ABSTRACT:

Accidental fires are a continuous threat for many structures, and the behaviour of the structural system during fire has to be computed and documented. Structures exposed to real fires will be heated up non-uniformly with large temperature differences between different parts of the structural system, which causes complex non-linear systems. During the fire, the mechanical strength of the structural system will be degraded, and single components could lose most of its capacity on an early stage of the fire. However, substantial reserve capacity exists in continuous 3D frame systems due to second order effects and load shedding. The present paper addresses the realism in use of non-linear finite element tools to describe the mechanical response of 3D frames exposed to fire. Welded connections undergo large plastic displacement and rotations, and the behaviour of these critical components at elevated temperatures, are compared with results from laboratory tests. The paper documents that real structures are highly ductile and are able to transfer substantial loads through the connections at high temperatures.

INTRODUCTION

Resistance against accidental loads has to be documented for offshore structures as well as for building systems. Accidental fires are a continuous threat for many structures, (as illustrated in Figure 1, showing a fire in the parking area of a high rise building), and the behaviour of the structural system during the fires has to be computed and documented.

Structural systems are normally thermal insulated in order to slow down the heating process of the components. Correct design of the structural system could make the structure less vulnerable for component heating achieving a substantial delay of the complete collapse. In many cases, this extra prolongation before theoretical collapse is enough to prevent this to happen if for example the amount of fuel is limited.

Looking into the fundamental behaviour of 3D structural system exposed to fire is therefore an important safety issue.

Different structural materials (steel, concrete, aluminium, wood, etc) behave differently with respect to ductility and loss of strength due to heating. However, the different structural systems have several things in common:

- Strength degrades down as temperature increases
- Material expands for increasing temperature
- Structural systems are composed by components, which are assembled (joints/connections)
- Failure is likely to start in the connections
A complex 3D structure contains a large number of details, and in the present paper focus is set on the behaviour of beams and beam connections.

**BEHAVIOUR OF BEAMS**

Structures are normally designed using linear methods, and the beams are carrying the loads in terms of bending moments and shear forces. Appropriate safety factors are used in order to avoid material yielding, and the beams behave elastically without any 2nd order effects activated.

In the linear (1st order) analysis, the simply supported beam without any axial fixation has the same capacity for transverse loading as a fully axially fixed beam. The governing parameter for the elastic capacity is normally the bending moment resistance at the beam midspan.

In Figure 2, the behaviour of an axially fixed and an axially free beam are presented in terms of force-displacement curves. Both beams are subjected to a concentrated lateral force at midspan. Load ratio = 1.0 is the midspan load causing a midspan bending moment equal to the plastic bending capacity ($M_P$) of the actual cross section. The span length of the beam is 6m.

![Simply Supported vs Axially Fixed Beam](image)

**Figure 2** Ultimate Resistance of a axially free and axially fixed beam.

The two P-δ curves are almost identical for moderate deformations, ($\delta/L < 1/100$). When the load ratio has reached 1.0, the axially free beam cannot carry any further load increase, and the deformations will increase.
dramatically and is likely to collapse, even for a small increase in the load, (or a small reduction of resistance due to heating by the fire).

The beam with end fixation behaves very differently. As the deformations increase, 2\textsuperscript{nd} order (geometrical) effects are gradually activated, which in this example means the increasing axial force, (membrane effect). Depending on the ductility of the system, the ultimate resistance could be increased by a factor in the order of 3. For example, a reduced cross sectional capacity, (due to heating), will only cause the deformation to increase until a new equilibrium configuration is found.

In real frame systems, the beam-ends are neither fully fixed, nor completely free. The ends are partly fixed, (axially and rotationally), in the continuous beam – column system.

Figure 3 shows the behaviour of a 2D frame where one beam is subjected to a lateral midspan force. All deformations are un-scaled. In the first phase of the loading process, (image to the left), the beam is still straight, and carries the load by bending/shear. For increasing load, beyond the limit for pure bending capacity, the deformations are increasing, and 2\textsuperscript{nd} order effects are activated. The load is now carried by a combination of bending and membrane action. For an increasing load, plastic hinges are formed at the beam-ends and at midspan, and the load is mainly carried by membrane (axial) force. In the images, the length of the load “arrows” indicates the relative magnitude of the forces at the different stages. In the P_δ curves, the actual positions corresponding to the loading stage are marked with a red circle.

The reserve capacity due to the 2\textsuperscript{nd} order effects demonstrated in this simple example, is caused by the axial force due to the forced elongation of the beam. If these axial forces cannot be anchored at the ends, these reserves would not exist.

