PERFORMANCE ASSESSMENT OF COMPOSITE MOMENT-RESISTING FRAMES

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ABSTRACT :

This paper examines the seismic performance of composite steel-concrete moment-resisting frames. Several sensitivity and parametric investigations are undertaken using an advanced analysis program that accounts for material and geometric nonlinearities. Particular emphasis is given to composite frames designed according to the provisions of the European seismic code, Eurocode 8. The validity of employing simplified nonlinear static loading approaches is evaluated by comparison against the results of incremental dynamic time-history analysis. Natural earthquake acceleration records, which are specifically selected and adjusted for compatibility with the adopted design spectrum, are utilised for this purpose. In terms of frame configuration, it is shown that the span of the composite beam and the number of stories can have significant implications on the actual inelastic response characteristics of the structure. The studies presented in this paper highlight important behavioural observations and trends, some of which point towards the need for further consideration and refinement of current design procedures.

KEYWORDS: Seismic Response; Composite Steel-Concrete Frames; Eurocode 8

1. INTRODUCTION

The use of moment frames incorporating composite steel-concrete floor systems offers several behavioural and practical advantages over bare steel and other alternatives. The increase of stiffness and capacity due to composite action enables the use of larger beam spans under the same loading conditions. Accordingly, the demand for larger and more flexible usable space, coupled with the need for faster optimised construction processes, has led to an increased utilisation of composite frames in recent years.

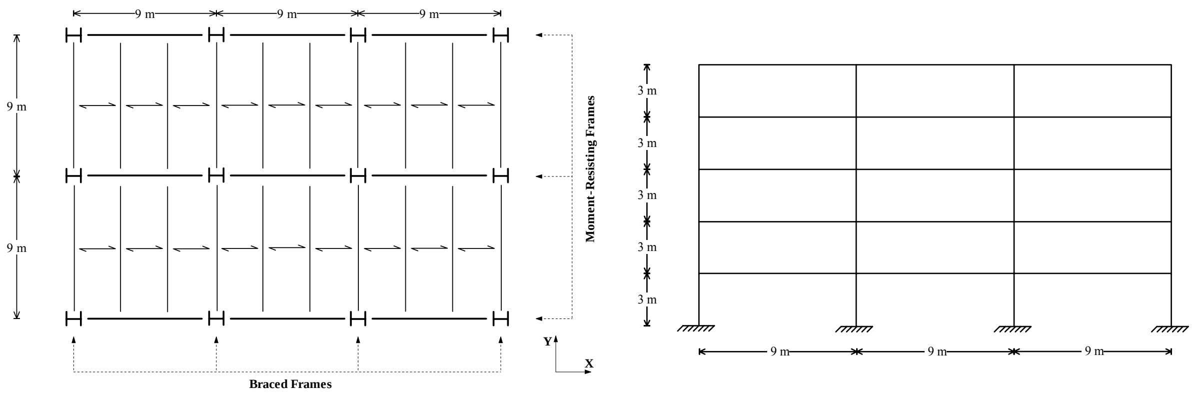
In terms of seismic performance, composite frames exhibit favourable behaviour due to the enhanced response characteristics including ductility properties. However, the number of detailed studies carried out on the seismic response of composite frames, in comparison with bare steel or reinforced concrete counter-parts, has been relatively limited. This is attributed, on the one hand, to the assumption that the behaviour could be largely inferred from studies on steel or reinforced concrete frames and, on the other hand, to the complexity of some of the analysis and design issues involved, as reported in previous investigations (e.g. Leon, 1998; Spacone and El-Tawil, 2004).

Although previous studies have addressed and resolved important behavioural aspects, there is a need for assessing the key parameters influencing the seismic performance of composite moment frames with typical geometric and loading configurations. In particular, with the recent introduction of Eurocode 8 (CEN, 2004), it is important to examine the performance of composite frames designed according to the new European seismic code.

In this paper, the seismic performance of composite moment frames is examined. The reference structure as well as the design procedures and assumptions adopted are firstly discussed. This is followed by a brief presentation of the numerical models utilised in this investigation. An assessment of the nonlinear static response of a typical composite moment frame is then presented and the results are validated by comparison with dynamic time-history analysis, using earthquake records which are carefully selected and adjusted for compatibility with the design spectrum. A number of numerical studies are then carried out in order to assess the influence of key parameters on the response of composite frames. The main behavioural observations and trends are discussed and their implications on seismic performance and design are highlighted.

