COMPUTER METHODS FOR OPTIMISING PASSIVE FIRE PROTECTION ON OFFSHORE PLATFORMS

Dr Yngve Anderberg, Fire Safety Design, Lund, Sweden

1. INTRODUCTION

Analytical procedures of optimising passive fire protection have been developed by Fire Safety Design (FSD) and used in North Sea and Atlantic drilling platforms since 1987. Important input for the optimisation process is provided by risk analysis and by defining realistic heat loads and fire durations of jet and pool fires for the modules.

The knowledge of thermal properties as function of temperature is necessary in the analytical design of passive fire protection. The thermal properties of the insulation materials used, are assessed by computer simulations of fire tests, which is a comprehensive task.

The loads and its partial factors (given in the national codes) in fire is the basis for the assessment of load utilisation of each member. By using the "correct" design strength-temperature curve for structural steel the critical temperature and/or the load-bearing capacity (assessed by for instance the computer program ***Fire Design***[2]) can be established.

When the critical temperature is found a steel member shall be insulated in an optimised way by using a computer program as for instance ***Super-Tempcalc***[1], which is approved by authorities in Canada and in a number of European countries. Optimisation has been performed for Chartek III, Thermo-Lag 440, Mandolite 550, Rockwool etc.

Thermal restraint forces and moments and the redistribution in the structure must also be analysed in order to prevent local collapse and risk for progressive collapse. A computer program ***Steel Fire Design*** dedicated for that purpose is also developed.

Platform collapse and overall safety margin structural analysis is performed by the computer software "***Global Collapse in Offshore Fires***" and it is also possible to compare the critical steel temperature designed for and the collapse steel temperature assessed.

Frequently, 30-40% less insulation is required when fire proofing is optimised by computer analysis. The Global Collapse Analysis allows for a further reduction in weight to be achieved, while simultaneously maintaining adequate structural safety margins.

2. FIRE TEST PROCEDURES AND CERTIFICATES BASED ON H_P/A

For a long period of time the manufacturers have been forced to perform pool and jet fire tests in several countries for exactly the same material to obtain an approval in each country. One country doesn't normally accept results from another country. This procedure of repeated fire testing is an enormous waste of money, time consuming and sometimes planning delays. Still different furnaces give different results on identical tests and no harmonisation between furnaces does exist. Vendors and manufacturers greatly suffer from this inadequate situation.

The manufacturers must perform comprehensive pool fire testing at the heat load of 200 kW/m^2 (HC-fires) and various endurancies for open (i.e. I-sections) as well as closed (box-sections) steel profiles and for steel deckplates at varying steel and insulation thicknesses. Jet fire testing is requested on profiles, deckplates and bulkheads at the heat load of 300 kW/m^2. This means a vast number of fire tests to be performed to obtain approval for one single insulation material. Many of these tests can be replaced by computer simulations.

The conventional form factor method H_p/A (H_p is perimeter of the section exposed to fire and A the cross-sectional area) was originally developed and used on onshore projects. It is based on the theory of lumped mass and assumes an uniformly distributed steel temperature over the cross-section at any point in time and that the heat flow is unidimensional. These assumptions are valid for thin steel structures in the building industry with normal size cross-sections but not for large offshore steel members with thick flanges and webs. The method is especially not valid for insulation materials with a high thermal capacity, which is typical for intumescent materials as well as for concrete based materials, mentioned in the introdution. Furthermore the swelling of intumescent makes the exposed area different from the perimeter of the steel profile as illustrated in Fig 1 and the form factor is radically changed. Despite all these limitations, the form factor method is generally used for fire proofing on large offshore structures.

