Advanced structural fire design of offshore structures

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ABSTRACT: Resistance against extreme loading has to be documented for offshore structures. Fire is a continuos threat as large amounts of oil and gas are passing through the installations. In a conventional design procedure, the 'cold' structure is optimized for the different mechanical loads. The responsibility of the structure is then transferred to the safety department, which normally recommend use of passive fire protection as the only way to avoid structural failure during fire. Including fire in the design process will result in more fire resistant structures which means reduced use of passive fire protection and thereby reduced initial and maintenance costs. Better understanding of the structural behavior under fire exposure opens also for extended use of light alloy structures. Design against accidental fires should be included in the design process conducted by the structural engineers in close cooperation with the safety department.

1 INTRODUCTION

Adequate resistance against extreme loads has to be documented for offshore structures. Accidental fires are a continuos threat as large amounts of oil and gas are passing through the installations. The requirements to the structures are based on general safety studies, and are often expressed as minimum time to failure of the most important structures exposed to on or more fire scenarios.

Structures exposed to fire will be heated up, and the heating rate is dependent on the intensity of the fire, the surface/mass ratio of structural components, the surface properties of the actual material and finally the presence of passive fire protection covering the surface.

The main effects of the heating are thermal expansion, reduced elastic modulus and yield stress and creep. Thermal expansion takes place from the very beginning, and due to the fact that the different components will be heated differently (different exposure and surface/mass ratio), the thermal expansion will vary from one structural component to another. The thermal expansion results in member axial forces and bending moments. Thin walled components are the most vulnerable, and if the ends are restrained member buckling take place even for moderate temperature increase, ($\Delta T=100°C$). As the yield stress and E-modulus reduce, and after a given time the whole structure may collapse.

In a conventional design procedure the 'cold' structure is designed and optimized by the structural engineer for the different loads. The "responsibility" for the structural performance is transferred to the safety department. The major measure to avoid structural failure is by using *passive fire protection*, i.e. thermal insulation slowing down the heating of the structural components. The requirements to the protection is expressed as maximum allowable temperatures after a given time (for instance 400°C after 1 hour). The necessary thickness of the passive fire protection is taken from tables depending on the surface to mass ratio (*HP/A ratio*), the fire exposure, maximum accepted structural temperature and the fire duration. Normally, this work is carried out by safety engineers who seldom have their background from structural engineering and *thus may have problems to suggest alternative structural design*. One consequence of using the temperature criterion for determining the passive fire protection is that the smallest components get the largest thickness and vice verse.

The fire scenarios used in design are often limited to standardized uniform 'fires' (e.g. 200 kW/m^2) which are assumed to expose all structural members "all way around" from the very beginning to the very end of the fire. Such assumptions are often overly conservative, yielding excessively high costs for fire protection. In some cases it may exclude the use of f ex light weight material such as aluminum alloys.

By contrast to such simplistic considerations, advanced computational fluid mechanics (CFD) codes recently developed allows accurate simulation of the combustion process from the ignition to termination. It is possible to trace differences in exposure of the various structural components giving a far better description of the real fire.

Similarly, advanced non-linear structural analysis tools are available. By simulating the mechanical response of structures exposed to fire it is possible to document and evaluate the consequences of a fire in a much more physically correct manner. It is possible to trace failure of components, force redistribution and global failure during the fire. With this knowledge it is possible for the structural engineer to suggest a more optimum design of the structure with respect to the effects of fire. In particular, alternatives to passive fire protection can be evaluated.

2 FUNDAMENTAL BEAM BEHAVIOR

To describe the behavior of complex structures exposed to fire, it is necessary to understand the fundamental behavior of the structural members.

2.1 *Simplified beam behavior*

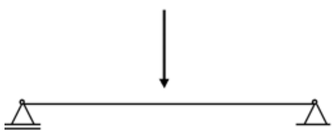

Figure 1. Simply supported beam.