2. STRUCTURAL CONFIGURATION AND DESIGN PROCEDURES

A reference structure is selected such that it can be used as a basis for parametric variations. As shown in Figure 1, the structure consists of a five-storey composite steel-concrete office building. In plan, columns are spaced at 9 m in both directions. With due account of column inertia, lateral resistance is provided by composite moment frames in one direction whilst the other plan direction is assumed to have a braced system. The floors consist of 120 mm thick composite slabs of the re-entrant (dovetail) profile with 1 mm thick steel deck supported by secondary beams spaced at 3 m.



(a)
Figure 1 (a) Plan configuration and (b) Elevation of the reference structure

The internal composite steel/concrete moment-resisting frame, depicted in Figure 1b, was adopted in this study as the reference frame (CREF). The frame was designed according to the provisions of Eurocodes 3, 4 and 8. To satisfy the gravity design situation, IPE500, HEB300 and HEB450 cross-sections were required for the beams, external columns and internal columns, respectively. The frame was then checked for seismic conditions according to EC8. The design peak ground acceleration (PGA) was assumed as 0.30g, whilst a rock soil (Ground Type A) and Spectrum Type 1 were considered. The fundamental period of vibration (T_1) based on the simplified expression given by EC8 (T_1=0.085*$H^{3/4}$, where H is the overall height in metres) was 0.65 s. However, T_1 was found to be nearly 1.0s from modal analysis performed in a simplified structural model assuming centreline dimensions for the members. This value was deemed more accurate and hence was used in the evaluation of the design base shear. The behaviour factor was taken as 6.5 on the basis that use is made of Class 1 cross-sections, which satisfy DCH (Ductility Class High) requirements. The total mass considered for the seismic design situation was about 1440 tons based on a combination of the unfactored dead load and 30% of the imposed load. The resulting design base shear was about 700 kN which was distributed linearly over the height.

In addition to satisfaction of the seismic strength demands in members, other seismic design checks include compliance with stability and drift criteria as well as capacity design considerations. Second-order stability effects are considered through the sensitivity coefficient (θ). Capacity design checks are performed to ensure ductile response of beams and the achievement of the general concept of weak-beam/strong-column behaviour. For steel and composite moment frames, a specific application rule is stipulated whereby the design moments (M_{Ed}) for the columns can be obtained from:

$$M_{Ed} = M_{Ed,G} + 1.1\gamma_{ov}\Omega\, M_{Ed,E} \tag{1}$$

where $M_{Ed,G}$ and $M_{Ed,E}$ are the moments due to the gravity loads and lateral seismic forces, respectively; γ_{ov} is a material overstrength factor typically assumed as 1.25; Ω is a beam overstrength factor determined as a minimum of $\Omega_i = M_{pl,Rd,i}/M_{Ed,i}$ of beams, where $M_{Ed,i}$ is the design moment in beam 'i' and $M_{pl,Rd,i}$ is the corresponding plastic moment.

For the reference composite frame under consideration (CREF), the member sizes selected on the basis of the gravity combination were found to satisfy all the seismic design checks as stipulated in EC8. However, the external columns had to be increased to HEB 340 in order to limit the stability coefficient (θ) to a value of 0.2 to avoid nonlinear analysis at the design stage. Six reinforcement bars of 20 mm diameter were provided in the slab at the beam-to-column joints, as recommended by the detailing rules given in Annex C of EC8. The steel part of the beam-to-column connections were considered to be fully welded in order to achieve an idealised rigid response. Additionally, to comply with EC8 provisions, the column panel zones had to be strengthened through the addition of sets of two doubler plates with 21 mm and 29 mm thickness to external and internal columns respectively, such that these regions can be considered as non-dissipative. Additional details and discussions regarding the design of the frame can be found elsewhere (Castro, 2006; Elghazouli *et al.*, 2008).