*Anchoring the axial forces in the joints is therefore of great importance for the reserve strength.*

**CONNECTIONS**

Ideally, the connection should, at least, have the same ultimate resistance as the beam, but this is not necessarily the case for real structures.

In order to demonstrate the importance of the connections, the simple system showed in Figure 4 is analysed. The beam is not fully fixed at the ends, but has a certain axial resistance. If the connection resistance is equal to zero, the system is reduced to a simply supported beam, and for connection resistance = beam resistance, the fully fixed system is obtained.

The behaviour of the connection is modelled by use of a non-linear spring with varying resistance, and with a certain ductility, (elongation corresponding to 10\% of the beam height is used in the current example as a fracture criterion, and this corresponds to typical behaviour of real connections).
The ultimate resistance of the beam system is computed for varying resistance of the connections, (i.e. ability to anchor the 2nd order, axial forces). In Figure 5, the ultimate resistance, (in terms of lateral force), relative to the resistance of an axially free beam is presented for varying connection performance. The joint resistance is expressed as capacity relative to the beam ultimate axial resistance.

![Figure 4 Beam with limited joint resistance](image)

The ultimate resistance will be limited by:

- The peak axial force which can be anchored in the joint
- The joint “elongation” before the capacity drops to zero, (fracture).

From the plot, (to the left), it is seen that even for connections with 50% capacity of the beam, more than twice of the ultimate resistance is obtained. For connections with strength equal or larger than the beam, an ultimate resistance more than 3 ½ times of that of the axially free beam is obtained. In the plot to the right, the beam midspan deformation, \((w/L)\), is presented for the different joint capacity cases, and deformation in the order of 4-6% will occur if the 2nd order (reserve) capacity should be activated.

![Figure 5 Ultimate resistance as a function of joint strength](image)

*Information of the behaviour of the connections is therefore important for the documentation of the ultimate resistance of the beam system.*

**BEHAVIOUR OF T-JOINTS**

In connection with non-linear FE analyses, 3D frames are normally modelled with beam/column finite elements, which are ideally connected in joints, (Figure 6, model to the left). Such models implicitly assume “unlimited” resistance of the connections.

In more advanced simulations, the load transfer between the components are accounted for by use of “connection elements”, (Figure 6, model to the right), which describe the resistance of the actual connections typically in
terms of $P, \delta$ and $M, \theta$ curves as used in the previous section. This type of modelling makes it possible to set limitations on the load transfer between the components.

Such advanced FE-simulations are, however, valid only if the data used to describe the resistance of the connections are realistic.

As shown above, the anchoring of the beam axial forces, which are introduced when the deflections are increasing, are important if the system should benefit from the 2nd order reserve capacity.

For a typical beam with lateral load, the connections will experience forces in following sequence:

- Bending moment due to rotational constraint in the joint
- Bending moment + increasing axial force. Increasing angle between the axial force and the original beam axis.
- Beginning failure of the connection. The forces transferred through the connection have to be reduced.

In connection with fire, the loads, (weight of the structure and equipment), are constant during the entire fire, but the capacity of the structure is gradually reduced. The reduced capacity will cause gradually increased deformations, which introduce 2nd order axial forces in the beams. The load pattern in the joins are following the main steps as described above, and it is of interest to find out how real joints behave on similar load exposure for room temperature as well as for elevated temperatures.

Questions:

- What is the joint resistance for ideal axial loading, (load parallel with the beam axis)?
- What is the joint resistance for a combined moment and “skew” axial loading? (which is the load pattern expected for beams/column connections in a 3D frame).
- How does the temperature influence the joint resistance? Reduction similar to the reduction of the basic material, or very different?

Two sets of identical T-joints, (HE140A, material St52 with yield stress = 350MPa), are tested, [1], one set with the load direction parallel with the beam axis and one set where a “skew” loading. For each set, one specimen is tested at room temperature and one at 500°C. The tests at elevated temperature are heated 1 hour before the load is applied.
The T-joints are heated locally, (approximately 2 profile heights to the sides of the intersection), using electrical heating mats. The force is introduced “far away” from the joint, in a part of the beam unaffected by the heating. A fairly uniform stress distribution over the beam cross section near the connection is therefore assumed.

Figure 7 describes the behaviour of the T-joints exposed to a tension force parallel with the beam axis. The plot, (to the left), describes the average beam stress versus the relative elongation of the actual component. The relative elongation is taken as the displacement at the beam end (where the load is applied) divided by a given length of the beam. For the tests at room temperature, the length is taken as the beam length, (assuming uniform elongation over the beam), while the elongation is expected to be concentrated to the heated section for the tests at 500°C. The actual lengths used are 1000mm for the tests at room temperature and 350mm for the tests at 500°C. (Due to strain concentration, the real strain within the component is expected to be significantly larger than this averaged “strain”.)