The simply supported axially free beam shown in Figure 1 carries the midspan force by bending, and in connection with linear design, no axial forces are introduced

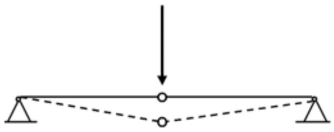

Figure 2. Fixed beam with middle hinge.

The hinged beam described in Figure 2 which is axially fixed in both ends, is assumed to carry the same force through *axial forces* only (pure membrane). In this simple example, the change in geometry due to *stress induced strain* is disregarded.

The two static systems will have a very different behavior during fire.

For the simply supported beam, the elongation of the beam due to thermal expansion will not change the system, but the material degradation due to increased material temperature will reduce the bending capacity.

For the hinged beam the thermal elongation of the beam introduces lateral deflections as indicated by the dotted lines. This deflection changes the load carrying system in a *positive* manner: Increased deflection results in increased capacity.

To demonstrate these effects, the following simple example is given:
Assume a steel pipe with diameter 1.0m and thickness 50 mm spanning 10 m. The steel with initial yield stress 350 MPa is assumed to be degraded according to Eurocode 3, /4/.

For the simply supported beam the ultimate midspan force P_u is taken as:

$$P_u = 4 \; \sigma_y \; W_p / L \qquad (1)$$

where

σ_y : Current yield stress
W_p : Plastic sectional modulus
L : Span length

For the hinged beam following ultimate midspan force is taken:

$$P_u = 2\ \sigma_y\ A\ \sin(\varphi) \qquad (2)$$

where

σ_y	:	Current yield stress
A	:	Cross section area
φ	:	Angle = ArcTan ($2\delta/L$)
δ	:	Midspan deflection

The midspan deflection caused by thermal expansion only is expressed as follows:

$$\delta^2 = [\ L/2\ (\ 1 + \alpha T\ )\ ]^2 - (\ L/2\ )^2$$
$$= L^2/4\ (\ 2\ \alpha T + \alpha^2\ T^2\ ) \qquad (3)$$

where

α	:	Thermal expansion coefficient
T	:	Temperature increase of beam

For the simply supported beam the capacity as given by Equation (1) is governed by the degradation of the yield stress. The equation is used for varying temperature of the beam which means changed yield stress. The initial capacity of 6.3 MN is kept unchanged up to 400° C, and then the capacity drops according to the material degradation curve.

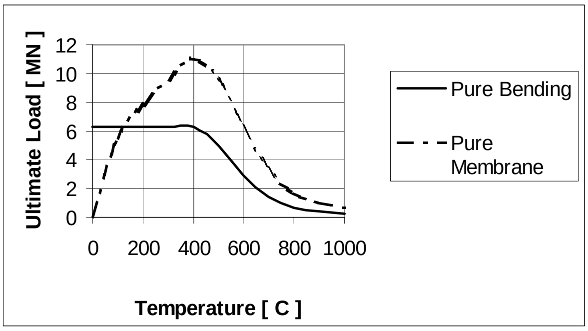

Figure 3. Bending capacity versus membrane as function of temperature.

For the axially restrained beam the capacity as given by Equation (2) is governed by the degradation of the yield stress as well as the angle φ. With an initial deflection equal to zero, the capacity curve of the hinged beam starts in zero when the stress induced strain is disregarded, (sin (0)=0). Increasing temperature results in increased deformation which means *improved membrane load carrying,* see equations (2) and (3). When the temperature passes 400°C, however, the material degradation is stronger than the improved geometric stiffness due to further increase of the midspan deformation, and the capacity drops. In Figure 3 the "ultimate" capacity is shown as function of temperature for the two simplified cases. The current maximum of the two cases are plotted in Figure 4.

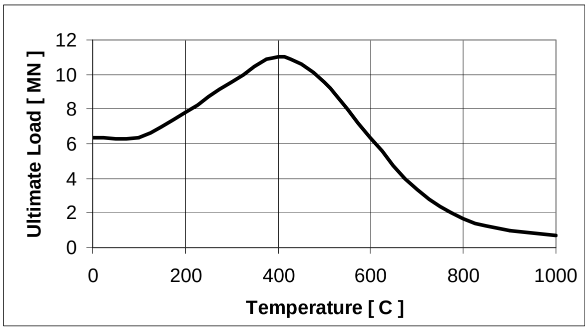

Figure 4. Ultimate capacity of the simplified beam.