3. MODELLING AND ANALYSIS CONSIDERATIONS

3.1. Numerical Models

The nonlinear finite element program ADAPTIC (Izzuddin, 1991) which accounts for material and geometric nonlinearities is used for the analysis of the composite frames. Beams and columns are represented with cubic elasto-plastic elements which account for the spread of plasticity across the section, using a fibre approach, as well as along the length using Gaussian sections in conjunction with a cubic shape function. In particular, the composite beams are modelled by two parallel lines of cubic elements considered at the centroids of the steel and concrete constituent parts, as illustrated in Figure 2a. As shown in the same figure, composite action is developed by introducing link elements with rigid properties connecting the steel and concrete line elements. For simplicity, the effective slab width was assumed as 1.25 m based on an estimate of the contra-flexure location and the provisions of EC4 for the sagging moment region. Although the provisions vary in EC4 and EC8 for sagging/hogging regions and for design/analysis, a sensitivity study indicated that these differences do not have a notable influence on the overall frame response (Castro, 2006). Further discussion dealing with the issue of effective width in composite beams can be found elsewhere (Castro *el al.*, 2007).

Special attention is given to the modelling of the panel zone regions, for which a modified frame approach (Castro *el al.*, 2005) is adopted by assembling link and joint elements. Figure 2b shows the model for a typical external connection. Distinction is made between the main panel zone and the top part which is in contact with the slab. The contact behaviour between the composite slab and the column is also taken into account through the introduction of joint elements. Complete details of the numerical models can be found elsewhere (Castro, 2006; Elghazouli *et al.*, 2008).

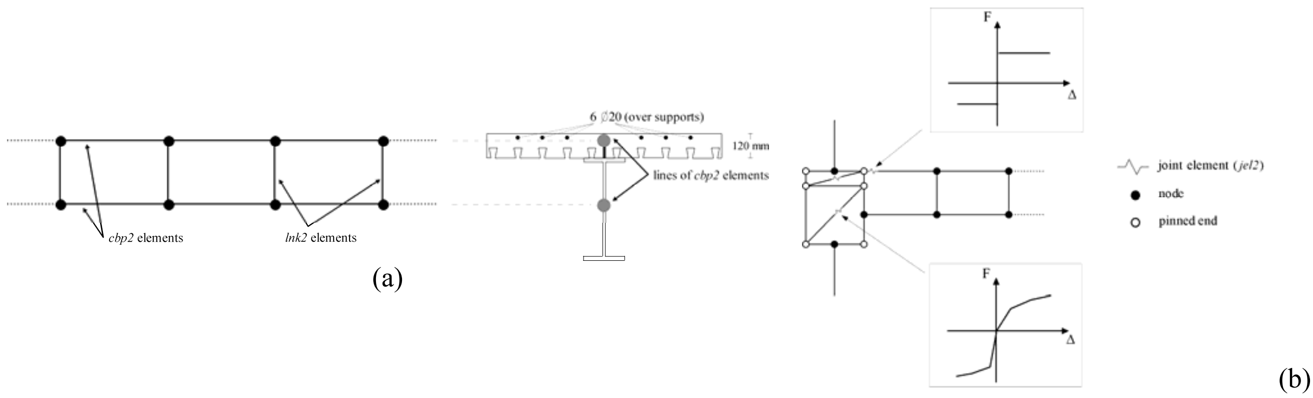


Figure 2 Modelling approach adopted for (a) Composite beams and (b) Composite joints

For steel materials, a bilinear elasto-plastic cyclic model with strain-hardening is adopted for both structural and reinforcing steel. On the other hand, concrete nonlinear behaviour is accounted for through a uniaxial cyclic constitutive model featuring both compressive and tensile softening. In all the analyses performed in this study, the steel elastic modulus (E) and strain hardening coefficient (μ) are assumed as 210×103 N/mm^2 and 0.5%. The yield strength values (f_y) for structural steel and reinforcement bars are assumed as 275 and 500 N/mm^2, respectively. With regard to concrete, a value of 30 N/mm^2 is considered for the compressive strength whilst tensile strength is ignored in all the analyses.