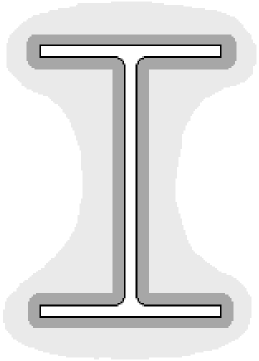


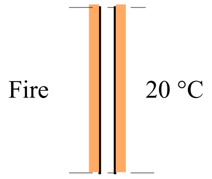
Fig 1 Insulated I-profile before and during fire due to swelling of intumescent material

Based on all these fire tests described above certificates are released by i.e. Lloyds Register, DNV etc for profiles (defined by the form factor), deckplates and bulkheads. These give tables for insulation thicknesses required for various steel core temperatures at the heat load 200 kW/m^2 (pool fires) or 300 kW/m^2 (jet fires). All the certificates are based on fire tests performed on small profiles, but extrapolated for use on large profiles without considering the limitations. However, it is not realistic to perform fire tests to cover all situations including different heat loads (50-300 kW/m^2) and especially not to make full scale tests. Rules of thumb methods are used for coat back insulation in heat transfer zones which can be predicted by computer simulations. It is not possible to continue in the same track in the future but it will be a necessity to follow new procedures and include an analytical approach as indicated in this paper.

3. COMBINATION OF FIRE TESTS AND COMPUTER SIMULATIONS

To meet the insufficiences from small scale testing and the limited use of certificates an analytical thermal modelling is necessary. A systematic planning of a proper number of fire tests combined with computer simulations is the superior approach to extend the applicability and open the possibility to develop a general design guide. Such a design guide can contain optimised design rules for large as well as for small members and the former extrapolation from small scale tests and from the heat load 200 kW/m^2 will no longer cause any problem. This procedure will generate a general guidance of how to use the material in an optimised way and saving a lot of money. Also bulkheads, deckplates and fire walls, fire exposed on one side (see Fig 2) can be solved in an general way.

Fig 2 Separating and/or load-bearing structure, fire exposed on one side but insulated on both sides.



A fire test programme needs to be defined in such a way that the results will provide a sufficient basis for reliable computer simulations of the thermal response of any insulated cross-section, profile, deckplate, bulkhead or influence from a heat transfer zone (i.e uninsulated secondary member welded to a primary member, see Fig 3, at any heat load (pool fire or jet fire) relevant for an offshore platform). Earlier, most fire tests were performed on profiles but the emphasis must now be on steel plates. The specimens can be manufactured quickly and easily and the tests can be performed in small furnaces to heavily reduced costs compared with fire tests on profiles.

4.. THERMAL ANALYSIS

4.1 Assessment of thermal properties

The thermal properties must be known in the thermal analysis and computer simulation of fire tests is the most appropriate procedure to assess these.

A simulation of well-defined fire tests on 20-25 steel plates must be performed to assess the ”true” thermal properties of insulation materials so that edge and corner effects typical for small profiles can be avoided. The thermal properties for intumescent materials (i.e. Chartek IV, Thermo-Lag 440 and Pittchar XT), will vary with the thickness of the plate and the heat load level as well as with the thickness of the insulation. Large profiles (height h ≥500 mm) can be considered as plates, where the flanges and the web have different thicknesses. A very special tool like Super-Tempcalc devoted to this type of problems will generate after a great number of iterations, a set of curves for the thermal conductivity applicable to various combinations of heat load, steel and insulation thicknesses. The thermal capacity is normally only a function of temperature.

4.2 Integrity distances in heat transfer zones

Secondary steel members on an offshore platform are frequently welded into insulated load-bearing structures, beams, columns, bulkheads etc... The secondary members need to be fire-protected for a certain distance x from the interface with the load-bearing member, which is illustrated in Fig 3. This will reduce the amount of heat transferred from the secondary member. According to the current very conservative rules of thumb methods, the length of insulation required are 450 and 750 mm. Is is however not only the coat back distance x, which is important but also the insulation thickness t_i on the secondary member.

The amount of passive fire protection in heat transfer zones has been optimised by FSD for the platforms Troll Gas, Troll Oil and Hibernia by Super-Tempcalc, 2- and 3-dimensional calculations. The analysis showed that the coat back distance can be reduced to 150-300 mm depending on the heat load level, fire duration, the critical steel core temperature and the size and configuration of the secondary member. The result is valid under the assumption that the thickness t_i of the coat back insulation for the secondary cross-section is calculated to keep the temperature equal to or below the critical one.

