three storey structures investigated here, but not in the six storey case.

ANALYTICAL RESULTS

Studies including the number of time-history analyses considered here gives rise to an excessive amount of response data. To isolate those response features of interest, it is necessary to predefine a number of performance indices or criteria against which the identified behaviour can be compared. In this instance, a number of response or failure criteria are defined at both local and global levels which are then used to assess the margin of safety available at any level of seismic load. As the applied ground acceleration is progressively increased in successive analyses, this margin of safety is reduced, until at the collapse ground acceleration, the critical failure criterion is met. Thus, design and failure loads may be compared and critical performance characteristics identified.

Response Criteria

Response criteria, which represent conditions which imply an unsatisfactory performance during response to a specific accelerogram, are defined at both global and local levels. The global response is defined in terms of the behaviour of individual storeys of a structure, while the local response represents the behaviour of individual frame members. Using dedicated post-processor software, each criterion is assessed after every time-step of the analysis to determine whether a structural failure has occurred.

Storey Response. Four criteria are established at the individual storey level: (i) a limit on interstorey drift, which is established as 3% of the height of the particular storey concerned; (ii) a limit on the storey stability index defined in Eurocode 8, which places a limit on storey drift in terms of applied total gravity load and seismic shear at the storey; (iii) the formation of a storey collapse mechanism implied by the concurrent existence of plastic hinges at the upper and lower ends of each column in the storey and (iv) a limit of 10% on the amount by which the lateral resistance provided by a storey decreases - due to second order effects and material inelasticity - below its peak value, during any single oscillation of the frame.

Member Response. In column members, a limit is placed on the allowable rotation ductility demand. This is expressed in terms of the rotation ductility capacity, determined through the methods outlined by Kato (1989) for steel members and through a modified version of this method for the partially encased column members (Broderick, 1994). For beam members, the calculation of member rotation ductilities is not as straight forward, hence this check is augmented through a limit to the strain level in the concrete flange for composite beams and in the steel compression flange for both steel and composite members.

Failure Mode and Behaviour Factors

Table 3 presents the identified behaviour factors with associated critical failure response criterion and accelerogram for each frame. These behaviour factors are calculated from the ratio between the accelerogram scaling factor at failure and that applied to obtain the design load level in the manner suggested by Kappos (1991). In all but Frame SS-63, the identified behaviour factors exceed the value of six used in design. The premature failure of Frame SS-63 occurred due to an excessive interstorey drift in the fourth storey of the structure. In all other cases, the failure acceleration exceeded that assumed in design, representing a margin of safety. However, much of this margin may be associated with the large increase in lateral resistance provided by each frame after an initial plastic hinge had formed. This behaviour is illustrated in Figure 3 which presents the force-deformation response displayed by Frame SS-63 under monotonically increasing roof displacement. In general for moment-resisting frames of the type considered here, the influence of drift control requirements on member selection and the redistribution of internal bending forces throughout the structure once initial yield has occurred, leads to lateral resistance capacities far in excess of the design load. This behaviour may be exploited in frame design through the use of higher behaviour factors (up to a limit of 8, Eurocode 8 (1994). In frames with composite beams, this difference between the ultimate lateral resistance and that at which initial yield occurs is greater, due to the lower bending resistance capacities in negative moment regions, than in those with steel beams.

Table 3. Critical response criteria and ground motion record.

Frame	SS-22	CS-22	CC-22	SS-35	CS-35	CC-35	SS-63	CS-63	CC-63
q'	7.68	7.38	9.60	7.92	9.72	7.92	5.13	10.62	10.38
Criterion	Mech'm	Drift	Drift	Mech'm	Drift	Drift	Drift	Drift	Drift
Accel'gm	Spitak	El Centro	Spitak	Spitak	Spitak	Spitak	El Centro	LP (EW)	LP (NS)

Although drift and mechanism formation dominate the list of failure criteria shown in Table 3, member rotation ductility capacity was also a significant factor, constituting the critical criterion in the response of some frames to earthquake motions other than those shown in Table 3. This was most notable in the case of those frames with composite beams, where a higher compression flange strains are experienced. However, the level of rotation capacity ensured by restricting the choice of steel elements to Class 1 sections only ensured that ductility demand experienced was never sufficient to cause local buckling to occur at ground accelerations below the design level. In contrast, column rotation capacity was seldom the critical response criterion, irrespective of the type of column employed. The formation of a storey mechanism was seen to occur only in frames with steel beams, being the controlling criterion in two of the cases described in Table 3. In the case of frames with composite beams, the code capacity design procedures ensured that flexural plastic hinges did not occur in any external column connected to a beam end which was in negative bending, thereby preventing the formation of a storey mechanism.