3.2. Input for Time-History Analysis

The selection of records is carried out herein by combining two different criteria. The records are selected such that they correspond to the shape of the code-spectrum employed (Type 1 of EC8) as well satisfy seismological criteria inferred by this choice. Seven records, with the lowest root mean square deviation (D_{rms}) from the target code-spectrum, were sought for this study from the strong-motion database available at Imperial College. The normalised response spectra, for 5% damping, for all seven records as well as the target design spectrum are shown in Figure 3a. The selection was then followed by another adjustment process for matching the records to the target design spectrum, based on the introduction of wavelets to the acceleration time series. The correction is performed in two steps. Firstly, each record is modified in order to match the target spectrum within the period range between 0 and 1 s. In the second step, wavelets are introduced in the time series in order to match the target spectrum for the whole period range, i.e. between 0 and 4 s. The response spectra for the final adjusted records as well as the target spectrum are shown in Figure 3. A detailed explanation of the record selection procedure is provided elsewhere (Castro, 2006).

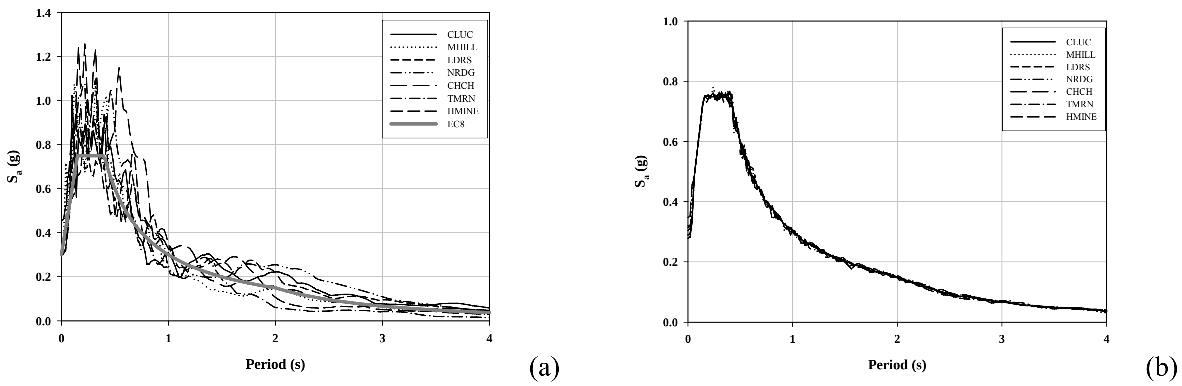


Figure 3 Response spectra (normalised to PGA) of the (a) Initial and (b) Wavelet-adjusted records

4. LATERAL RESPONSE OF THE REFERENCE STRUCTURE

The main purpose of this section is to examine the response of the reference frame using ‘conventional’ forms of pushover analysis, commonly referred to as the ‘triangular’ and ‘uniform’ distributions. The adequacy of the pushover idealisations in representing the inelastic response of the frame is then assessed by comparison against the results of incremental dynamic analysis.

4.1. Nonlinear Static Response

The nonlinear-static response of the reference frame is presented in Figure 4a. The analysis is conducted by increasing the displacement at the top of the frame incrementally up to a global lateral drift of about 3% of the overall height. The results are shown for both linear (triangular) and uniform patterns of lateral load. Due to the regularity and simplicity of the structure, the triangular pattern represents closely a first mode-dominated response. On the other hand, the uniform pattern is expected to be important if higher modes contribute notably to the response or when significant inelastic concentrations occur.

