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Study of the mechanics of progressive collapse with simplified beam models

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ABSTRACT

Methods for assessing structural robustness need to move away from the traditional norms of prescriptive rules and become more similar to those used in conventional structural design. They should therefore be based on a sound understanding of the mechanics of the problem and provide quantitative indication of its effects. Several Codes and Design Guides consider the sudden column removal approach as their principal method for progressive collapse assessment. The level of robustness is defined based on the capability of the remaining structure for sustaining the additional loading imposed by the column loss. Most likely, the beams adjacent to the lost column and their supporting connections form the principal load paths. The present paper presents a detailed study of the response of those components under the conditions experienced following column removal. Suitable analysis approaches that have been previously developed at Imperial College London are employed to investigate the basic features of the behaviour, while several simplifications are applied for exploring particular effects. The study concludes with the development of a simplified method for simulating the nonlinear dynamic response of axially restrained and unrestrained beams following column removal. The capability of the new simplified method to accurately describe performance is demonstrated through a set of suitable applications presented in a separate publication.

1. Introduction

An aspect of structural design that has become increasingly prominent over recent years is the need to provide robustness so that in the event of an unforeseen or unlikely action the consequences will not be disproportionate to the original incident. A particular form of this is design to minimise the likelihood of the type of progressive collapse failures seen, for example, with Ronan Point and the World Trade Centre towers. Because these types of failure involve a complex interplay of gross deformations, dynamics and inelastic material behaviour, conventional structural analysis needs to be used with care and the ideas and concepts associated with effective ways of providing adequate resistance for the more usual gravity and lateral loading cases on buildings are not necessarily appropriate.

One answer is, of course, to conduct nonlinear dynamic Finite Element analysis, including representation of all important structural components and effects [1–4]. Whilst such analyses can certainly simulate the collapses observed in actual incidents, they require very substantial computing resource coupled with considerable skill from those conducting the analyses. Thus they should be considered investigative tools or an approach to be used only in special circumstances and not as being suitable for routine use by those not adequately equipped to undertake such a specialist task. It is for this reason that Codes and Design Guides [5–9] advocate the use of simpler approaches. One of these is the so called 'alternate load path' method, in which a single column is removed from the structure and the ability of the remaining damaged system to safely withstand the applied loading in its new guise is examined. Various approaches for conducting this check [10–12], ranging from linear elastic analysis to full nonlinear treatments including provision for dynamic effects have been advanced. This approach has the advantage that it is conceptually easy to appreciate, permits the making of quantitative comparisons between alternative designs and, depending on the level of complexity associated with the analysis stage, is relatively straightforward to implement. However, of crucial importance is the ability of that analysis stage to allow for all the key physical features of progressive collapse, including being based on a realistic criterion of failure.

The 'alternate load path' approach has been the subject of considerable study at Imperial College London [13–18], where an implementation that uses the avoidance of separation at

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Nomenclature

D	beam effective depth
F_t	connection tensile force
F_c	connection compressive force
$F_{c,0}$	connection limit compressive force developed during compressive arching
K_s	support axial stiffness
K^a	effective axial stiffness of beam system
K_b	equivalent axial stiffness that accounts for beam bending
K^e	elastic tensile stiffness of beam system
K^p	post-limit tensile stiffness of beam system
m_{Rd}	sum of moment capacities of beam end connections
N_0	beam limit axial force developed during compressive arching
q_c	static compressive arching capacity
$q_{c,0}$	static limit compressive arching capacity
q_p	static post-limit flexural capacity
q_t	static tensile catenary capacity
q_y	static plastic capacity
$q_{d,c}$	pseudo-static compressive arching capacity
$q_{d,c,0}$	pseudo-static limit compressive arching capacity
$q_{d,c,\max}$	pseudo-static maximum compressive arching capacity
$q_{d,p}$	pseudo-static post-limit flexural capacity
$q_{d,t}$	pseudo-static tensile catenary capacity
$q_{d,y}$	pseudo-static plastic capacity
r_c	ratio of difference between the connection compressive and tensile resistances to the beam limit axial force
R	sum of rotational stiffness of beam end connections
w	beam deflection
$w_{d,c,0}$	beam deflection associated with the pseudo-static limit compressive arching capacity
$w_{d,c,\max}$	beam deflection associated with the pseudo-static maximum compressive arching capacity
$w_{d,f}$	beam ultimate deflection
$w_{d,t}$	beam deflection associated with the minimum pseudo-static tensile catenary capacity