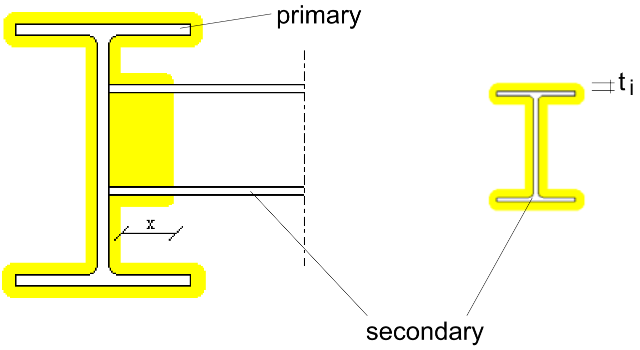


Fig 3 Principal picture of the coat back distance x for uninsulated secondary profile

4.3 Assessment of edge and corner effects of small and large profiles

It has been observed from fire tests on small profiles that a small heat leakage takes place especially at the corners of closed rectangular sections, probably due to corner-cracking but it is also noticed on flanges of I-sections. This phenomenon has been modelled by Super-Tempcalc for small as well as for large sections.

For small I-sections the swelling of intumescent is very advantegeous for the web (see Fig 1) and together with the edge effects of the flanges, the temperature of the flanges sometimes are higher than that of the web. Without these effects the temperature of the web should be considerably higher and this is always the case for large profiles. The insulation required on small I-sections can be assessed by using ”effective” thermal properties derived from fire tests.

The influence of a corner crack for two small box girders versus two large box girders (200 · 200 · 4.8, 1500 · 1200 · 10 and 1500 · 1200 · 10, 1500 · 1200 · 40) taken from a real platform will be illustrated below. The thickness of 10 mm is chosen in order to make a direct comparison between the small and the large profile with the same steel thickness and H_p/A value. The profiles are insulated to reach the maximum steel core temperature of about 450 °C after 1 hour at the heat load of 200 kW/m^2 without any heat leakage at the corners.

The results of the small profile (2) and the large box profile (3) are illustrated in Fig 4 and indicate that the temperature increase due to heat leakage is 130 - 150 °C for the small profile but only 25-60 °C for the large offshore profile presuming they have the same H_p/A factor.

The other two profiles, a small one (profile 1) and a large one (profile 4) have been analysed and these results together with profile 2 and 3 are presented in Table 1 as average temperature without any corner cracks and the maximum and minimum temperatures due to leakage effects.

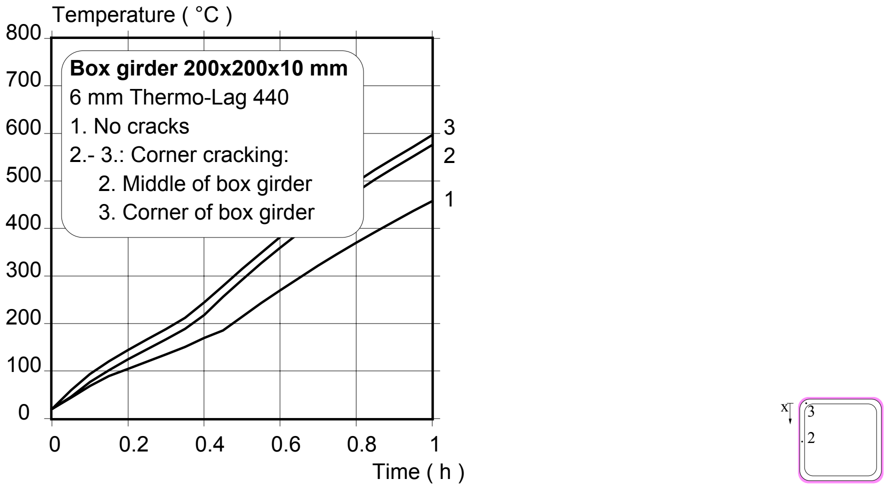
The tempeature increase due to corner cracks for the thinner profile (No 1) with the steel thickness of 4.8 mm is as much as 245-265 °C but only 25-50 °C for the thicker profile (4) with the steel thickness of 40 mm. It can also be noticed that the temperature difference over all the profiles (1 and 2) is only about 20-35 °C after 1 hour.