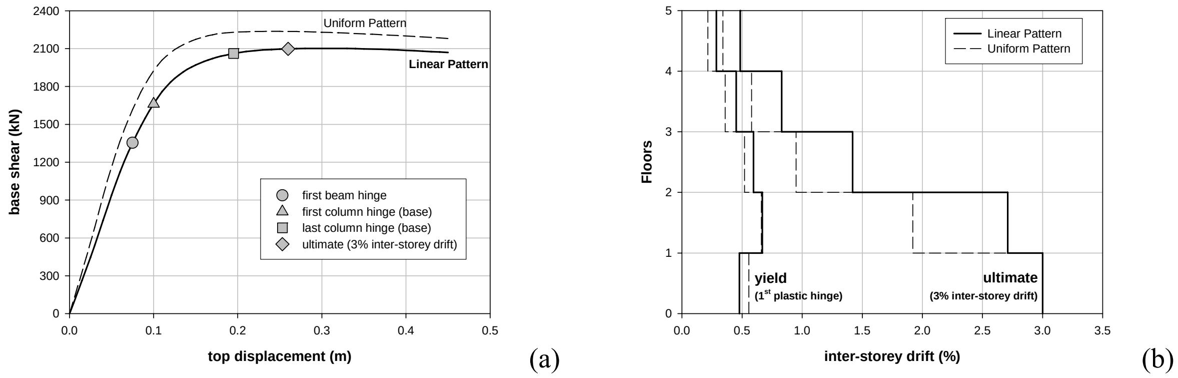


Figure 4 Reference structure: (a) Pushover curves; (b) Inter-storey drifts

As shown in Figure 4a, the uniform loading pattern results in higher stiffness and capacity in comparison with the triangular distribution, as expected. The inter-storey drifts in the frame at yield (formation of first plastic hinge) as well as at ultimate (achievement of 3% inter-storey drift) are depicted in Figure 4b. The inter-storey drift distribution is almost uniform at the yield stage since the behaviour is largely elastic. At ultimate, however, lower stories exhibit significant inter-storey drifts compared to upper levels. The inelastic concentration is more pronounced for the uniform load pattern which forces a more distinct soft-storey behaviour at the first storey. The inelastic distributions shown in Figure 4b are closely related to Figure 5 which illustrates the sequence and location of plastic hinges occurring in the beam and column members up to the attainment of the defined ultimate limit.

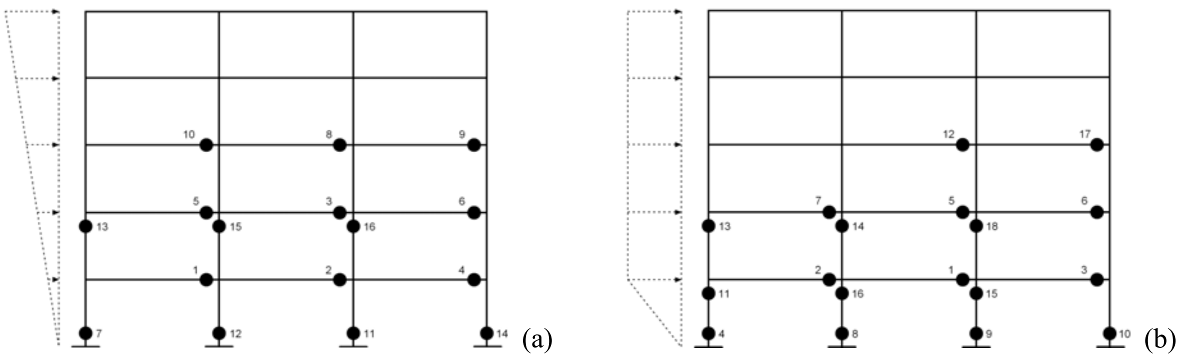


Figure 5 Reference structure: Hinge pattern for (a) Linear and (b) Uniform load patterns

Apart from the difference in response for the linear and uniform patterns, a number of important observations are noteworthy with reference to Figure 4. Firstly, the frame is shown in Figure 4a to possess an actual capacity which is considerably higher than that assumed in design. A degree of overstrength is generally expected for framed structures (Elghazouli, 2005), particularly when a relatively high behaviour factor is employed, since the member sizes are likely to be governed by the gravity loading scenario or by deformation-related limits (from inter-storey or stability checks). This overstrength effect becomes even more pronounced in the case of composite frames.

Another important observation is related to the formation of plastic hinges predominately on one side of the beam spans, as illustrated in Figure 5. This is caused by a combination of the influence of gravity moments (which can be significant in relatively large spans enabled by composite action), coupled with the difference between the plastic moment capacity of the cross-section under positive and negative moments which is characteristic of composite beams. Unless considerable levels of overstrength are provided to the columns to preclude the formation of column hinges at significant inelastic drifts, beam hinges may not form in the other side of the spans at ultimate, which is the case in the reference frame.

4.2. Dynamic Response

In order to examine the general validity of nonlinear static procedures for assessing the overall seismic response of composite moment frames in further parametric studies, the pushover results of the reference frame are compared with those from dynamic analysis. Using the seven modified records described in the previous section, incremental dynamic analysis was performed for the reference frame. Each of the seven records was applied with increasing ground motion intensity, resulting in over 100 analyses. The overall dynamic response is compared in Figure 6 to that obtained from pushover analysis, with both linear and uniform patterns. The response is presented in terms of base shear versus maximum drift at the top of the frame.