This simple "linear" approach shows that membrane effects represents a substantial reserve which should be utilized in connection with extreme load situations where requirement to deflections are less than under normal, service conditions.

2.2 *Advanced beam behavior*

In the above example, pure bending and pure membrane effects have been demonstrated under fire conditions. In a real case both membrane and bending effects take place at the same time limited by current plastic interaction of the cross section. With increasing temperatures, the yield surface shrinks. In the following, the example described in the previous section is used without the middle hinge inserted.

In Figure 5 the plot of midspan bending moment versus axial force at temperature 250°C shows that after pure bending, compression forces are introduced due to thermal expansion.

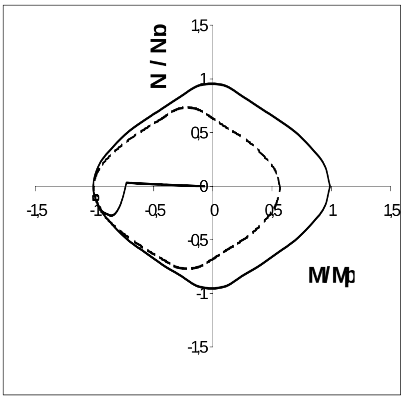

Figure 5. Yield surface and force path at 250° C.

Due to second order effects, the midspan bending moment increases with the increasing compression force. At same point the member buckles, and the axial force is being relieved. At this point the response is governed by the plastic interaction between axial force and bending moment. Later the response travels through the initial condition with pure bending, see Figure 6. However, by further increase of the deflection the axial force will turn from compression to tension.

The yield surface is reduced continuously. New equilibrium conditions must be established always limited by the current size of the yield surface. The mechanical load which is kept constant during the heating, is carried more and more by membrane effects. The force state at 600°C is shown in Figure 6.

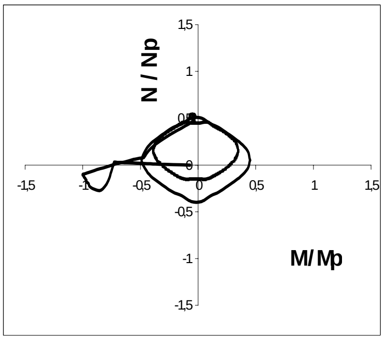

Figure 6. Yield surface and force path at 600° C
Both ends translation fixed

The beam now carries the lateral force by pure membrane action.

Without fully fixation in both ends, the beam will have a very different behavior. No compression forces will be introduced due to thermal expansion, and the beam will carry the load by bending throughout the whole heating process. Figure 7 describes the force state for the simply supported axially free beam at 600°C.

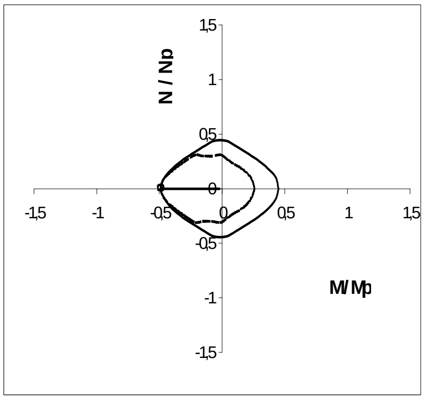

Figure 7. Yield surface and force path at 600° C.
No axial fixation.

