uses and, as modelling techniques and programs develop, will become increasingly useful. However, the question we would like to raise is whether, in the light of our comments above, it is a technique that can be justified for determining structural response to an extreme eventsuch as an explosion? By admission it is an expensive technique (3 to 5 runs is regarded as large!). The scope for error is high. A practical model requires compromises. Only one or two events can be analysed. Response is heavily influenced by loading, joint stiffness, mass and failure mechanisms, each of which contains a significant level of uncertainty. These factors highlight the importance of the 100% design check to ensure that the correct mechanisms are being analysed. Nevertheless, can we be certain that the use of dynamic NLFEA gives us any more confidence than, for example, the intelligent use of SDOF coupled with linear static and non-linear static analysis? If not, should we use dynamic NLFEA?

This reply to article R137 was written by Mr Hugh Bowerman, FABIG Project Manager. That article and this reply pose many questions. Hopefully the article by Dr Czucko in the next newsletter will address some of these questions. However we would welcome any additional comments or answers to these questions. These can be sent to:

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R139

A RATIONAL APPROACH TO FIRE CONSEQUENCE ASSESSMENT

Introduction

Structural fire response calculations are performed in a large number of different ways by different organisations. The complexity of the approach used appears sometimes not to be based on any rational appraisal of the assumptions that can sensibly be made, leading to unnecessary design expenditure with no additional confidence in the results. This Article shows the assumptions which may be made in virtually all circumstances and indicates the appropriate level of analysis.

The governing partial differential equation for thermal response calculations is the three dimensional diffusion equation:

$$\frac{\partial T}{\partial t} = \alpha \nabla^2 T \qquad (1)$$

where:

- T is the temperature at point (x,y,z) and time t;
- α is the thermal diffusivity of the material define by $\alpha = k/\rho C$;
- k is the coefficient of thermal conductivity of the material;
- ρ is the density of the material;
- C is the specific heat of the material;

and:

∇^2 is the Laplacian operator which is defined by:

$$\nabla^2 \equiv \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} + \frac{\partial^2}{\partial z^2}$$

in rectangular cartesian coordinates.

This equation applies for each volume of material, whether steel or insulator, and will have to satisfy boundary conditions involving heat fluxes and prescribed temperature boundaries. The parameters k and C will themselves be temperature dependent. When solved, this equation will give the temperature variation with space and time and in combination with similar partial differential equations describing thermal expansion and elastic deformation, will yield stress fields in the material given the restraint conditions.

Clearly, it would be ridiculous to solve the problem in this way.

Time Dependence

The temperature of an unprotected steel structural element may realistically be assumed to have reached the steady state during any time interval. Changes in temperature immediately reflect changes in thermal loading.

A good measure of the time of response of a conductor is the thermal penetration time[2] t_p defined by:

$$t_p = \delta^2/4\alpha \tag{2}$$

where:

δ is the thickness of the material in metres, the panel or section thickness

For steel cladding of thickness 5 - 10 mm, t_p is less than 1 second. For a typical unprotected engulfed steel beam section, t_p will be similar and depend on the section thicknesses. This is confirmed in Reference 3 where it is stated that the time step for simulations based on Equation (1) must be less than $\delta^2/2\alpha$ for one dimensional heat flow.

The penetration time for an insulator will naturally be much longer as α will be much smaller. The α value for Mandolite is 30 times smaller than that for steel making the penetration time 30 times longer. Fire protected members will behave in this way.

Conduction

Conduction between structural members may be ignored.

The steady state assumption established above enables Equation (1) to be reduced to the one dimensional heat conduction Equation (1):

$$q = -k \frac{dT}{dx} = k(T_2 - T_1)/\delta \tag{3}$$

where:

q is the heat flux conducted per square metre of the section (W/m^2)

T_2 is the temperature of the hot face

T_1 is the temperature of the cold face

For a steel UB 457 × 191 × 82, for example, the heat conducted along one metre length of the member with a temperature difference of 500°C would be only 0.23kW; this compares with typical incident fluxes of 100 to 150kW/m^2 present in an engulfing pool fire. Furthermore, the thermal capacity of this beam per metre length is of the order 40kJ per degree which would indicate that a one metre length of the beam would take about 5 hours to heat up by 100 degrees if conduction is the only heat source.

This result is confirmed by the limited length of fire protective coating required to protect against conduction along a beam penetrating a fire barrier.

Typically, the extent of a fire results in radiative and convective thermal loading along the total beam length. In these circumstances the beam may be considered to be at a uniform temperature through thickness and along its length. The critical section determining the structural performance of the member will be at mid-span or the end points. For variable thermal loading of a beam along its length the temperatures at these critical locations may usually be taken to characterise the capacity of the beam.

Thermal Expansion

Thermal expansion and the resulting external restraint loads may be neglected.

