

## ROBUSTNESS OF STEEL FRAMED STRUCTURES IN FIRE

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### Abstract

Progressive collapse of a structure occurs when an initial local failure causes a sequence of failures in other elements, eventually resulting in collapse of a disproportionately large part of the structure, or even the entire building. It has been an important structural issue in the UK since the collapse of a large part of the Ronan Point apartment building in London in 1968 (The Institution of Structural Engineers 1969). The collapse of the World Trade Center towers (Shyam Sunder *et al.* 2005) has attracted attention to the robustness of steel structures in fire, within which the need to evaluate the performance of the steel connections at elevated temperature has become a key topic. The fire tests at Cardington (Kirby 1998), and the collapse of at least one of the WTC buildings (Gann 2008), illustrated that the connections are potentially the most vulnerable parts of a structure in fire. In steel-framed buildings connections must provide structural continuity, redundancy and ductility. Failure of connections can cause structural discontinuities and reduce the resistance provided by alternative load paths. An accurate understanding of connection performance is therefore essential to the assessment of structural robustness, and so to structural design against progressive collapse. It has traditionally been assumed that beam-to-column connections have sufficient fire resistance, because of their lower temperatures and slower rate of heating than the members to which they are attached. However, the full-scale fire tests at Cardington in the 1990s showed that the connections are actually more vulnerable than previously assumed during fires, because the forces and deformations to which they can be subjected during a fire differ significantly from those assumed in general design, either for the ambient-temperature Ultimate Limit State or for the Fire Limit State. The internal forces in connections change in three stages, starting with moment and shear at ambient temperature, becoming moment, shear and compression due to thermal restraint to beam expansion in the initial stages of a fire, and finally changing to shear and tension at high temperatures, when beams essentially hang in catenary. If a connection does not have sufficient resistance or sufficient ductility to accommodate the large rotations and normal forces which it experiences at any stage of fire exposure, connection fracture will occur, and this may lead to extensive damage or progressive failure of the structure (Wang *et al.* 2010).

These complexities are difficult to represent in design analysis. Full-scale structural testing is expensive, and its reliability is considerably affected by the particular test methods and laboratory facilities used. Hence, assessment of the robustness of steel connections in fire will inevitably rely largely on numerical modelling using appropriate predictive methods. However these are unlikely to include general-purpose finite element modelling of large structural subframes, including the varied details of the connections, because of the complexity of such models and the time required to create them. The most promising alternative to detailed modelling is the component method, a practical approach to the modelling of connection behaviour which can be included within global three-dimensional frame analysis. In this method, which was initially proposed by Tschemmernegg and Humer (Tschemmernegg, Humer 1998) for ambient-temperature semi-rigid frame design, the connection is represented by an assembly of individual components with known mechanical properties; it has been validated as an adequate, if not 100% accurate, representation of the key behaviour of certain connection types in fire and is included in current Eurocode steel frame design. For use in the context of robustness in fire, component characterization must include elevated-temperature behaviour and represent this behaviour up to failure (in many cases up to fracture).

The essential problem of maintaining the integrity of connections in fire is of keeping the forces in key elements (such as bolts, welds and plates) low enough that these elements do not fracture, causing a cascade of subsequent fractures and detaching members from their supports. The stages of behaviour at which such fractures may take place are:

- The initial heating stage, when thermal expansion of beams will create very high compressive forces if there is little compressive ductility in their connections or the ability for the connected structure to move without causing further damage. The latter is difficult to achieve except at the perimeter of a building. In terms of current connection types, very few allow more than a few millimetres of movement before hard contact occurs. Beyond this stage the lack of further compressive ductility may cause the beam ends to rotate rapidly as thermal buckling of the beam happens, so that upper bolt-rows may fracture while the net force is still compressive.
- The high-temperature stage, when the beam has largely lost its bending resistance and hangs essentially in catenary tension between its connections. To some extent this tension is attenuated by the deflection caused by the net thermal expansion of the beam, and can be attenuated further by any capacity for tensile movement at each of the connection components. Once again the upper bolt-row will often be the most critical in triggering a progressive fracture sequence.

It is possible to estimate approximately the demand for connection ductility imposed by a frame layout as a function of beam spans and temperatures.

Fire causes relatively slow growth and decay of structure temperatures in buildings, compared with key dynamic characteristics such as the periods of their natural frequencies, so the evaluation of thermo-structural behaviour is conventionally carried out by static or quasi-static numerical analysis. Because of the nature of quasi-static analysis, it can usually only track a structure's behaviour up to the point at which its first component fails; in limited cases this may extend to more than one component's failure. In reality a connection may either be able to regain its stability after the initial fracture of one (or a few) components, or the first failure may trigger a cascade of failures of other components, leading to complete detachment of the supported members. In considering these possibilities it should be recognized that the forces on a connection may be relieved considerably after an initial component failure, so a cascade of failures is not inevitable. If connections are to be designed to avoid the possibility of members becoming completely detached, then numerical modelling must be capable of predicting the sequence of failures of components, rather than considering the first loss of stability as signifying building failure. After a beam has completely detached from its connections, the load-paths within the remaining frame are changed, and the effective lengths of columns may be changed by the loss of restraint from the detached beam. In order to model these effects it is necessary to use dynamic analysis so that loss of stability and re-stabilization can be tracked, including the movements of disengaging members and the load-sharing mechanisms between members which still maintain their integrity and stability within the remaining structure, until total collapse happens.

**Keywords:** steel framed structures, connections, robustness, progressive collapse, fire.

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