Similar to the simplified case described in 2.1 the *ultimate capacity* of the beam as a function of the temperature with different boundary conditions are calculated utilizing the non-linear computer code *USFOS* /1/. The beams are first heated up to the actual temperature and then the mechanical load is applied. Each calculation terminates when the midspan deflection exceeds 0.75 m in this example. Stress induced strain as well as thermal expansion effects are included. The example demonstrates the much higher capacity of an axially fixed beam compared to one with free ends. *This documents the importance of designing the joints for the ultimate member forces including accidental loading rather than optimizing the connections for the actual forces in "cold" condition.*

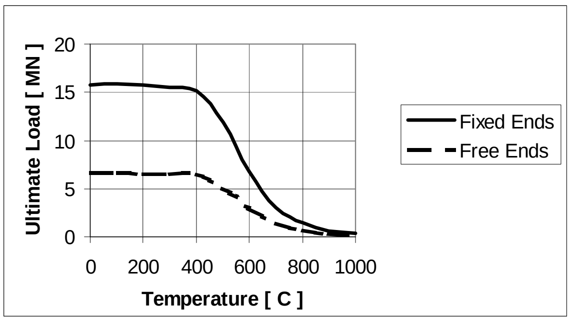

Figure 8. Ultimate beam capacity versus temperature.

2.3 *Column behavior*

Columns have a very different behavior under fire exposure than beams. No inherent reserves are available due to large deformation – rather the opposite is the case as any disturbance from straight configuration will reduce the ultimate capacity.

Several factors influence the ultimate capacity of the column:

- Reduction of yield stress
- Reduction of stiffness (E-modulus)
- Uneven exposure and associated uneven thermal expansion over the cross section causing column curvature
- Thermal expansion which may lead to column buckling depending on the column boundary conditions

Figure 9 describes the ultimate load as function of temperature of a steel column. The temperature is uniform over the cross section (no gradients), and following column data is used:

Outside diameter:	Do	=	355 mm
Thickness:	T	=	25 mm
Length:	L	=	6 m

The material properties are assumed to be degraded according to Eurocode 3, /4/.

The column is first heated up to the actual temperature, then the axial compression force is applied until the column buckles. The ultimate (peak) force level is recorded for each case.

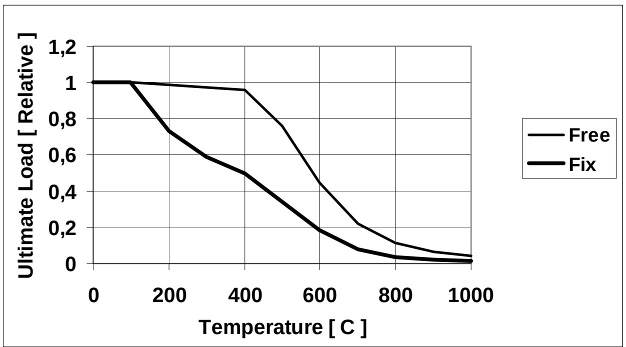

Figure 9. Ultimate load of free and fixed column

In the "free" case, the column is free to expand axially. In the "fixed" case the column upper node is free to move inwards, but is fixed in the outward direction. This means that thermal expansion may cause column buckling before the mechanical load is applied.

The ultimate load is given relative to the initial "cold" situation.

In the "free" case the buckling load is little influenced by temperatures up to 400°C as the ultimate load is primarily governed by the material yield stress. A slight decrease caused by the E-modulus degradation is observed for this particular column.

The "fixed" case the ultimate capacity is influenced by the temperature already from slightly above 100°C. This is mainly caused by thermal expansion forcing the column into a bent configuration. This out-of –straightness reduces the ultimate capacity significantly when the column is subsequently loaded in compression.

The two curves represent the limits for "real life" column behavior; in practice the capacity curve will be somewhere between the two extremes

3 BEHAVIOR OF PLANE FRAMES

The next case to be studied is a portal frame braced with an X-trusswork, Figure 10 and a K-trusswork Figure 11. The frame is loaded with a horizontal force, which is primarily carried by tension and compression in the braces. The X-truss represents an indeterminate structure in the sense that once the compression brace fails, it load can be shed to the tension braces. The K-truss is a determinate structure. Equilibrium requires that the compression brace carries the same force as the tension brace. Hence, failure of the compression brace signifies global failure of the frame.

