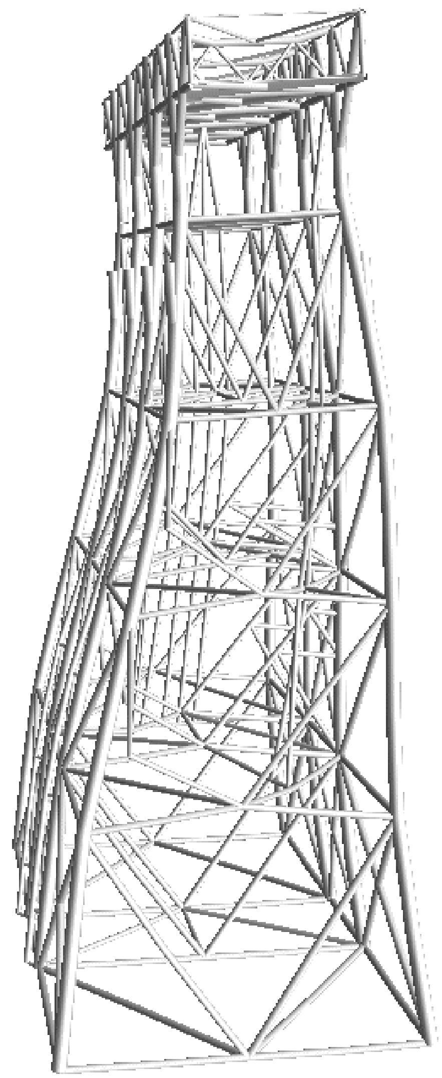
PUSHDOWN ANALYSIS

In

USFOS



Contents

1 Introduction

USFOS is used for integrity analysis of structures exposed to fire. This document describes some fundamental definitions needed for the fire analysis assessment.

The document describes also how to use the “pushdown” feature, which was released in version 8-7 of USFOS. The “pushdown” procedure is used to establish the “fire degradation” curves for the entire structure, which expresses the “RSR_fire” as a function of fire time.

NOTE! This document focuses on *ultimate strength* during fire, and the examples therefore describe structures, which will collapse, (i.e.: structures without fire protection).

Problems where thermal expansion is important are discussed in a separate section.

2 Fundamental Mechanical Response

The mechanical behaviour of structural systems exposed to fire is complex. The temperature increase causes thermal expansion. In *statically determinate* structures, the expansion does not induce additional stresses and have no effect on the resistance.

In *statically indeterminate* systems, the thermal expansion induces increased compression in compressive members and relaxation of stresses in tensile members. The thermal induced compression may cause members to buckle at low temperatures, often in the range of only 100-200° C.

Fortunately, thermal induced buckling is *displacement controlled*, i.e. buckling allows the member to obtain a new equilibrium position with increased lateral displacement, but it maintain considerable strength, it is the resistance of the adjacent structure to displace that cause the member to buckle, after buckling the same adjacent structure resists inward motion of the end of the buckled member.

By further heating the member continues to expand, but the expansion is at an increasing rate counteracted by the softening of the elastic modulus. For temperatures in the range of 500° C and above, the expansion is not much larger than at 100°C, and its significance is reduced substantially because the resistance decreases in absolute terms as the compressive member moves deeper into the post-collapse range.

In essence, thermal induced expansion induces a very complex force history in the structure, but at the ultimate resistance the structural system returns very much back to the behaviour as if expansion was neglected.

The fire mechanical response analysis may be carried out with three different methods:

2.1 The temperature-load domain method.

This is the most complete and complex analysis; it mimics real physical process closely and is considered to produce the "true" behaviour of the structure. The functional loads are first applied, and then the temperature is incremented stepwise.

As thermal expansion is included, compressive members will "buckle", but still contribute to the resistance of the system. As the temperature increases, the softening of elastic modulus causes increased deformations, but more important is the reduction of the yield strength.

At certain stages, the degradation of members subjected to heating may require a significant redistribution of forces in the system in order to carry the functional loads. In a real system this will cause the structure to displace dynamically to a new equilibrium state, but in static analysis it may be necessary to perform intermediate unloading of the functional loads followed by reloading to the equilibrium state. This process may occur several times, but "at the end of the day", it is not possible to reload to equilibrium level; the associated temperature is considered to be the critical on for the system. By this method is necessary to switch between application of temperature loads and functional loads. The switch from temperature load to functional loads is necessary when the determinant of the tangential stiffness matrix becomes negative.

2.2 The temperature domain method.

The vertical load is first applied, and then the temperature is increase up to ultimate collapse. The material properties are reduced according to the temperature level and thermal expansion is included. The method is similar to the temperature-load domain, but intermediate unloading of the functional loads is not carried out. The analysis will generally terminate at a lower temperature due to numerical ill-conditioning, (negative determinant)

2.3 The load domain *method, ("PushDown").*

The material properties are modified according to the maximum temperature of the member during heating, and then the functional loads are increased until collapse is triggered. The method is simplified as it neglects thermal expansion.

The procedure is denoted the *Pushdown* method in analogy with the *Pushover* concept adopted for jacket structures exposed to extreme waves. A primary objective of the method is to obtain information of the ultimate strength of the structure when the functional loads are increased beyond the nominal load level when the structure is exposed to the maximum temperature distribution and the associated collapse mechanism.