Box Girder		No cracks	Corner cracks	
Size of profile mm	Insulation in mm	Average temp °C	Max temp °C	Min temp °C
1. 200·200·4.8	11	465	730	710
2. 200·200·10.0	6	450	600	580
3. 500·1200·10.0	6	450	510	475
4. 1500·1200·40.0	1.7	450	500	475

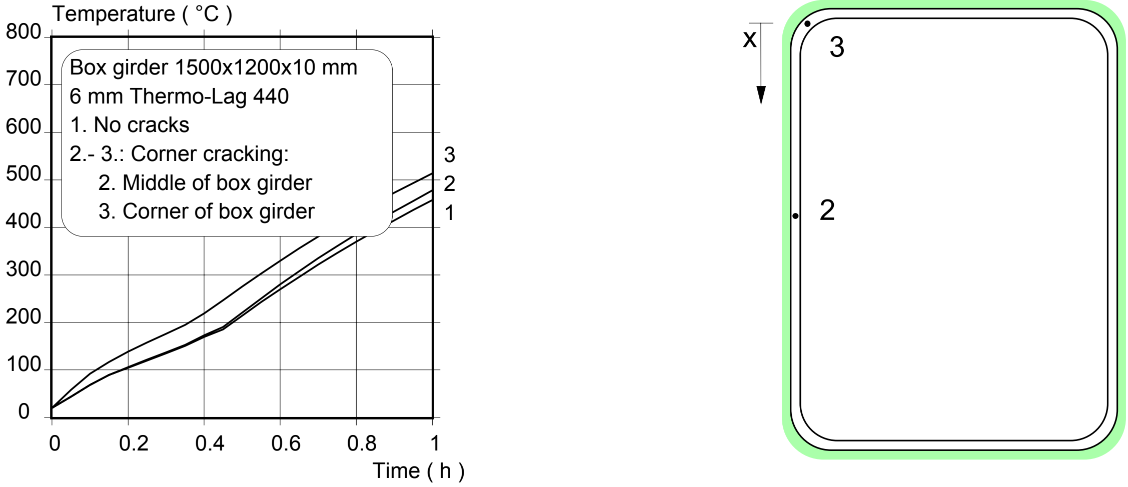
Table 1 Average temperature if no cracks occur and the maximum and average temperatures for small and large profiles due to a heat leakage simulated at the corners.

The computer analysis, which is checked against tests, reveal that the influence of a heat leakage in the corner of a large offshore profile is almost negligible compared with a small box-girder and consequently results from small profiles cannot be used on large offshore profiles.

Using data from tests on small box girders consequently provides very conservative and unrealistic insulation on large box girders. Differences of more than 200 °C are assessed.



a) Small box girder 2



b) Large box-girder 3

Fig 4 Predicted temperature - time curves for Box-girders at two different positions with and without a heat leakage at the corner

4.4 Assessment of erosion effect due to jet fires

FSD has experience of modelling the erosion effect due to jet fires as function of time by Super-Tempcalc. The modelling is based on computer simulations of jet fire tests on profiles and bulkheads. By an iterative procedure the mechanical erosion as function of time is assessed.

4.5 Temperature analysis for special cases

Optimisation of passive fire protection cannot solely be solved by design tables on insulation thickness required, when based on a critical temperature. Many special cases exist where complementary temperature analyses as well as load-bearing calculations are necessary. Two different cases are indicated in Fig 5. A temperature analysis of the two cases gives the temperature gradient as function of time and that must be followed by a load-bearing calculation (i.e by Fire Design). Such a calculation is examplified in chapter 6 for the case of fire exposure on the top flange only.