This assumption may break down locally. For example, the local thermal stresses due to internal restraint in a plate acted on by a jet fire may be important but the global response of a compartment will be adequately described using the above assumption. In all the structures examined by SLP, there is only one case where failure has been caused directly by thermal restraint, and this affected a single member.

The thermal stress in a fully axially restrained steel beam with infinitely stiff supports is given by:

$$\sigma_t = E\,\beta\,\delta T \tag{4}$$

where:

β is the coefficient of linear expansion of steel (14×10^{-6} per degree)

E is Young's modulus for steel (206 kN/mm^2)

and:

δT is the temperature change in degrees C.

The Young's modulus for steel itself reduces with temperature[1], resulting in a softening effect which reduces the resulting thermal stresses. Unfortunately the yield stress σ_s also decreases with temperature so, as a proportion of yield, the thermal stresses will fail the member in the fully restrained case.

Figure 139.1 shows the variation with temperature of the thermal stress in a fully restrained member. There is a reduction of stress above 400°C due to the reduction in Young's modulus. The yield stress also reduces at these high temperatures. The variation of the thermal stress as a proportion of the (0.5% proof) yield stress is shown in Figure 139.2. For a fully restrained member failure would occur at 150°C!

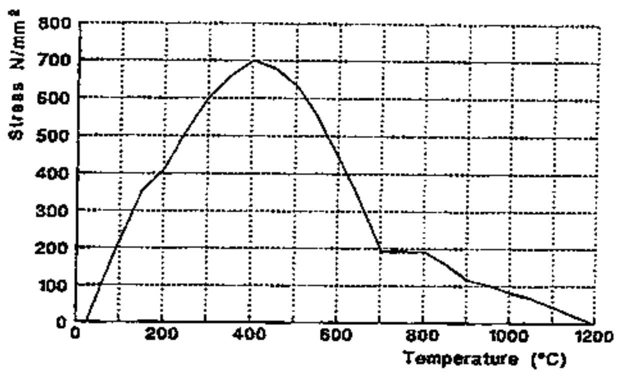

Figure 139.1
Thermal Stresses in Fully Restrained Members

Of course members are never fully restrained. Consider the situation shown in Figure 139.3. A stocky beam of type UB 457 × 191 × 82 is heavily restrained between two similar support beams. All spans are 3 m. Major axis bending is out of the plane of the paper for all beams. The two support beams are assumed built in at each end. These beams give a restraint stiffness of 12.8 kN/mm in the axial direction.

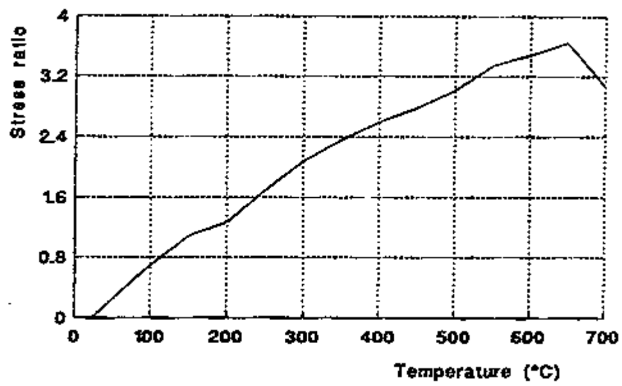

Figure 139.2
Ratio of Thermal Stresses 0.5% Proof Yield Stress Fully Restrained Member

Figure 139.4 shows the variation of the thermal stresses induced in the connecting beam with temperature. The upper curve assumes that only this beam is heated, the support beams are protected and no change of support stiffness occurs. The peak thermal stress in this case is about 30 N/mm² at 700°C. If the support beams are assumed to be unprotected and heated in the same way then the softening effect of the supports gives a peak thermal stress of about 11 N/mm². A geometry of this type is probably associated with a heavily loaded floor. The dead loads on the floor are likely to be such that the contribution from thermal stresses is negligible.

This is only one example. However, SLP's experience obtained from detailed consideration of a number of topsides structures is that thermal expansion and restraint forces are not critical. Globally, the major factor is the softening and weakening of the steel with increasing temperature.

