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A basis for comparing progressive collapse resistance of moment frames and connections

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ABSTRACT

The progressive collapse resistance of a moment frame in the event of an inner column loss has been studied in terms of either the beam span or the beam span-to-depth ratio. This study reiterates that, strictly speaking, it is the beam span-to-depth ratio that is the underlying factor. Basically the larger the span-to-depth ratio, the better the progressive collapse resistance if the beams have been similarly optimised against plastic hinging under the design floor load. In comparing the performance of various types of steel double-span assemblies against each other, it would not always be appropriate to normalize their resistance against the same uniformly distributed load on the floors unless the beams have been similarly optimised. This article explains how the progressive collapse resistance of various types of steel double-span assemblies can be clearly compared against each other with respect to their development of the catenary mechanism, independently of the optimisation extent of the beam section against plastic hinging under the design floor load. The quasi-static resistance is normalized against the plastic hinge load, and the chord rotation is normalized against the plastic rotation. The proposed procedure further enables a rigorous comparison between different types of steel moment connections since the effects of different beam sections and different spans are filtered out under the same span-to-depth ratio. Issues such as the dynamic effects of sudden column loss, the contributory effects of floor slabs and the second-order effects of topological changes are not relevant to the present study.

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1. Introduction

In the literature of progressive collapse prevention, the Alternate Path (AP) method is widely used to examine the surviving structural capacity following a vertical load-carrying element loss [1,2]. In order to prevent the initial damage from spreading to the surrounding areas in a frame system, the remaining structure within the damaged bays should be able to bridge over the lost column. When an inner column is removed from the frame, the load-carrying capacity of the double-span assembly above the removed column plays a crucial role in the progressive collapse prevention. In the simplified framework for multi-storey buildings proposed by Izzuddin et al. [3], the double-span beam-column assembly within the damage bays is deemed as the lowest level of the structure among the various levels of sub-structure idealisation, and its response is used for composing the higher level sub-structures. The performance of the double-span assembly has been found through experimental study [4–9] to primarily depend on the beam section as well as the beam-to-column connection configuration in developing the flexural and the catenary mechanisms to carry the load previously supported by the damaged (removed) column.

Since the catenary mechanism involves the development of axial forces in the beams as the double-span assembly deflects downwards, the beam span l_0 has been identified to be another important factor [10–12]. However, the effect of the beam span on the performance of the double-span assembly has not been explained very thoroughly, and this technical note will show that comparisons on the basis of the beam span alone can sometimes be ambiguous or even misleading unless the context is defined clearly.

In the literature, the bridging capacities of different beam spans are often simply evaluated in terms of the vertical load resistance and the corresponding vertical translation of the damaged inner column, which were actually due to the combined effects of the beam section and the beam span [13]. In fact, Weigand & Berman [11] have stated that even the use of connection rotations would not be sufficient to objectively compare the performance of connections between double-span assemblies of varying spans.

In order to compare the performance of various types of steel double-span assemblies with respect to their development of the catenary mechanism in a straightforward manner, this technical note will

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Table 1
Primary design parameters of frames in the pushdown analyses of Rezvani et al. [10].

Frame and case labels	Beam span l_{0i}[a]	Beam section of the first two storeys	Span-to-depth ratio R_i	$\alpha_i = l_{0i}/l_{02}$	$\beta_i = W_{pi}/W_{p2}$	α_i^3/β_i
Frame 1 (Case2)	4 m	IPE360, $W_{p1} = 1.02 \times 10^6$ mm^3	11.1	0.667	0.434	1.486
Frame 2 (Case4)	6 m	IPE500, $W_{p2} = 2.20 \times 10^6$ mm^3	12.0	1.000	1.000	1.000
Frame 3 (Case6)	8 m	IPE600, $W_{p3} = 3.51 \times 10^6$ mm^3	13.3	1.333	1.596	0.925

[a] Sub-subscript 'i' denotes the frame number.

propose an evaluation procedure that is independent of the optimisation extent of the beam section against plastic hinging under the design floor load. The premise of the procedure is that, in order to compare the performance of various types of double-span assemblies and objectively assess the integrity of the steel beam-to-column connections, all the beams should have been similarly optimised against plastic hinging under the design floor load. The robustness of the proposed procedure will be demonstrated through numerical examples. Issues such as the dynamic effects of sudden column loss, the contributory effects of floor slabs and the second-order effects of topological changes are not relevant to the present study.