The frames are first subjected to uniform heating followed by application of the mechanical load. The ultimate strength is recorded for all cases and normalised against the capacity at normal temperature.

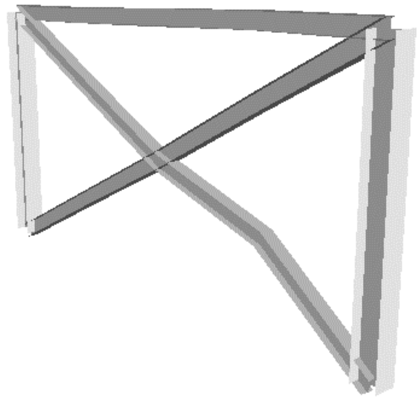

Figure 10. Horizontal loaded X-Truss exposed to fire.

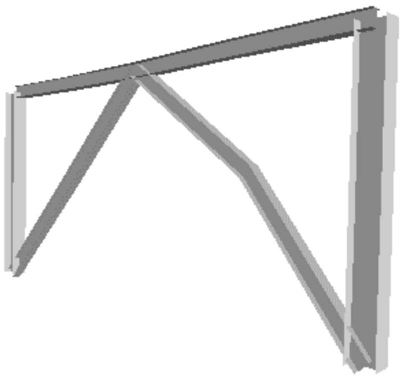

Figure 11. Horizontal loaded K-Truss exposed to fire.

During heating very small thermal strains are induced in the K-brace analogous to the axially free case of Section 2.3. Failure of the compression brace is again predominantly governed by the reduction in the yield stress and elastic modulus, but a further reduction in the buckling strength is also attributed to a frame induced lateral deformation.

Conversely, due to the static indeterminacy significant thermal strains are induced in the X-truss-work, closer to the axially fixed case in Section 2.3 The compression brace fails early in the heating process, but this does not of, course, signify ultimate strength. Global failure occurs when the tension brace reaches yield.

Figure 12 displays the normalised capacities for the frames. It is seen that both curves lies within the domain spanned by the axially free and axially fixed cases plotted in Figure 9. The X-truss suffers the largest *relative* reduction of ultimate capacity. The *absolute* strength is nevertheless larger.

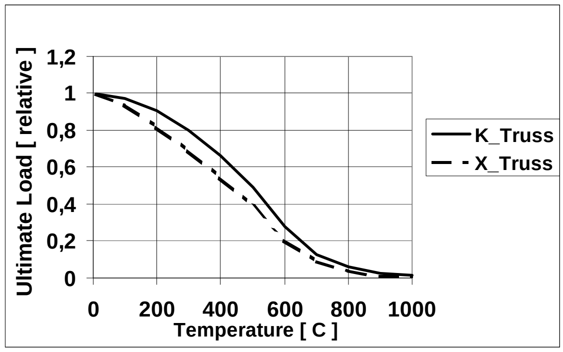

Figure 12. X-Truss versus K-truss behavior during fire.

4 OFFSHORE STRUCTURE EXPOSED TO FIRE

4.1 *Background*

The steel frame bridge composed of tubular members shown in Figure 13 connects two offshore oil platforms in the Northern Sea.

The bridge has two main functions: Supporting hydrocarbon pipelines and human traffic. In connection with fire the bridge is a part of the escape routes which must remain intact for a given time. In addition it is of great importance that the pipelines do not break and then cause escalation of the fire.

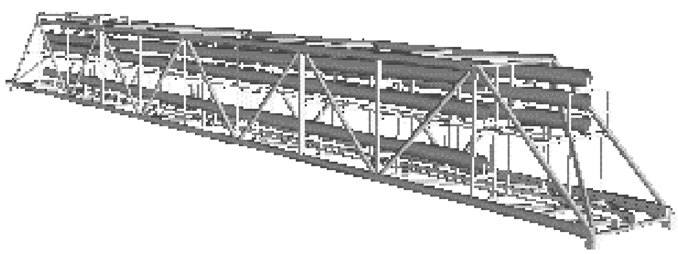

Figure 13. Bridge connecting two offshore oil platforms supporting hydrocarbon pipelines

From the safety studies the actual heat flux (or gas temperature) associated with the assumed “fire-on-sea” scenario is found. The bridge is not located in the center of the fire and is assumed to mainly be exposed to smoke gases, (~40 kW/m^2) It is required that the bridge should withstand the fire with substantial margin at least for 1 hour.