3 Fire Degradation of Structures

3.1 Introduction

Steel becomes weaker for increasing temperatures. Material tests carried out for different temperatures have given the stress-strain relationship from room temperature and up to 1200°C. These stress-strain curves have been used to create the "material degradation" curves. Figure 3-1 describes typical stress-strain curves (curves to the left) and the corresponding degradation curve (right). Degradation curves are defined in codes, f ex Eurocode / /, and used as follows:

Actual yield stress to be used at a given temperature, T:

$$\sigma = \sigma_{ROOM} \text{ x } fac(T)$$

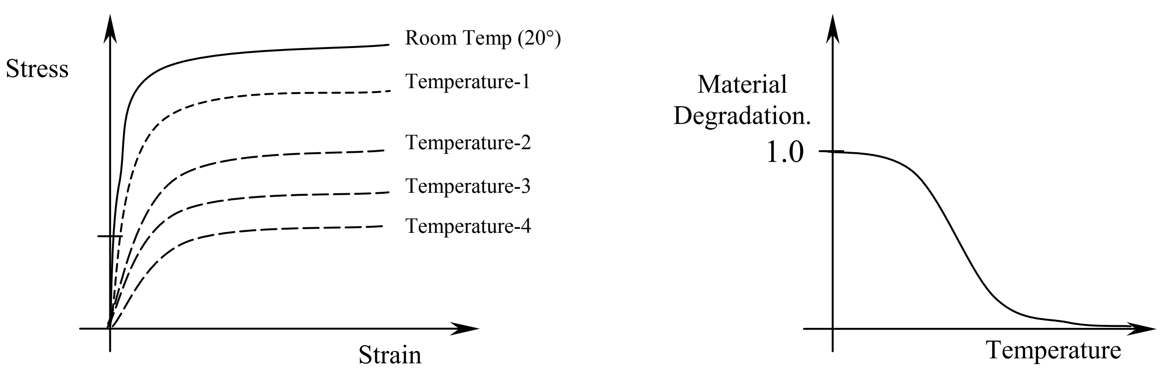


Figure 3-1 - Material properties for different temperatures and degradation factor vs temperature.

Similar to the lab testing of material, non-linear structural analysis tools may be used to determine the ultimate strength of a structure at different fire-times, (time = 0 when the fire starts). These ultimate capacity curves can then be used to create strength degradation curves of the entire structure.

The resistance of a structural system is typically expressed by "global history" curve, where the X-axis describes characteristic structural deformations (f ex vertical movement of one or several points on the topside) and the Y-axis describes the resistance, where level 1.0 represents the actual functional load (design load) to be carried. The load factor is increased until the peak resistance value is found for each fire time, (for example every minute).

Figure 3-2 presents typical global history curves for a structure exposed to self-weight. The degradation of resistance after the peaks is caused by local component instabilities (f ex buckling).

The peak resistances (circles) are then plotted vs. fire time, and this single curve describes the performance of the structure during fire.

The virtue of this method is that it gives a clear picture of how much the strength degrades during the fire and the available strength margins as a function of time. By contrast, a single simulation if the fire history provides "binary" information; either the structure fails or it survives- the margins are not known.

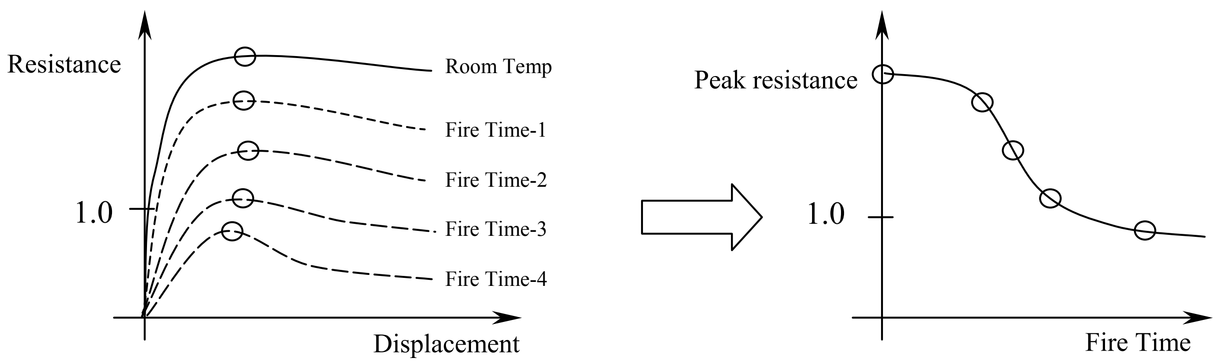


Figure 3-2 – Performance of the entire structure for different fire times.

Different fire scenarios will have different impact on the overall structural performance.

One resistance degradation curve is established per fire scenario and these curves together contain valuable information for the safety engineers, (critical scenarios, time to failure, margins, etc.).

The demand for resistance is = 1.0, i.e. when the functional loads are applied 100%. The resistance level gives information about the structure's reserve capacity. A robust topside structure will typically be able to carry 2-3 times the characteristic functional loads at room temperature, (with material factor of 1.15 and load factor of 1.3, the margin should be at least 1.5), and can therefore handle substantial weakening of the steel, (i.e.: high temperatures).

3.2 Pushdown procedure

In order to measure the gradual change of the structure, the peak resistance (ultimate strength) of the structure is computed for different stages of the fire as described above. The determination of peak resistance is given the name "PushDown", which is the counterpart to the "PushOver" for jacket structures.

Figure 3-3 shows resistance degradation curves for three fire scenarios. Initially, the structure is able to carry approx 2 ½ times the functional load, but depending on the fire, this capacity degrades over time. The red line describes the required minimum capacity, and global failure occurs when this curve is crossed, (here indicated with blue and black circles). Case-3 could be a transient gas fire with an initial high rate, and then a rapid reduction due to efficient depressurization of the process limits the temperature rise.

For fire Case-1, the structural collapse occurs for approx. 23 min, but since the curve is relatively horizontal, the actual time is more uncertain than for case-2, where the curve has a steeper, negative slope. Case 3 withstand the fire with sufficient margin.