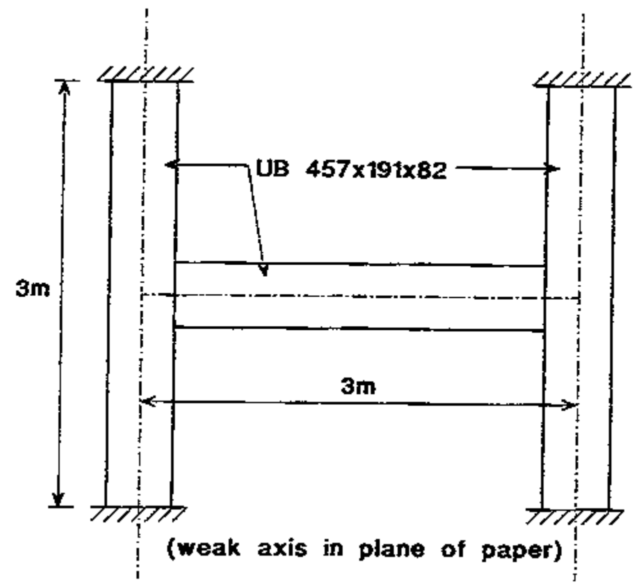

Figure 139.3
Test Example - Thermal Stresses

Local effects such as bowing of panels, columns and beams due to differential heating on one side due to internal restraint are important. These effects introduce secondary moments resulting from loads which were previously axial. This 'P-delta' effect may considerably reduce the strength of the structure.

The above treatment does not consider premature buckling.

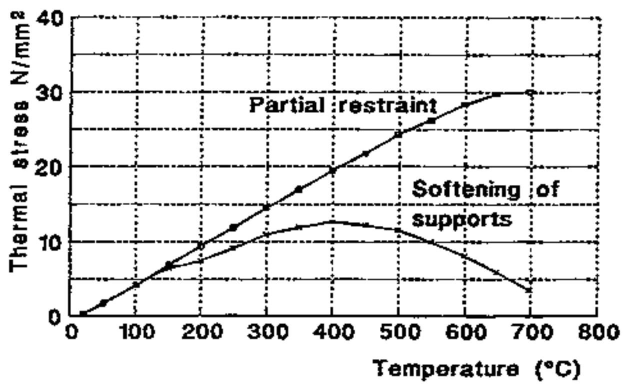

Figure 139.4
Test Case Configuration - Thermal Stresses

Conclusions

The SLP approach is to deal with the problem in the simplest way possible to give reliable results whilst being aware of any factors which may indicate the need for a more detailed approach in local regions. This approach also has the incidental consequence of producing understandable results quickly and at minimal cost.

Global frame fire response analyses may be performed based on the following general assumptions:

1. Each member is considered to have reached a steady state: variations of temperature are due entirely to changes in boundary conditions and incident heat fluxes.

2. Conduction between members does not occur.

3. Unprotected structural members and panels have no variation of temperature through thickness or along that part of their length exposed to thermal loading.

4. For fire protected structural members and panels, the thermal insulation has a linear variation of temperature through thickness.

5. Thermal stresses due to restraint may be neglected.

The first four assumptions are in fact implicit in the methods described in the Interim Guidance Notes[1].

Further details

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References

1. Bowerman, H. et al., 'Interim Guidance Notes for the Design and Protection of Topside Structures Against Explosion and Fire', SCI, November 1991.

2. 'Behaviour of Oil and Gas Fires in the Presence of Confinement and Obstacles', JIP Fire and Blast, WPFL2.

3. 'Methodologies and Available Tools for the Design/Analysis of Steel Components at Elevated Temperatures', JIP Fire and Blast, WPFR2.

R140

ENCAPSULATED FIRE PROTECTION OF COMPOSITE PIPES

Introduction

Composite pipes with their low weights and good corrosion resistance are of interest in offshore situations provided fire safety requirements can be satisfied. They are believed to be very cost effective in the larger pipe sizes. The viability of a composite pipe system is likely to be improved if the fire protection can be "built-in" at the production stage. The joints, however, may still require separate fire protection after the system has been installed.

One application of composite pipes which is being investigated is for the provision of firewater systems on the rigs. Typically there is a ring main at a low level filled with stagnant water from which vertical riser pipes will convey water to the sprinkler heads in the event of fire. These risers are normally dry. In the event of an emergency they may be exposed to fire for a short period of time whilst still dry. This dry exposure time in measured trials may be as short as 30 seconds, but a longer time might apply in a real fire situation. There is no fixed exposure limit for design purposes. Five minutes has been proposed but it is now thought to be rather long. Perhaps three minutes will provide an adequate safety margin. Once the pipes are full of flowing water they are likely to remain functional for extended periods of time[1].

Unprotected filament wound epoxy resin pipe available commercially has proved in simulated hydrocarbon furnace tests to have a limiting dry exposure time of approximately one minute, i.e. when subsequently water filled and subject to the design pressure it will not weep. This exposure time is too short and the fire resistance of the pipes needs to be improved.

Fire Resistance Improvement at the Production stage

The wall thickness of the pipe could be increased using the normal materials, e.g. glass fibre and epoxy resin in the case reported, to provide a sacrificial layer. This is likely to require a doubling of wall thickness to achieve the required effect. A second method is to provide suitable additives in the resin which will enhance the behaviour in fire conditions. Since the additives increase the viscosity of the resin, this leads to an increase in wall thickness for the same original glass and resin content. A third method is to encapsulate ceramic fibre in an outer wrapping of the pipe. This latter method is the subject of the present notes.