Proposal for an alternative to ultimate limit state design that will improve robustness for steel framed structures

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Abstract
This paper will argue that it is impossible to predict the maximum load a structure will receive during a working life, particularly given the hazards presented by global terrorism. Given the unpredictable nature of extreme loads, structures should arguably be designed to fail in a robust and ductile manner if the serviceability loads are exceeded. The present limit state design approach is creating over strong beams with relatively weak connections. It is therefore failing to ensure ductility. The alternative approach advocated is similar to the capacity design approach used in seismic design, whereby the upper-bound flexural strength of beams is determined; thereafter components further down the load path are designed to receive these loads. The alternative approach would lead to savings in steelwork for structures governed by ultimate limit state checks.

Introduction
It is feasible to design routine structures to survive blast or impact loading, since composite beams are generally very ductile and are therefore capable of absorbing large amounts of energy. The prime factor to ensure robustness is that connections should be capable of resisting the loading they receive from the beams. However, ultimate limit state (ULS) design may significantly underestimate the load capacity of beams, thus creating a mismatch between beam strength and that of the connections. There are four main reasons why ULS design can underestimate beam strength:

1. *Strain hardening.* Hot-rolled sections begin to strain-harden shortly after yielding. During the rotation of a plastic hinge in a composite beam a considerable amount of extra strength will be generated due to the comparatively large strains that occur in the lower half of the section.
2. *Over strength steel*. A survey of over 7000 mill tests[1] collected from two leading EU producers revealed that on average yield stress was 16% higher than the nominal value assumed in design, if material thickness is greater than 10mm. For thinner material the average yield stress was found to exceed the nominal value by 37%.
3. *Moment resisting nominally pinned connections*. Nominally pinned connections such as the partial depth end plate connection have been shown to transform into moment connections when subjected to significant beam end rotations[2]. This is because a couple is generated between the bolts and the lower beam flange given sufficient rotation, see Fig. 1. The test frame shown in Fig. 2 represented a bay of a multi-storey building. The moments generated in the partial depth end-plate connection in this frame represented 40% of the nominal plastic moment capacity of the connected beam in sagging, see Fig. 3, substantially increasing load capacity. Furthermore, the prying action in the connection was shown to lead to the failure of the upper rows of bolts, further increasing the likelihood of a brittle connection failure.
4. *Strain rate effects*. Blast loads result in very high rates of strain and the dynamic yield stress can be as much as 70% higher than the static yield stress assumed in design,

although such an enhancement would be associated with a component located close to a high explosive.

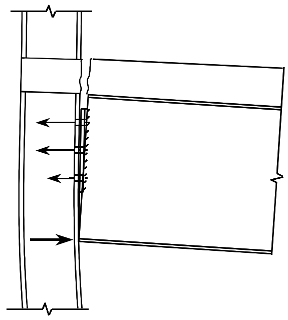

Fig. 1: Behaviour of partial depth flexible end-plate connection subjected to large rotation

Fig. 2: Composite beam test at BRE

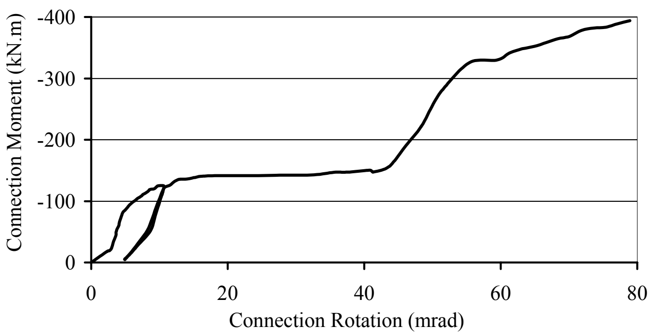

Fig. 3: Moment vs. rotation curves for partial depth flexible end-plate connection

A realistic measure of the true strength of a typical composite beam with typical connections can be gained by consideration of the frame test discussed earlier (Fig. 2). The design moment capacity of the section, M_c equalled 1007*kN.m*, although the maximum observed flexural strength was 1500*kN.m* (due mainly to strain hardening and over-strength steel). The test also demonstrated that the flexible end-plate connection could generate a moment of 400*kN.m* even though it is a nominally pinned connection. Thus the beam and connection system was found to be capable of resisting almost double the ULS loading. This can be regarded as a conservative measure of the true flexural strength since the test was aborted after only 300*mm* of mid-span deflection due to concern regarding the rupturing of connection bolts. Furthermore, the yield strength of the beams was close to the nominal value at 294N/mm^2. As stated previously, this can often be much higher.