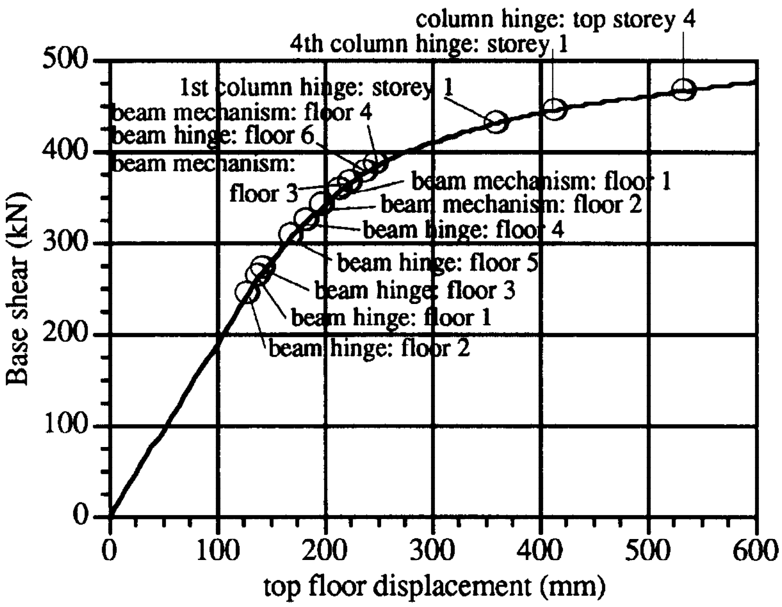


Figure 3. Global force displacement response and hinge formation pattern for Frame SS-63.

COMPARATIVE OBSERVATIONS

In this section, some observations are made on the comparative response of the frame types considered above. These are based upon the response features displayed during transient analyses, on the member characteristics which caused these features and on the design procedures upon which the member dimensions were determined. It is hoped that these illustrate the areas where potential benefits can be drawn from the application of composite construction to seismic design and also those areas where additional study is required to determine the suitability of such application.

Of primary interest is the influence the code-imposed limits on interstorey drift due to the design loads, which tends to dominate member selection and lead to a reserve of frame resistance which is drawn up during seismic response as represented by transient analyses. This influence decreases the significance of the level of design force reduction achieved through the application of the behaviour factor to the elastic design forces.

While the significance of drift control affects both composite and steel frames equally, collapse mechanism formation, which in this study represented the second most relevant response characteristic, was seen to be relevant only for frames with steel beams, with the irregular strength distribution arising in composite beams serving to protect some columns from excessive bending moments. The likelihood of mechanism formation in taller steel frames is enhanced by the tendency to use the same structural sections over two or three storeys, leading to a sudden reduction in the difference between applied storey shear and resistance capacity. This was observed in the current study despite the fact that the vertical regularity requirements of Eurocode 8 where adhered to.

To realise the full potential of composite beams to resist seismic loads, levels of plastic resistance moments similar to those achieved under vertical loads must be assured. Whereas in negative bending this should be possible through a sensible distribution of longitudinal reinforcement, member plastic resistance in positive bending will depend upon the area of floor slab which is place in compression. While under transverse beam loads, this is area is assured to be sufficient due to the large width of slab over which compression stresses are experienced, no similar assurance is given in the case of applied end moments. However, it should be considered that what is in fact required is a compression force within the floor slab which is sufficiently large to balance the maximum tensile force which can be developed within the structural steel section. Therefore while under gravity loads, a wide but narrow compression stress block is established, in seismic loading an equivalent narrow but deep area may be set up. In these circumstances, the longitudinal reinforcement in compression will also make a significant contribution to bending resistance.

This issue becomes most important at connections of beams and columns. Here, the effective width of the beam is reduced to that of the column face against which it bears. In this case, the compression area which can be developed is obviously limited, with large rotations leading to high local compression strains and crushing on the top face of the floor slab. However, the benefits of composite beams are most applicable when relatively large spans are to be employed. In this instance, the influence of gravity loading has a significant effect, even with high design ground accelerations. As a consequence, while high negative bending moments may exist close to beam column nodes, positive moments in these regions tend to be much lower. This feature of response is enhanced through the fact that as moment redistribution is not employed in seismic design procedures, the negative moment due to gravity loading is increased, as is the level of lateral load required to produce a net positive moment.

The use of composite columns doe not seem to provide any additional benefits in terms of usable rotation ductility capacity over that provided by Class 1 steel sections. The advantages of using composite columns are limited to those pertaining in other areas of structural design (such are fire insulation and ease of construction) and the fact that the required levels of rotation capacity may be achieved with more slender steel cross sections. While offering benefits in terms of economy, this last observation depends upon correct member detailing.

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