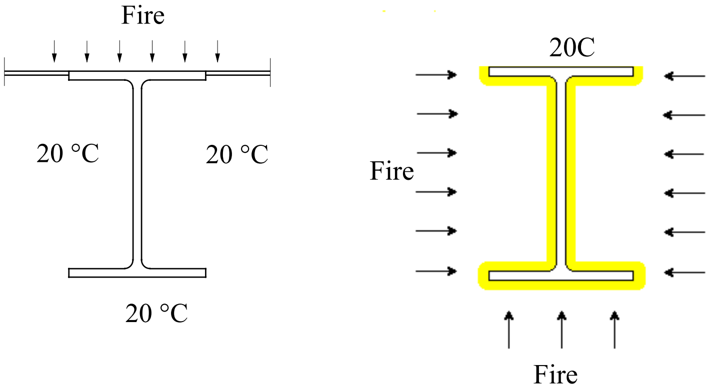


Fig 5 Beams partly fire-exposed

5. LOAD UTILISATION AND CRITICAL STEEL TEMPERATURE

Fire is considered as an accidental load and the fire-related partial coefficients are different from those at normal design and they are therefore necessary to derive. The general reduction factor at a fire engineering design is about 0.70-0.75 which means that the load-bearing capacity is allowed to decrease to 70-75% of its capacity at ambient conditions without any precautions needed to be taken.

Normal design for steel structures is based on the yield stress or 0.2% strain limit but at high temperatures there is no distinct yield plateau in the stress strain relationship. The strength based on 0.2% limit is too conservative in a fire engineering design. A more realistic strain limit for steel 0.75% is suggested, which however for beams still is somewhat conservative when compared with the recommendations in the Eurocode 3, Part 1.2[4]. The relative strength-temperature curve based on the the 0.75% strain is shown in Fig 6. If the general reduction factor is 0.75 the maximum (critical) temperature at a factored degree of loading of 100 % is 470 °C as indicated in Fig 6. The critical temperature as function of load utilisation can be derived from Fig 6. Table 2 is illustrating the result of this relationship.

When a structure has a temperature gradient no critical temperature exist and consequently the load-bearing capacity must be assessed and compared with the external load effect.

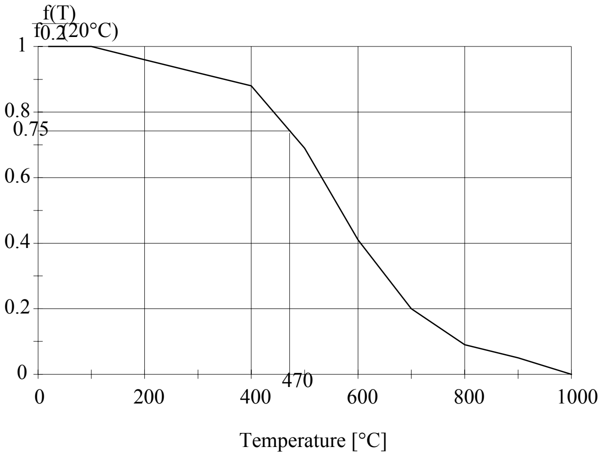


Fig 6 Relative strength-temperature relationship related to Eurocode 3, Part 1.2

Utilisation factor in normal design %	Critical steel temperature °C
100	470
80	530
60	580
40	650
20	750

Table 2 Critical temperature as function of utilisation factor

6. LOADBEARING CAPACITY FOR MEMBERS WITH A TEMPERATURE GRADIENT.

In many cases the fire exposure is not uniform around a structural member and therefore a steep temperature gradient will arise and consequently no critical temperature exist. This means that a the structural consequence must be analysed. As an example the structural behaviour is briefly presented for an uninsulated underdeck girder in case of fire during two hours on main deck (see Fig 7).