Transfer to a capacity design approach

A common feature of extreme loads is that they are intense and of short duration. This applies to impact, seismic and blast loads. A further feature is that they result in large deformations. It is the ability to absorb energy through ductility that is the primary factor in survivability. Unfortunately extreme loads do not fit easily within the ULS design approach because the implication of the ULS load being exceeded is not considered. Furthermore, ULS design, based on lower-bound strength calculations, has been shown to produce a mismatch between the strength of beams and their connections. When combined with low ductility connections, this approach will produce brittle buildings.

These problems can be overcome using a design philosophy originally termed *capacity design*. Developed in New Zealand, it targets failure due to the lateral loads from earthquakes to specifically designed plastic hinges[3]. This is achieved by ensuring that all other components in a frame have a strength exceeding that required to generate the plastic hinges identified. Thus, the systems response is considered rather than the response of components in isolation from each other. A transfer to a modified form of this approach would overcome the shortcomings highlighted above. In such an approach the initial objective for the engineer would be to achieve fitness for purpose under working loads. Beyond that no attempt would be made to predict the maximum loading a structure would receive (since it is unpredictable for at-risk structures). Thus, the lower-bound flexural strength of beams would not be required to resist the ULS loads of 1.4 dead + 1.6 imposed. Instead, the upper-bound flexural strength of beams would be established. All other beam failure modes and components further down the load path would be designed to receive these upper-bound loads, not the conventional ULS loads. Importantly, the interaction between components throughout the load path is considered. This would improve robustness by targeting flexural strength of beams to be the weakest mode of failure. It would also produce economies in beam design because the serviceability limit state (SLS) loads would govern beam sizing. The form that the approach could take is sketched in Fig. 4.

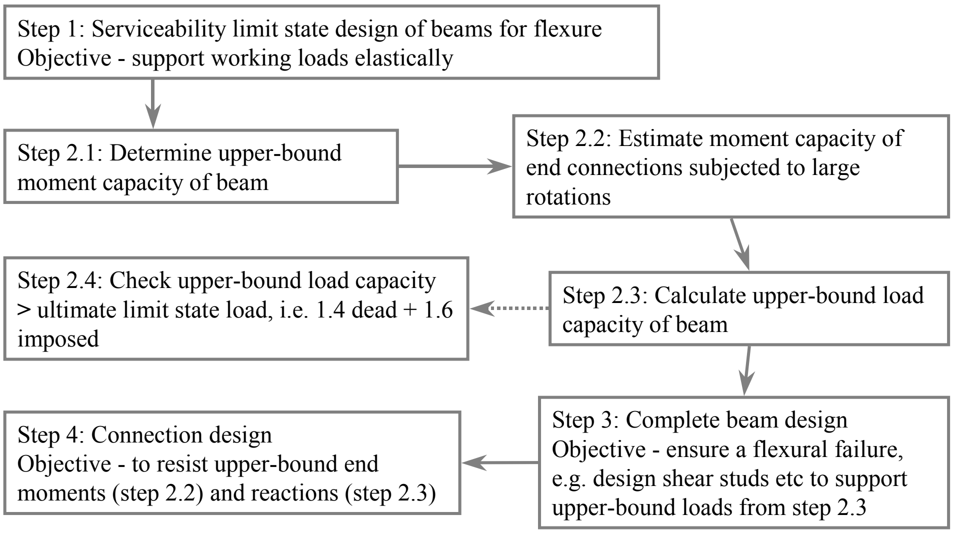

Fig. 4: The alternative design approach

Conclusions

This paper proposes a modification to the limit state design approach that would improve the robustness of multi-storey buildings, as well as providing economies in the use of steel. Robust frames require relatively weak beams, but strong connections and columns. Limit state design may provide the reverse. The composite beams used in the majority of steel framed buildings are shown to possess a “hidden” reserve of strength. When subjected to the large sagging deformations associated with terrorist attacks, these beams are capable of resisting typically twice the design load. This creates a situation whereby the weak point in a frame can be transferred to either the connections or the columns, leading to non-ductile and potentially catastrophic failures in the event of severe overloading.

To address this problem a systems approach is advocated, whereby ductile beams are designed to resist only working loads, albeit elastically. Thereafter the upper-bound flexural strength of beams is established. It is the corresponding upper-bound reactions that are used for the subsequent design of components lower down in the load path. This approach would improve robustness. It would also provide substantial economies in the use of construction materials.

Acknowledgements

This paper is based on Paper No 1637(J): The design of steel framed buildings at risk from terrorist attack, submitted for publication in The Structural Engineer.

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