4.2 *Conventional fire design*

In a conventional design procedure, the structural department optimizes the steel based on the mechanical loads only. The structural response due to member heating is disregarded except for checking of necessary clearances in connection with thermal expansion.

The structural engineers presuppose that the member temperatures do not exceed a given temperature, 400°C is a widely used temperature threshold. At this temperature the steel maintain most of the initial (cold) properties.

The "responsibility" of the structure is then transferred to the safety department.

The safety engineers will normally use *passive fire protection* (PFP) to protect the structure. The thermal insulation will slow down the heating of the structural members avoiding the members to reach f ex 400°C within 1 hour. Correct thickness of the selected PFP product is found by utilizing the surface to mass ratio (*HP/A ratio*) which means in practice that the smaller members (easiest to heat up) get the largest thickness of PFP and vice verse.

In this particular example approx. 1500 m^2 surface was to be protected. With a minimum thickness applied of a typical spray-on product this results in approx. 8 ton passive fire protection.

4.3 *Advanced Fire Design*

Application of passive fire protection on exterior surfaces which are exposed to rough Northern Sea climate represents a maintenance challenge. Covering the whole surface with f ex a spray-on product makes it hard to inspect welds etc for fatigue cracks. Corrosion on the steel surface may cause the PFP to loosen and fall off.

Avoiding use of passive fire protection is then of great interest.

Assuming the heat fluxes from the safety studies to be design requirements, the computer code *FAHTS* (*Fire And Heat Transfer Simulations*) /2/ is used to simulate the individual member temperature histories. The design finite element model of the bridge is automatically transferred to surface shell elements by the code in order to capture the thermal effects over the cross section.

Simulations of the unprotected structure result in member temperatures up to approx. 650°C. At this temperature the steel has "lost" more than 50% of the initial strength. The temperature history for each individual structural member are transferred automatically to the mechanical response module *USFOS*. Both FAHTS and USFOS have been verified against large scale testing of a fire exposed 3D tubular frame /3/.

In connection with the USFOS simulations the following analysis procedure is used:

- Apply deadweight (loadfactor=1.0)
- Apply member heating (results from FAHTS)
- Increase deadweight up to system collapse

Figure 14 shows the collapse mode of the original design configuration. The compression members in the upper girders at midspan buckle with a load factor of the permanent loads equal to 1. This is an unacceptably small margin.

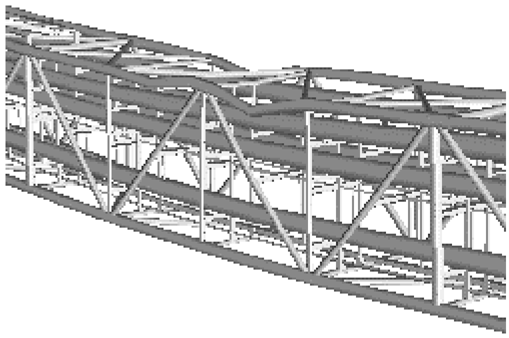

Figure 14. Collapse mode due to fire. Original design

The first modification is to increase the wall thickness of the upper girder tubular members. The idea is to prevent member buckling.

New simulations are carried out with the modified structure. However, now it is observed that member buckling takes place at the supports, see Figure 15. System failure corresponds to a load factor =1.05.