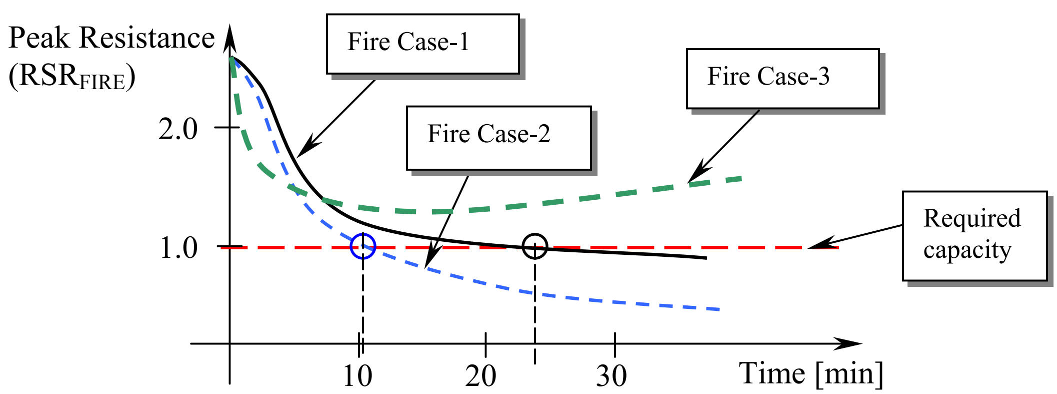


Figure 3-3 - Fire Degradation for two different fire cases

The pushdown procedure requires following to be included in the non-linear analysis:

- Temperature development of all structural components exposed to fire
- Degradation of every element according to EuroCode-3
- Member imperfection according to Eurocode, (Curve-C).

3.3 RSR_fire(t)

The peak resistance (scaling factor of the loads) shown in the global history plots (ref Figure 3-2) indicates the structure's reserve capacity. This load factor is defined as the RSR_fire(t):

" *Reserve Strength Ratio during fire*" and is a function of (fire) time

4 Thermal Expansion. Impact on Ultimate Strength.

4.1 General

The heating of steel results in increasing volume, approx. 0.1% elongation per 100°C. Free expansion does not introduce any forces, but within a frame structure with constrains, the expansion will typically introduce compression forces in the heated members and corresponding reaction forces in the neighbour members.

Thermal expansion forces do no represent an external load. The load sum is zero. A ductile metal structure therefore does not collapse due to these internal forces. Local yielding will typically limit and release these forces.

The pushdown procedure disregards the internal expansion forces. Instead, every member is given a certain imperfection in order to account for possible curvature caused by uneven heating of the cross sections.

Below, comparisons are made between the pushdown procedure and the “temperature-load domain” procedure, which includes the expansion effects for basic components in a steel frame.

4.2 Beam Bending

The *simply supported* I-Girder is spanning 20m and carries a mass of 20 metric tonnes at the midpoint. This gives a force of 200 kN as indicated in the figure. The stress level is approx 150 MPa, which is a typical level under "normal" operation conditions of a topside component.

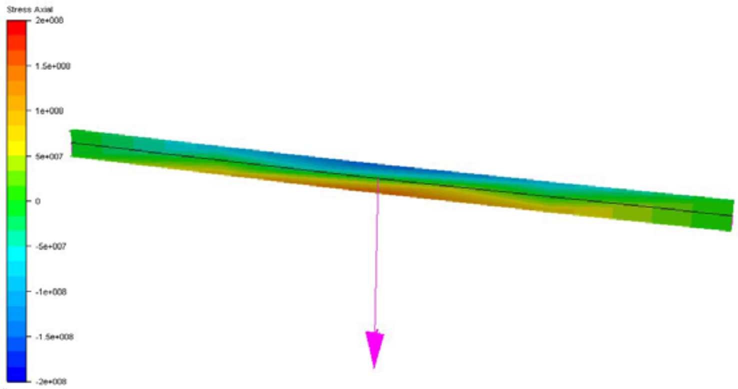


Figure 4-1 - Simply supported beam subjected to mid-point load.

The unprotected beam is exposed to a standard (simplified) hydrocarbon (HC) fire, and Figure 4-2 presents the temperature field after 7 minutes.

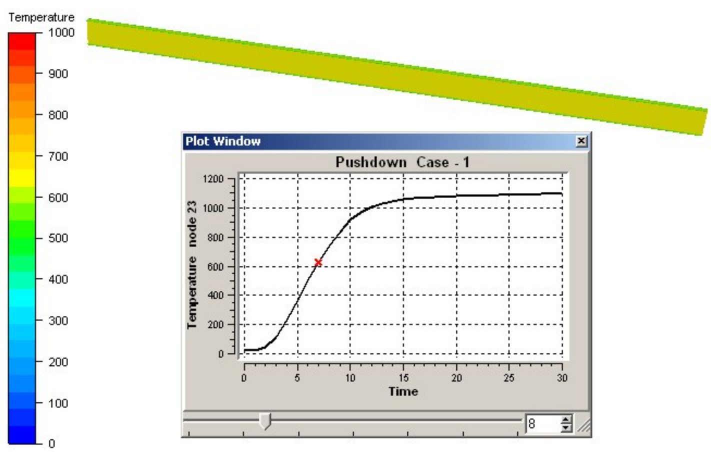


Figure 4-2 – The simplified HC-fire gives Uniform Temperature field.

This structure has no redundancy, and when the bending capacity at the mid-span is exceeded, the deformations increase rapidly. Figure 4-3 presents the midpoint deformation vs. fire time, and at approx 7 ½ minutes, the beam fails. (If the beam were axially fixed, the response would have been very different, with a more gradual reduction in the capacity).