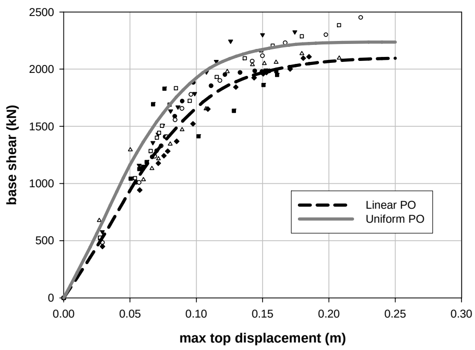


Figure 6 Time-history results *vs* pushover curves

The figure indicates relatively low dispersion between the results from the seven earthquakes, which is clearly aided by the record-selection and spectrum-matching procedures described before. More importantly, the plot reveals relatively good correlation with the pushover results, particularly for the linear pattern case. As shown in the figure, the average of the time history results is more closely related to the linear pattern for most of the response range, except perhaps at significant drift levels when inelastic deformations may tend to concentrate in lower storeys. In contrast, the uniform pattern provides more of an envelope of the global behaviour rather than a realistic representation of the mean level of dynamic response.

5. PARAMETRIC STUDIES

The design process of a composite moment frame involves several assumptions and choices that can have a direct influence on the performance. Using the reference frame described before as a basis, a number of variations to several design parameters and assumptions were considered. Due to space limitations, only a subset of the parametric study is presented in this section. The parameters investigated here are related with the geometrical configuration of the frame and include the beam span and the number of stories. The modifications carried out to the reference frame are summarised in Table 5.1, together with the resulting member sizes and governing criteria. In all cases, only one design parameter or assumption is varied within each specific frame. It should be noted that, where possible, it was decided to retain the beams sizes based on gravity design and adjust column sizes to satisfy the various design criteria. Details of the full parametric study can be found in Elghazouli *et al.* (2008).

Table 5.1 Summary of parameters considered and section sizes adopted

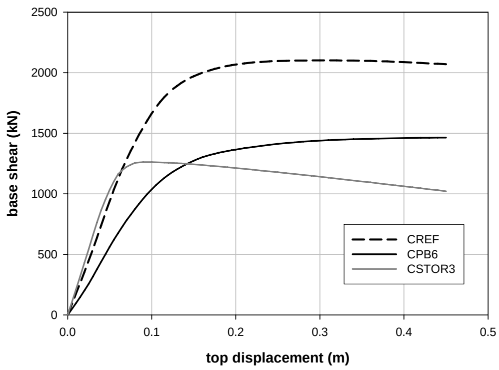
Study	Designation	Remarks	Seismic Design			
			Beams	External Columns	Internal Columns	GC§
Reference frame	CREF	—	IPE 500	HEB 340	HEB 450	c
Spacing and geometry	CPB6	6m beam span	IPE 330	HEB 450	HEB 500	c
	CSTOR3	3-storey frame	IPE 500	HEB 260	HEB 320	a

§Governing Criterion: a. seismic strength demand, c. stability coefficient

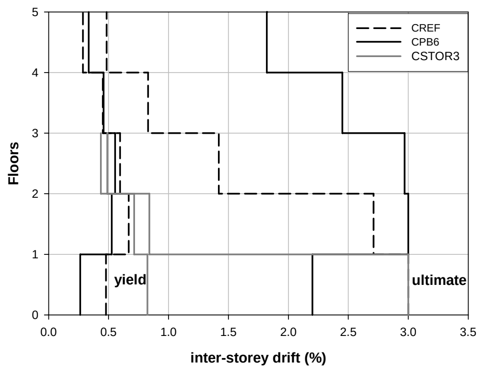
5.1. Influence of Beam Span

In order to examine the influence of the span of the beam on the seismic response of the composite frame, the beam span is modified from 9.0 m in the reference frame (CREF) to 6.0 m (CPB6). Due to the shorter length and reduced gravity loading, the member sizes required to resist the gravity situation are smaller than in CREF as listed in Table 5.1. However, the column sizes are ultimately governed by the second-order stability criterion. The adopted design procedure therefore leads to larger column sizes in CPB6 compared to CREF.