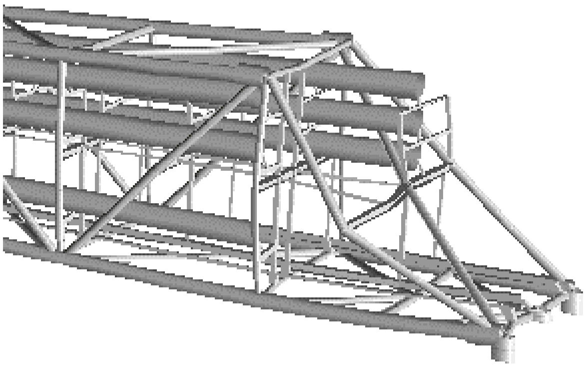

Figure 15. Collapse mode due to fire and with 5% overload. First modification.

Increasing the wall thickness of the most exposed diagonal members gives a substantial increase of the collapse load. In Figure 16 the midspan deflection versus load factor is presented for the three cases. Linear behavior is observed for all 3 cases up to a load factor of 1.0 causing a midspan deflection of approx. 0.12 m. The fire is then applied causing the deformation to increase to approx. 0.5m mainly due to reduced E-modus. Further increase of the mechanical load results in a very early system collapse for the two first cases. For the third case (final design), the system does not collapse before a load factor of 1.6 is reached. A rather 'linear' path is observed up to the peak, and the slope is approx. 1/3 of the initial slope which corresponds well with the fact that the E-modulus is reduced to approx. 1/3.

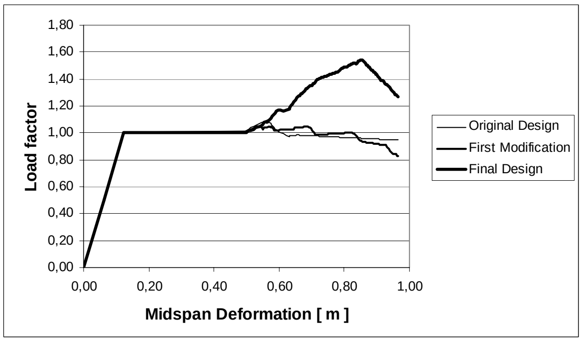

Figure 16. Midspan deflection versus load factor for three design alternatives.

The bridge situation at the peak load level of 1.6 is presented in Figure 17 with unscaled displacements.

The increase of the steel weight caused by the increased wall thickness of some compression members represents 1% of the total permanent load. The saved weight by using no passive fire protection represent the same mass which means that *the total weight of the bridge is unchanged*. The fabrication and maintenance *costs* are, however, *substantially lower*.

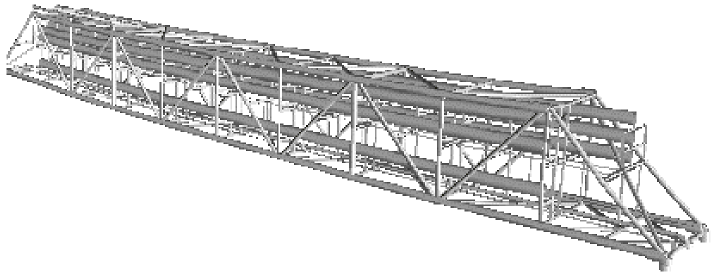

Figure 17. Final Design. Situation after 1 hour fire and 60% overload.

5 CONLUSIONS

The importance of including the accidental fire in the design process is documented.

Advanced computer codes which have been verified against large 3D tests simulating structural behavior during fire might be an efficient and reliable tool for the structural engineer.

Increased knowledge about structural behavior under fire opens for more fire resistant structures, reduced fabrication and maintenance costs and extended use of light metal structures.

6 REFERENCES

/1/ USFOS. Ultimate Strength of Frame Offshore Structures. User's Manual, SINTEF Report STF71 F88039.

/2/ FAHTS Fire And Heat Transfer Simulations User's Manual, SINTEF Report 1995.

/3/ Laboratory Test of a 3D Steel Frame Exposed to Fire. SINTEF Report, 1995.

/4/ Eurocode 3: Design of Steel Structures Part 1.2: Structural Fire Design