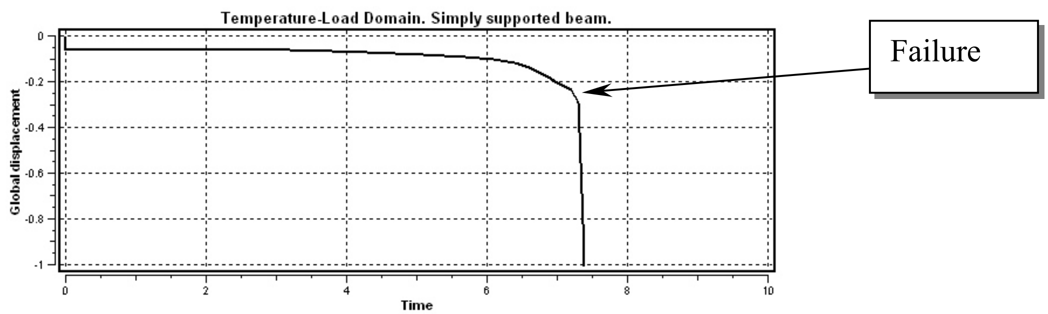
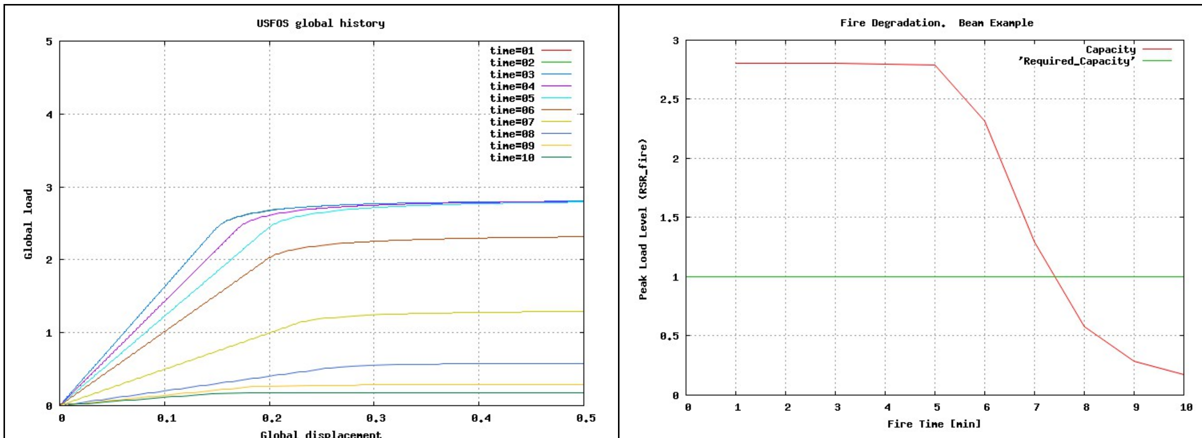


Figure 4-3 – Temperture-Load Domain procedure. Midponint deformation vs. fire time.

The same example is then computed using the "pushdown" procedure. The ultimate strength is found for times: 1, 2, .. , 10 minutes.

Figure 4-4 presents all global history plots (to the left) and the corresponding "Fire Degradation Curve" (right). For time 7 ½ minutes, the red (capacity) curve crosses the green (requirement) line.

Both methods predict the collapse time to 7 ½ minutes.



Global Histories for different fire times

Constructed Fire Degradation Curve

Figure 4-4 - Pushdown procedure. Global histories (left) and Fire egradation curve (right)

4.3 Column Buckling

4.3.1 Uniform temperature. HC-fire. No Gradients

A pinned column with diameter 400mm, thickness 20mm and length 10m carries a vertical load of 100 metric tonnes at the top point. The column is exposed to a uniform HC-fire, (200kW/m^2). After approx 11 minutes, for a uniform temperature of approx 700°C, the column becomes unstable and buckles as shown in Figure 4-5.

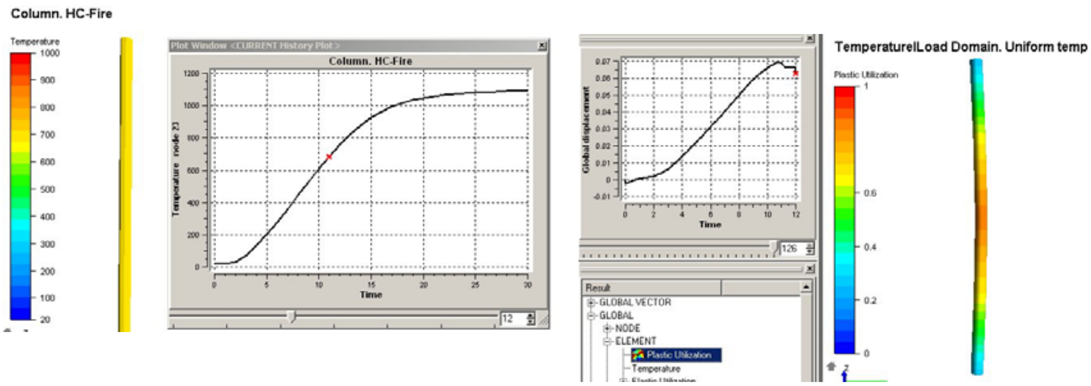
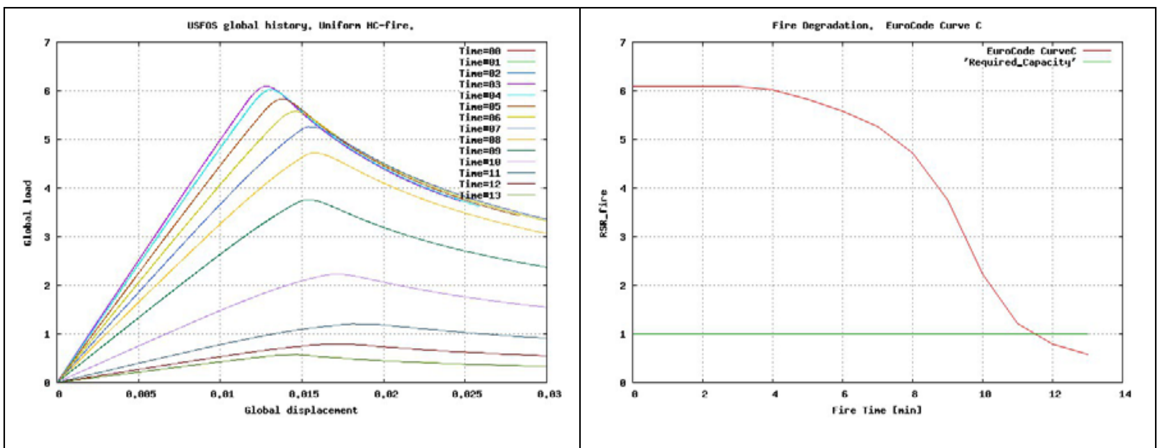


Figure 4-5 - Temperature field (left) and Temperature-Load Domain response.

The same model is then analysed for fire times 0, 1, 2, .., 13 minutes using the pushdown procedure, and Figure 4-6 presents the global histories and the constructed fire degradation curve. The pushdown procedure predicts collapse after approx 11 minutes.

=> Both methods predict the collapse time to 11 minutes.