In case of fire on main deck the deck will presumably be covered by fluid. The fluid will reduce the heat load most significantly, which means that the presented results are very conservative. The temperature profile of the I-beam after 1 hour pool fire exposure (150 kW/m^2) is illustrated in Fig 7 b and the temperature at a position of about 350 mm from the top flange is only 100 °C, but the top flange temperature is more than 1000 °C. The temperature does stabilise after 1 hour exposure, which can be observed from Fig 7 c illustrating the relative load-bearing capacity of 50 % stabiliesed after 1 hour. The cooling effect on unexposed surfaces makes the thermal gradient constant after 1 hour. The minimum relative moment capacity of 50% means that a load utilisation degree of about 67 % can be acceptable for this girder.

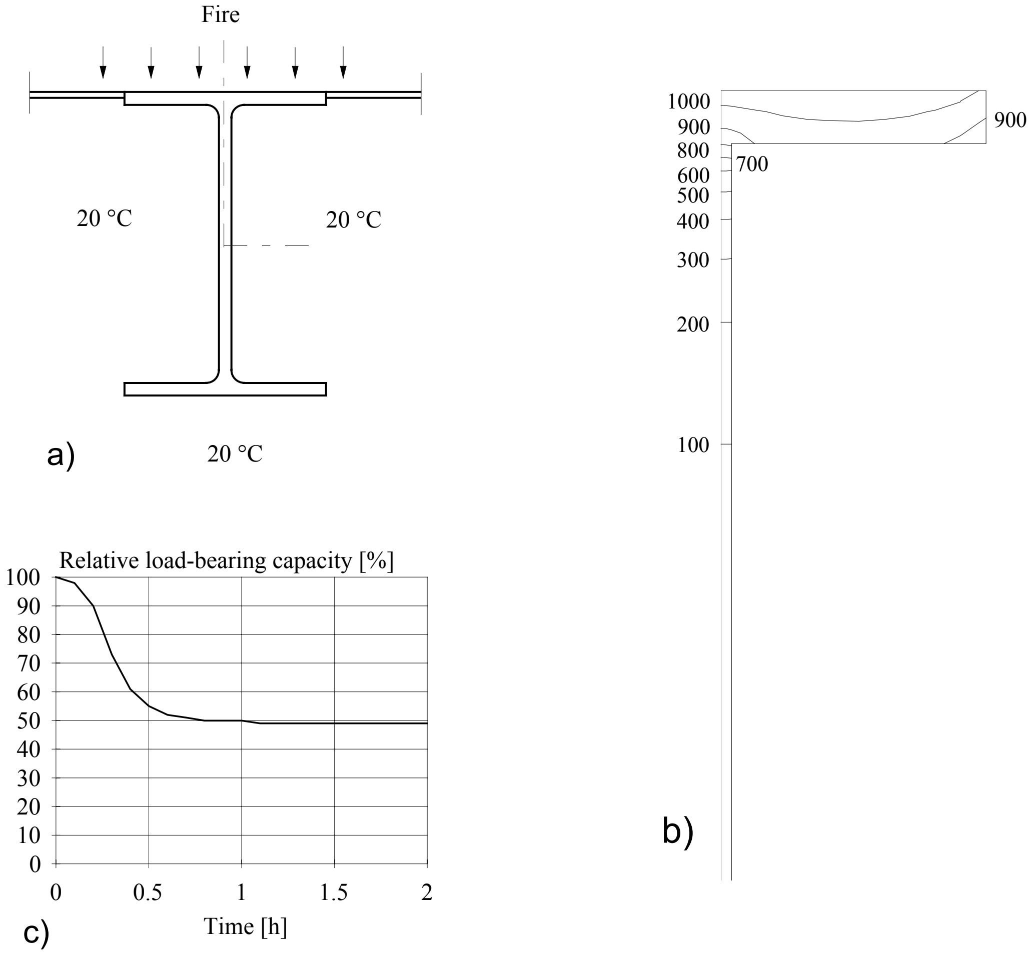


Fig 7 Relative load-bearing capacity for underdeck girder exposed to heat only on the top flange and unexposed on other surfaces

a) I 1500 x 500 x 20 x 50 exposed at the heat load 150 kW/m^2 for 2 hours
b) Isotherms of the I-beam after 1 h fire exposure.
c) Relative moment capacity as function of time.