beam-to-column connections based on available levels of deformation capacity has been used as the criterion for robustness. In addition, by employing an energy balance approach [14] the need for explicit dynamic analysis is avoided. The result is a method that, when implemented for a grillage approximation of the floor response in association with an advanced slope-deflection technique [18], requires orders of magnitude less computation than any based on the use of analysis software. An outline of this approach is provided in the next section of this paper.

The ability to conduct the structural analysis phase of the 'alternate load path' method quickly and, moreover, to do this using explicit expressions and relationships means that insights into behaviour that would be prohibitively expensive if attempted using nonlinear FE approaches may be obtained. In this paper the relative importance of several features of progressive collapse are investigated; in some cases simplifications of the original method are used to explore particular features. The result is better understanding of the various phases of progressive collapse and the different resistance mechanisms associated with each, together with a clearer indication of the role and significance of different components within the structural system. The work is particularised to cover multi-storey building frames of composite steel/concrete construction but it is believed that many of the points made have more general significance.

2. Overall nonlinear static beam behaviour following column loss

The basic concept of column loss in a multi-storey frame is illustrated in Fig. 1(a), which shows the loading, deformations and forces associated with the double-span beam arrangement required to bridge over this loss. As discussed in [14], the response of the complete frame may be approximated with good accuracy by considering the behaviour of a single floor grillage in terms of the performance of its longitudinal and transverse beams. Depending on the exact arrangement, the beams may be regarded as axially restrained, i.e. the surrounding structure provides resistance to horizontal pull in, or may, in the case of corner beams, need to be treated as cantilevers. Assuming a symmetrical beam arrangement, Fig. 1(b) shows the idealised system that needs to be analysed, with the beam-to-column connections represented by rotational springs having rotational stiffness, deformation capacity and moment capacity values that depend upon the exact connection configuration and the resistance to pull in represented by an axial spring. Full details of the extended slope deflection equations allowing for non-uniform beam properties due to composite action, large deflections and axial-bending interaction are available elsewhere [18]. Similarly, modelling of connection behaviour in terms of strength, stiffness and rotation capacity by using the arrangements shown in Fig. 1(c) is available [17].

By combining the beam analysis with connection behaviour, the full load–deflection response of the beam from its initial linear elastic phase up to its defined failure when the deformation capacity of the more critical end connection is reached can be obtained. Fig. 2(a) shows a typical result, on which a series of distinct regions are marked. Not every region will always be present since, for some systems the failure criterion will be reached beforehand. For instance, the tensile catenary stage is not attained when failure takes place during the earlier stages. It is for this reason that design approaches, such as the tying method, that presume the attainment of the final tensile catenary stage without checking that such behaviour is possible are potentially unsound [19–23].

Initially, the beam responds elastically in the manner assumed by small deflection elastic analysis. The development of some inelastic regions will then cause a softening; it is during this stage that axial forces begin to develop within the beam leading to the compressive arching stage. At some point during this phase beam axial forces start to reduce, pass through zero and then become tensile, resulting in the transient tensile stage. Finally, providing the end connections have sufficient ductility, the tensile catenary stage is attained. Consideration of the conditions under which each of these phases develops and the system properties needed for them forms the major theme of this paper.

During this growth of vertical beam deflections important changes occur in the forces that the beam is required to withstand and these are illustrated in Fig. 2(b). Initially the beam functions in flexure but as vertical deflections grow axial compression develops as a result of resistance to outward movement. At some stage, point C, these forces attain a maximum whereupon they start to reverse, passing through zero and becoming tensile.

Taken together, Fig. 2(a) and (b) illustrates the complex and changing nature of beam response. Only as a result of understanding when each stage occurs, the conditions necessary for it to occur and the role of the key physical features of the configuration in controlling the extent to which it occurs can a full appreciation