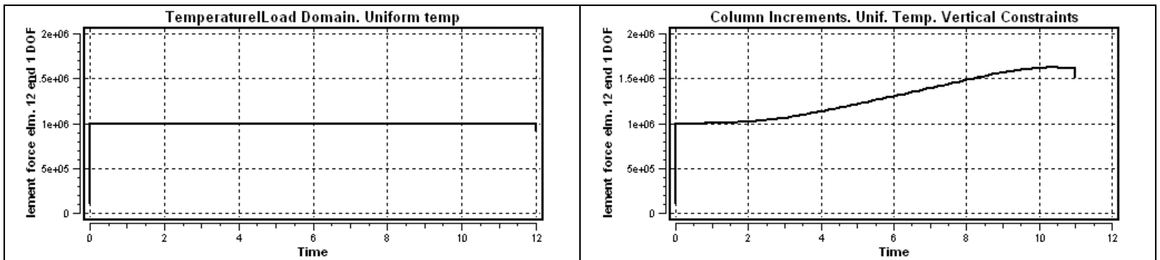
Global histories | Fire degradation curve

Figure 4-6 – Pushdown. History Plots (left) and Fire Degradation curve, (right). Eurocode C.

USFOS
Reality Engineering

The column is also checked for a special boundary condition at the top node. A contact spring with stiffness 10 MN/m will prevent the column to expand freely, and expansion forces are introduced. (The contact spring gives no resistance for downward movements. It is introduced to check how the increased imperfection caused by the expansion will affect the final results). This upward resistance could represent a typical beam, (when the column spans between beams in a real 3D frame).

Figure 4-7 presents the axial force histories for the two cases. With no constraints, the axial force is constant equal to the external load of 1MN. For the constrained alternative, the thermal expansion will introduce additional forces, which are released when the column starts buckling.



Free | Constrained

Figure 4-7 – Axial force history. Free (left) and constrained (right)

Figure 4-8 presents the history plots for the two cases, and for both alternatives, the failure starts after approx 11 minutes, (indicated with circles).

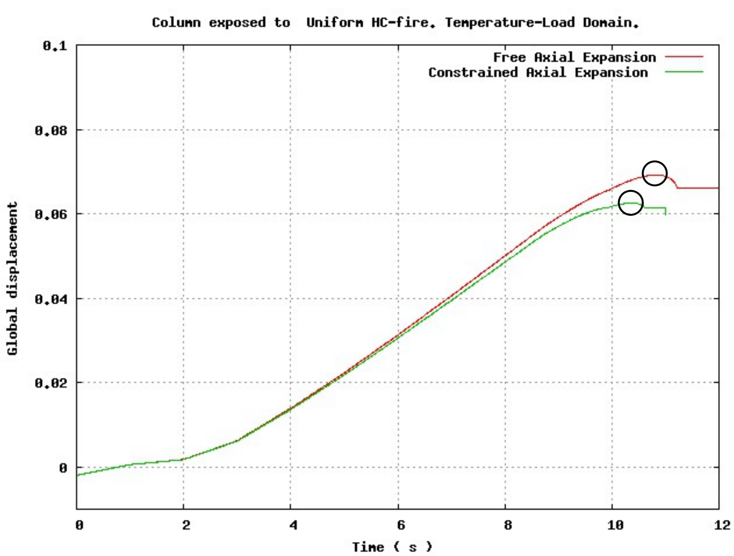


Figure 4-8 – Temperature-Load Domain. Free expansion (red) and constrained (green)

4.3.2 Uneven Radiation. Temperature gradients

The column is now exposed to a radiation field of approx 210 kW/m^2 from one side and 180 kW/m^2 from the opposite. This will create temperature gradients over the cross section. The heat-flux (in contrast to the HC-fire) starts with full intensity from time =0, and the column is heated more rapidly than in the previous fire example. After approx 7 minutes, the column becomes unstable.

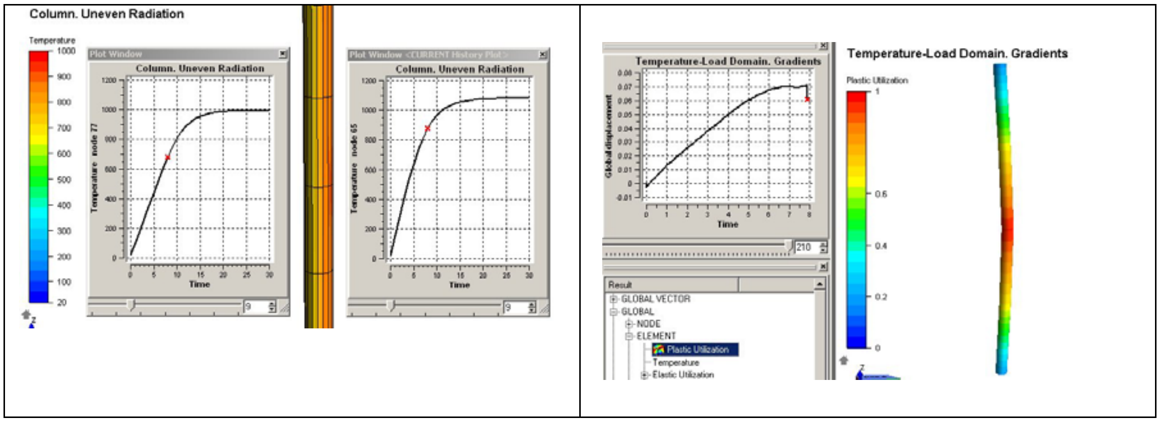
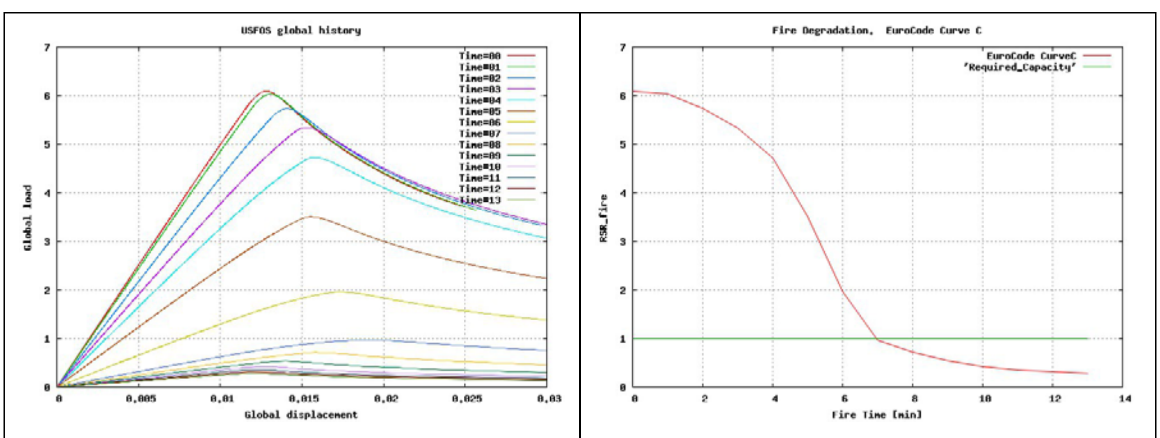