As seen from the figure, the observed average peak stress is more than 450MPa, (specimen does not fail), at room temperature and 300MPa at 500°C. The heated T-joint has a constant resistance over a wide range, and for an average strain of 7%, (total elongation of 30mm), the component fails due to rupture in the web, (see figure 7 to the right).

Figure 8 describes an identical T-joint as presented in Figure 7, but with a completely different load pattern. This test is designed to represent the load history, which will take place in a real structure, starting with bending moment, and ending with a plastic hinge and axial tension force. The test set-up is described in Figure 8.

The idea is to find the axial capacity of the joint after the joint has been exposed to large plastic rotations, (in the order of 10° rotation). This corresponds to a midspan deformation in the order of L/12, which is a large deformation. At this stage the beam carries the load by membrane action only.

The realism to assume that the beam axial force could be anchored at such extreme plastic rotations of the joint is therefore investigated. Two tests are performed: one at room temperature, and one with the joint heated to 500°C.
The curve to the left in Figure 9 shows that an average stress of 400MPa is obtained at room temperature, while the peak stress at 500°C is 200MPa. Failure occurs in both tests. The relative displacement (or average axial strain) corresponding to joint failure is 4.5%, (total elongation of 45mm), at room temperature and 6%, (total elongation of 20mm), at 500°C. Again, it should be emphasized that this “strain” measurement does not describe the local strain levels within the cross section, which is likely to be significantly higher in this case with combined bending and axial force. The relative displacement is, however, a useful way to document how much the connection is “stretched” axially prior to fracture. Such ductility measurements are required if the joint capacity is introduced in the FE calculation. Otherwise, the connection may be stretched unduly.

The tests document the ability of the T-joints to transfer axial force depending on the direction of the force to be anchored. In Figure 10, the results from the four T-joint tests are presented together with the plot of the Eurocode 3, “effective yield stress” temperature degradation curve. This curve describes the impact of the temperature on the yield stress, using properties at room temperature as the reference. For an increasing temperature, the material is loosing its capacity, and for temperature above 1200°C, no capacity is left.

The results from the tests are presented in terms of the average axial stress in the beam to be anchored, and can therefore be compared directly with the basic material properties. The component itself will be able to handle a stress corresponding to the material’s yield stress, but the connections are likely to fail for a lower average stress level. The ideally loaded T-joints have a fairly high resistance, while unfavourable loading conditions, (skew loading), reduces the anchoring capacity.
The degradation due to temperature for the three cases have almost the same slope between 0\(^\circ\) and 500\(^\circ\), and is likely to follow the same trend also for higher temperatures. Using the material degradation curve also for the connection capacity seems to be a good approximation. With known performance data of connections at room temperature, the performance at elevated temperature could therefore be derived.

The tests clearly state that the connections are likely to fail rather than the basic components, (beam profile itself), if loaded to the extreme. Using non-linear finite element analysis assuming 100\% force transfer through the joints for all loading and deformation conditions will therefore over predict the structure’s ultimate resistance. Realistic limitations on force transfer between the basic building components have to be included in the finite element model.

However, the tests also document that the connections are able to anchor a substantial portion of the beam axial forces, also for extreme deformations. This performance exists also at elevated temperatures, (when reserve capacity is needed). In many cases this reserve will be sufficient to carry the loads.

In Figure 5, it is shown that 2\(^{nd}\) order, (geometrical) effects represent a substantial reserve, even for structures with limited joint strength. In the actual test, the connections are able to transfer 60-90\% of the beam’s ultimate axial force. With such performance of the connections an ultimate resistance of the beam system with magnitude 2-3 times higher than the resistance according to linear design could be achieved.

Deformations at the beam midspan in the order of L/15 could occur without failure in the T-joints.

CONCLUSIONS

Reserve capacity exists in 3D steel frames due to 2\(^{nd}\) order geometrical effects, in particular for beams exposed to lateral loads. The level of reserve depends on the connections’ (anchoring) capacity, i.e., the frames’ ability to remain continuous also under extreme conditions. In connection with accidental fires, this reserve capacity may delay the time to structural instability, and even better, avoid structural collapse.

Tests have demonstrated that correct designed and fabricated connections are able to anchor substantial forces, even for extreme deformations (highly ductile). The ultimate resistance of a given connection is likely to follow the temperature degradation curve for the actual basic material.

Non-linear finite element analyses with realistic modelling of the non-linear ductile behaviour up to fracture of the connections are recommended.

REFERENCES