2. Identification of the first problem

The effect of the beam span on the progressive collapse behaviour of seismically designed steel moment resisting frames were investigated by Rezvani et al. [10] using three frames having different beam spans l_0 but similar beam span-to-depth ratios R. The design details of the three frames are shown in Table 1 and Fig. 1. The steel beam-to-column connections are assumed to be rigid in the models consisting of beam elements. The variable W_p in Table 1 denotes the plastic section modulus. It should be noted that the span-to-depth ratios of the three frames are not significantly different from each other.

In the pushdown analysis by Rezvani et al. [10], the uniformly distributed load on the beams in the damaged bays was proportionally increased in a quasi-static manner, and it was found that the vertical resistance of the studied frames increased as the beam span decreased, as implied by Fig. 2. The pushdown analysis results of the three frames were interpreted using a load factor ζ_1, which is the ratio of the applied load in the damaged bays to its reference load computed from a given uniformly distributed load q_0 on the floor and the beam span l_0, and the vertical displacement of the top of the removed column δ.

The increased load factor ζ_1 at a given vertical displacement of the smaller span assembly can be attributed to the larger chord rotation of its shorter beams and the way the load factor ζ_1 has been calculated. The latter also affects the apparent progressive collapse strength of the assembly shown in Fig. 2. The load factor ζ_1 of the three frames was computed using the same uniformly distributed load q_0 on the floor, which would be objective only if the beams had been similarly optimised against plastic hinging under the governing design load combination (whether the normal or the seismic load). However, it is not entirely clear whether this condition holds in the case study of Rezvani et al. [10] as the plastic section modulus W_p of the smallest span given in Table 1 appears to be somewhat large relative to that of the largest span if both are to be optimised against plastic hinging (whether under the normal design load or under the seismic load), perhaps due to the discrete availability of the beam sections in practice where the difference in plastic section moduli between one beam section and the next satisfactory size may be significantly higher than that required by the theoretical design. The finding by Rezvani et al. [10] that decreasing the span by half led to 91% increase in the progressive collapse strength should therefore be interpreted with caution since it does not appear to be general.

3. Resolution of the first problem

Herein, the variable F denotes the concentrated load applied at the top of the removed column in the push-down analysis, which is equal to the progressive collapse resistance of the steel double-span assembly. When the design load is applied uniformly on the floors in a rectangular frame, the load F can be expressed as

$$F = \zeta_1 \zeta_2 q_0 l_0^2 \tag{1}$$

For the three frames analysed by Rezvani et al. [10], the reference uniformly distributed load q_0 and the distribution factor ζ_2 are constant due to the square pattern of columns in plan (refer to Fig. 1(a)).

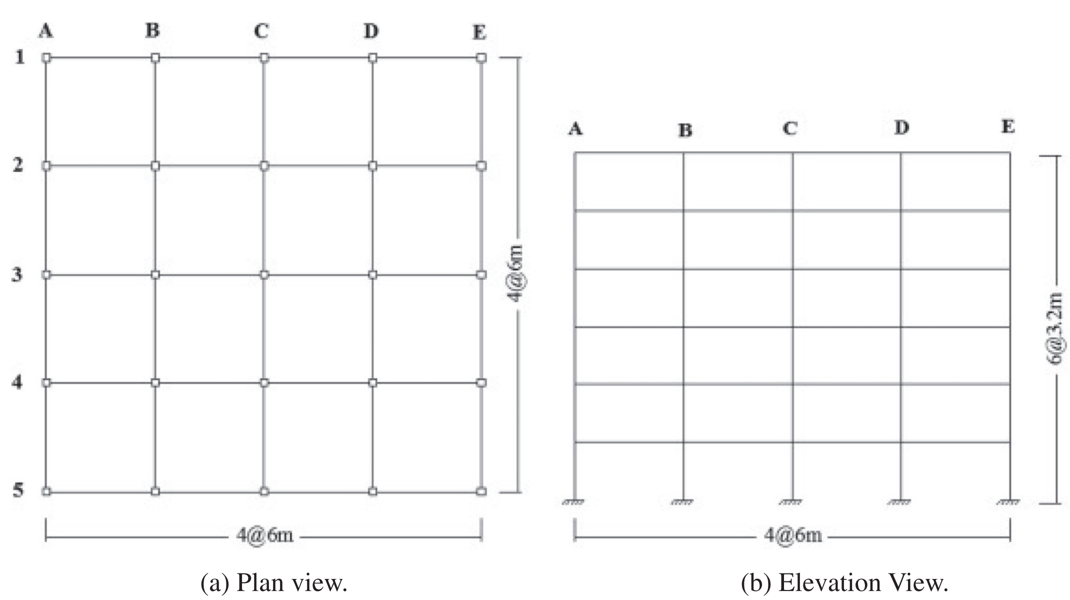

Fig. 1. Topology of one studied frame having the beam span of 6.0 m [10].

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