Figure 4-9 - Temperature field (left) and Temperature-Load Domain response. Gradients.

Pushdown analysis is carried out for fire times 0, 1, 2, .., 13 minutes using the pushdown procedure, and Figure 4-10 presents the global histories and the constructed fire degradation curve. The procedure predicts collapse after approx 7 minutes.

=> Both methods predict the collapse time to 7 minutes.



Global histories | Fire degradation curve

Figure 4-10 – Pushdown. History Plots (left) and Fire Degradation curve, (right). Eurocode C.

Also this fire case is checked for a free and constrained column.

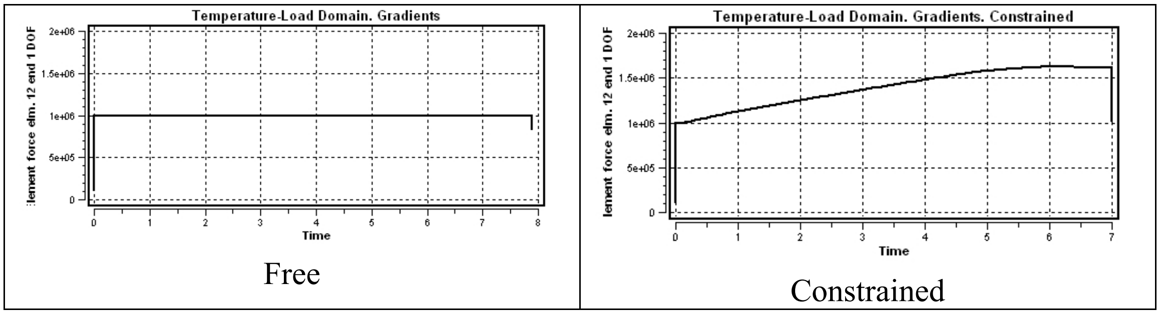


Figure 4-11 – Axial force history. Free (left) and constrained (right)

Figure 4-12 shows that the constrained column becomes unstable after approx 6 ½ minute, while the free "lasts" ½ minute longer, (the peaks are indicated with circles).

It should be emphasized that predicting accidental response contains many uncertainties. The most uncertain is the fire accident itself, and how the fire develops in time and space. Differences in the order of ½ minute will have little impact on the fire degradation curve of the structure and have no practical impact on the conclusions.

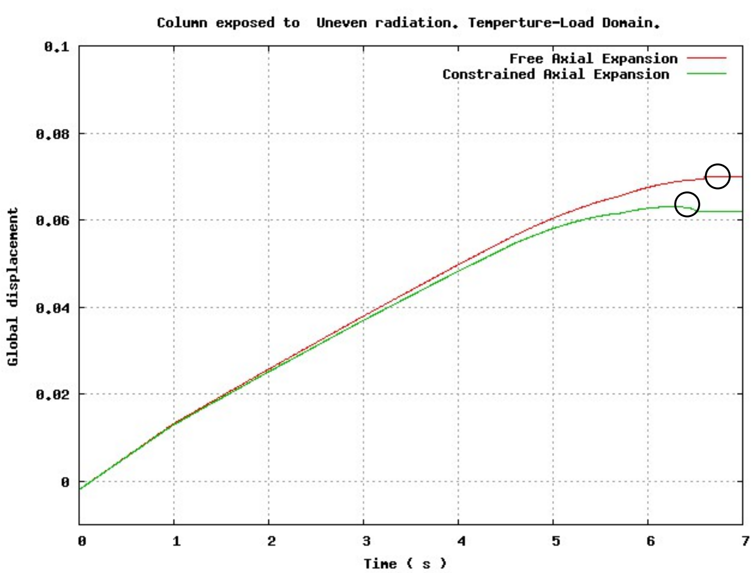


Figure 4-12 – Temperature-Load Domain. Free expansion (red) and constrained (green)

4.4 K-braced frame.

A K-braced cantilever frame is loaded as shown in Figure 4-13 with a concentrated force with magnitude 187kN. The braces have diameter 168mm and thickness 6mm. The braces are spanning 6.7m between “legs”, which have diameter 356 and thickness 13mm. The yield strength is set to 287MPa. The stress levels in the braces are approx 70 MPa in the cold condition.

Parts of the frame are artificially heated 100°C per minute as shown in Figure 4-14. The frame is then analysed using the Temperature-Load procedure and the Pushdown procedure.

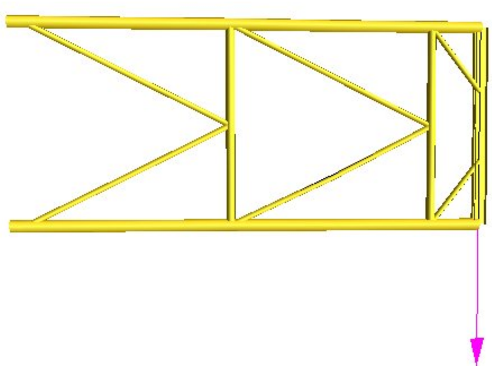


Figure 4-13 – K-braced frame exposed to vertical force.