The response of CPB6 in comparison with CREF is shown in Figure 7a. The reduction in beam size leads to lower stiffness and capacity in CPB6. However, as expected, the higher column-to-beam capacity ratio results in a more favourable inelastic distribution over the height as shown in Figure 7b. This is also illustrated in Figure 8a, indicating an 'ideal' hinge formation in the beams.



(a)

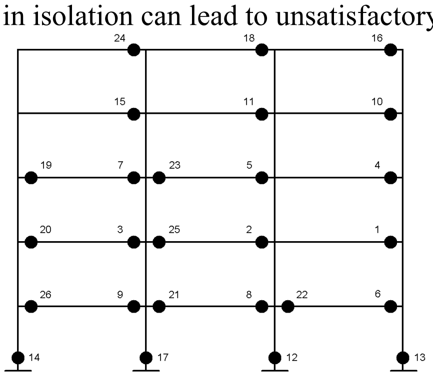


(b)

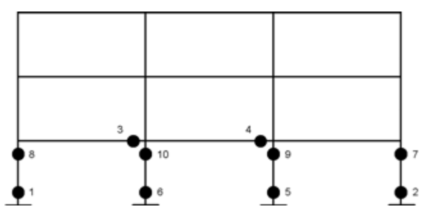
Figure 7 Influence of beam span and number of stories on lateral response: (a) Pushover curves (b) Inter-storey drifts

5.2. Influence of Number of Stories

Another variation of the reference frame (CREF) is considered whereby the number of storeys is reduced to three (CSTOR3). The relatively low overall gravity loading applied to the ground floor columns means that the stability coefficient does not govern the size of the columns and it is instead determined by seismic strength demands. In principle, the capacity design rules for columns should ensure that relatively strong columns are provided such that more dominant beam hinging occurs. However, direct implementation of the application rule of Eqn. 1 in isolation can lead to unsatisfactory performance.



(a)



(b)

Figure 8 Sequence of hinge formation: (a) CPB6 case; (b) CSTOR3 case

Figures 7a, 7b and 8b depict the pushover response, inter-storey drift and plastic hinge pattern, respectively, obtained for CSTOR3. Clearly, the column sizes are inadequate in this case to prevent the formation of an undesirable column mechanism. This is largely an implication of Eqn. 1 which in its suggested form ignores the influence of gravity loading in calculating the overstrength parameter Ω. As a result, Ω is underestimated considerably for cases in which the gravity moment in the beams ($M_{Ed,G}$) constitutes a significant proportion of the total moment (M_{Ed}). This is often the case in composite configurations as these typically incorporate relatively large beam spans. Although this issue is not necessarily related to the number of stories, the limits imposed on the stability coefficient lessen its effect on the required column sizes except in low-rise frames.

6. CONCLUSIONS

This paper assesses the inelastic seismic performance of composite steel/concrete moment-resisting frames designed according to the provisions of Eurocode 8. After discussing the design procedures and assumptions, the numerical models adopted in this investigation are described.

The results obtained for the reference composite frame confirm the validity of adopting nonlinear static approaches for this type of structure. This is further confirmed by comparison against the results of incremental dynamic analysis. For this purpose, seven natural earthquake acceleration records, which are selected and adjusted for compatibility with the design spectrum, are utilised. It is shown that the results of the dynamic time-history analysis are closely related to the nonlinear static response adopting triangular loading pattern, following the trends observed in other frame types of regular configurations. Both the pushover and time-history results confirm the high level of overstrength which is normally expected in framed structures. This overstrength becomes even more pronounced in composite frames, due in-part to the typically large spans which increase the dependence of beam sizes on gravity loading conditions.

The parametric investigation shows that several parameters and assumptions can have direct implications on the inelastic behaviour of composite frames, as assessed through the overall lateral response, inter-storey drift distribution and plastic hinge patterns. In particular, it is shown that a number of geometric parameters, related to the structural configuration, including beam span and structural height, have a significant influence on the behaviour. The results also evidence some limitations of the capacity design provisions prescribed in Eurocode 8 indicating therefore the need for further research and refinement of current design provisions.

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