7. GLOBAL BEHAVIOUR OF THE PLATFORM IN CASE OF FIRE

The design of the platform is performed in such a way that a fire will be prevented to spread within a module horizontally and from one level to another for a certain time (1 to 2 hours) by separating deck plates and bulkheads. The fire spread from module to module is furthermore stopped by fire rated bulkheads. This means that all primary members supporting the deck plates are not allowed to collapse during the design fire because of the fulfilment of the separating function, even if they can be regarded as redundant members from a collapse point of view.

7.1 Load Re-Distribution Potential

At fire in a limited space of a module only a part of the structure is fire-exposed. This means that those members which are connected and unexposed may re-distribute the load and increase the fire resistance time and the maximum steel core temperature considerably. When a fire starts in a module the exposed structures tend to expand and considerable thermal forces and moments arise, which will be transferred to those structures unexposed. Large axial forces on columns and/or horisontal thermal elongations may result in local collapse of that column or as a worst case ultimately progressive collapse. However, computer simulations made by **Steel Fire Design** indicate that a considerable re-distribution of loads can be expected and those structures heavily exposed will get assistance to carry the load from unexposed members. There is obviously a considerable load re-distribution potential due to the high degree of statically indetermination. The reserve load-bearing capacity is important and gives an extra safety margin to collapse.

7.2 Consequence and Types of Local Failures

It is important to clarify the status of each member as concerns its influence on the stability of the whole platform. If a collapse of a member only causes a local failure in the structure and does not jeopardise the overall stability this member is considered as a redundant member and hence its existance is not necessary from a stability point of view. If however, the separating function can be lost, the member is not considered as redundant.

7.3 Global Collapse And Safety Margin Analysis

The computer software ”**Global Collapse in Offshore Fires**” is recently developed and at this first stage consists of a 2-dimensional FEM-program on PC integrated with Super-Tempcalc. The programme is provided with an intelligant and userfriendly interface for efficiant collapse analysis. It is also easy to make sensitivity studies of the influence of different parameters. Tentative calulations on the Hibernia platform has already been performed. The main purpose of the programme is the assessment of the collapse steel core temperature and the safety margin to progressive collapse module by module as well as for the whole platform. In a near future authorities and insurance companies will require platform safety margin analysis in case of fire.

9. CONCLUSION

- Extrapolation of fire test results on small profiles cannot be used on large offshore profiles. A combination of fire tests and analytical thermal modelling is the only solution.

- A proper assessment of thermal properties and computer simulations provide a basis for a general design guide to be developed.

- Coat back insulation can only be optimised by computer calculations.

- When temperature gradients arise in a structure no critical temperature exist and the load-bearing capacity must be computed and compared with the load effect.

- Load re-distribution potential and global collapse analysis in case of fire can be expected to be required in a near future.

REFERENCES

1. **Yngve Anderberg:** SUPER-TEMPCALC, A Commercial And Userfriendly Computer Program With Automatic Fem-Generation For Temperature Analysis Of Structures Exposed To Heat. Fire Safety Design AB, Lund 1991

2. **Yngve Anderberg:** Fire Design, Computer program, integrated with SUPER-TEMPCALC, for analysing the load-bearing capacity of thermally exposed concrete and steel beams and columns, Fire Safety Design AB Lund 1991

3. **Yngve Anderberg:** Optimizing Fire Protection On North Sea Platforms. Fire Safety Design AB, Lund, C438/001 IMECHE 1992.

4. **Eurocode 3:**: Design of Steel Structures, Part 1.2 Structural Fire Design. Draft prENV 1993-1-2, August 1993