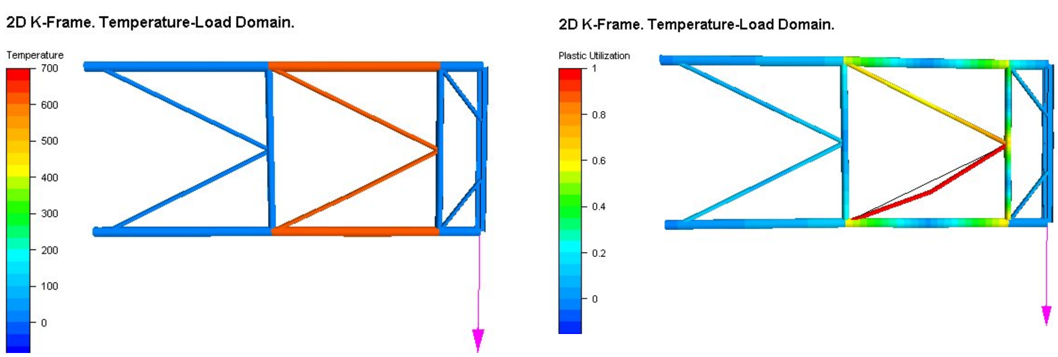


Figure 4-14 – Heating of one section of the frame. Failure mode (right)

This structure has little redundancy, (due to the K-bracing), and when the compression brace buckle, the frame capacity reduces rapidly.

Figure 4-15 shows the “global history” and the axial force history. For time 6 ½ minutes, the frame fails.

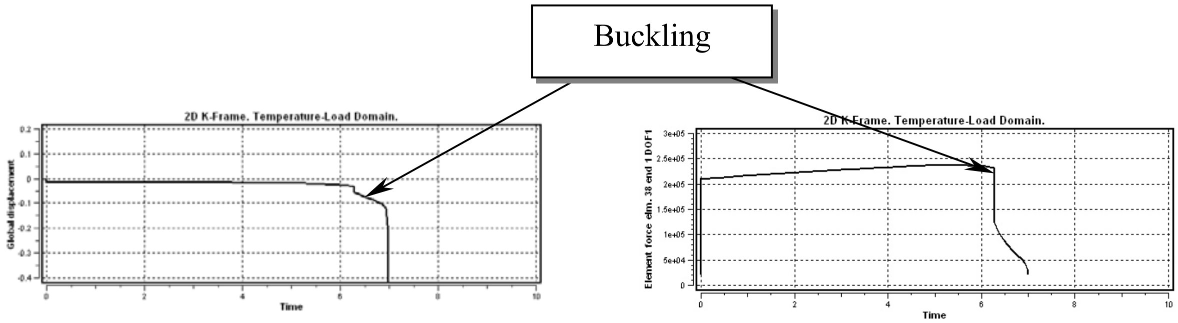
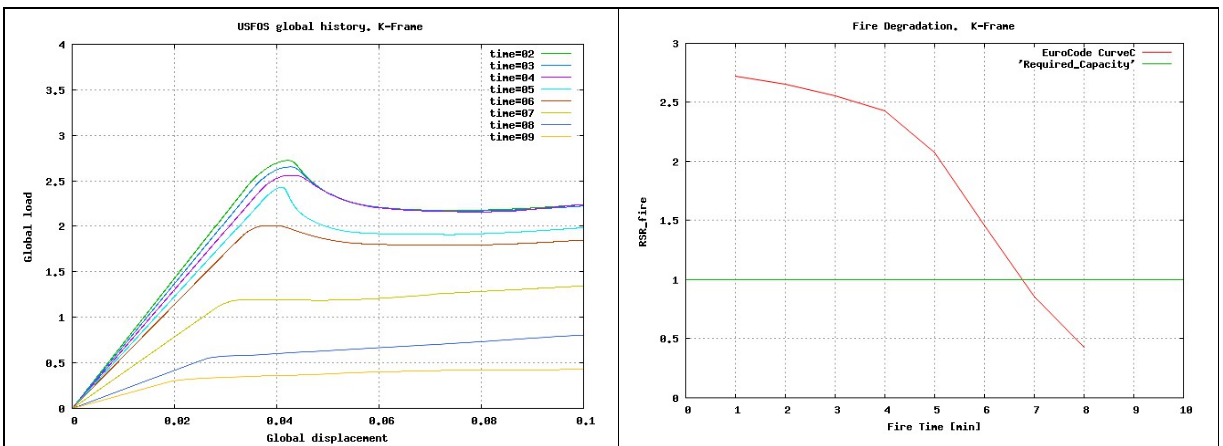


Figure 4-15 – Temperture-Load Domain procedure.Displacement (left) and brace force (right).

The same example is then computed using the pushdown procedure. The ultimate strength is found for times: 1, 2, .. , 8 minutes.

Figure 4-4 presents all global history plots (to the left) and the corresponding Fire Degradation Curve (right). For time 6 ½ minutes, the red (capacity) curve crosses the green (requirement) line.

Both methods predict the collapse time to 6 ½ minutes.



Global Histories for different fire times

Fire Degradation Curve

Figure 4-16 - Pushdown procedure. Global histories (left) and Fire egradation curve (right)

4.5 X-braced frame

Figure 4.17 shows a plane frame subjected to a vertical load at the top right corner node #1,indicated by an arrow in the figure. The members indicated with the red colour in the lower storey of the frame are subjected to uniform heating.

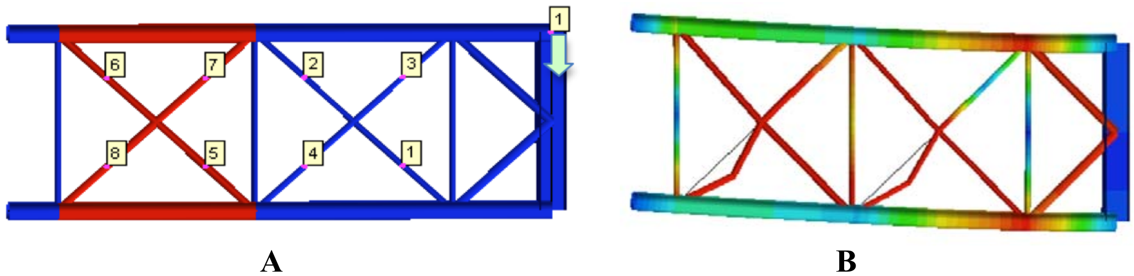


Figure 4.17 - Plane frame subjected to fire loads. A) Model and Temperature field. B) Collapse mode

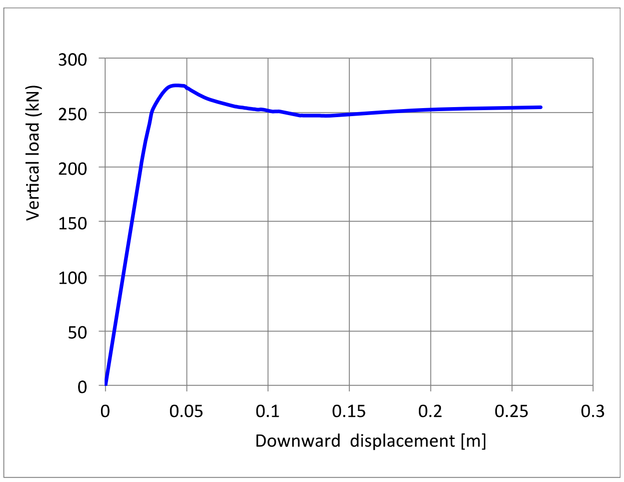


Figure 4.18 - Vertical force versus Downward displacement

The behaviour of the frame under normal temperature is determined by a pushdown analysis. The vertical force versus downward displacement of the loaded point is plotted in Figure 4.18. The ultimate strength is 275 kN. The collapse mode is illustrated in b; failure is initiated by buckling of the upper compressive brace #4, followed by the lower by the lower compression brace #8. For large lateral deformations plastic hinges are formed in the legs ate the upper storey.

Temperature-load domain
The frame is analysed for a vertical load of 80 kN, 140 kN and 200 kN, respectively. This corresponds to 30%, 50% and 70% utilisation respectively.
Figure 4.19 shows the temperature and vertical load history for the frame for a vertical load equal to 200 kN. The temperature is incremented after the vertical load has been applied. Brace #8 "buckles" for a temperature of about 100 ^{0}C, but this is not critical.

When the temperature reaches approximately 500 ^{0}C, buckling of brace #3 causes redistribution of forces in the system. In the displacement range 0.08m – 0.16 m, the analysis is conducted in the load domain, with intermediate unloading of the vertical load. Equilibrium is restored and the temperature is increased to 540 ^{0}C. This induces significant side sway of the frame. A new load domain phase takes place between 0.44 m and 0.54 m, but the temperature increase after that is negligible.

Final collapse is governed largely by downward motion of the lower story. For large downward displacements rupture may take place due to excessive plastic straining of the lower tension brace. Plastic straining for members with high temperatures is uncertain; few laboratory tests have been conducted.

An important point is *that thermal expansion is beneficial* in this respect; it reduces the demand for plastic straining in tensile members.
Figure 4.20 shows the plastic elongation in brace #6, with the highest plastic straining.. The elongation is less than 0.1m up to a downward displacement of 0.5 m. The member is 2.1 m long, which gives an average strain of less than 5%. It is possible that this may be take place without fracture

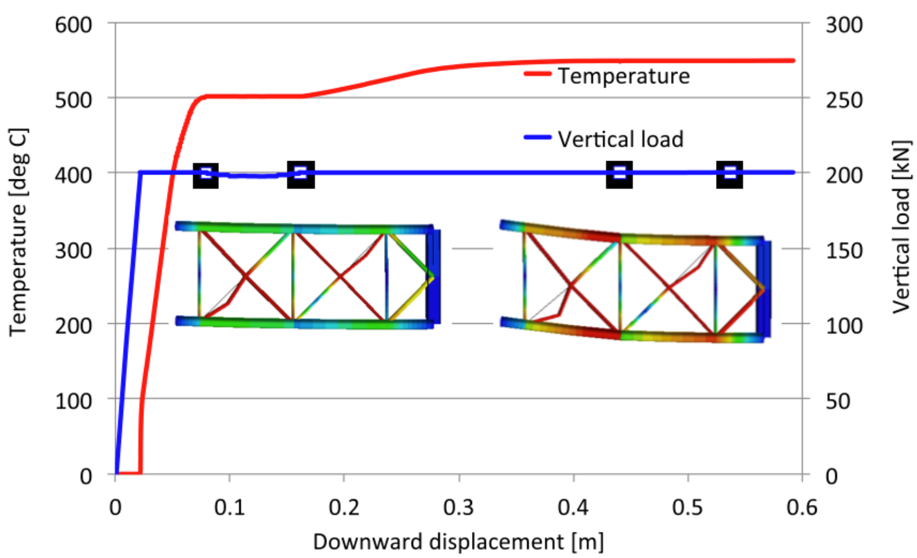


Figure 4.19 Temperature and vertical load histories. The square marks on the vertical load history indicate start and end of the load domain stages.

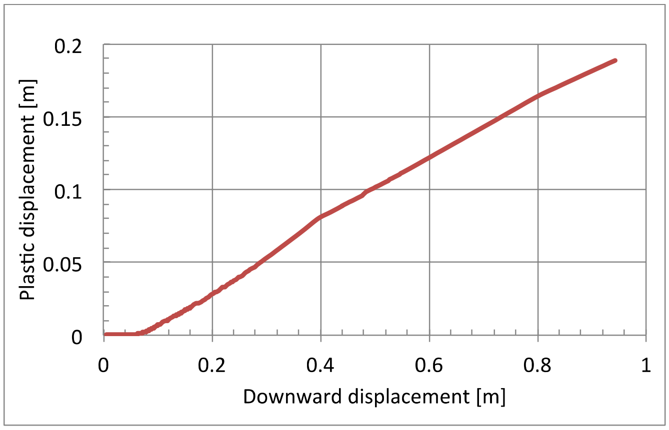


Figure 4.20 Plastic elongation of brace #6

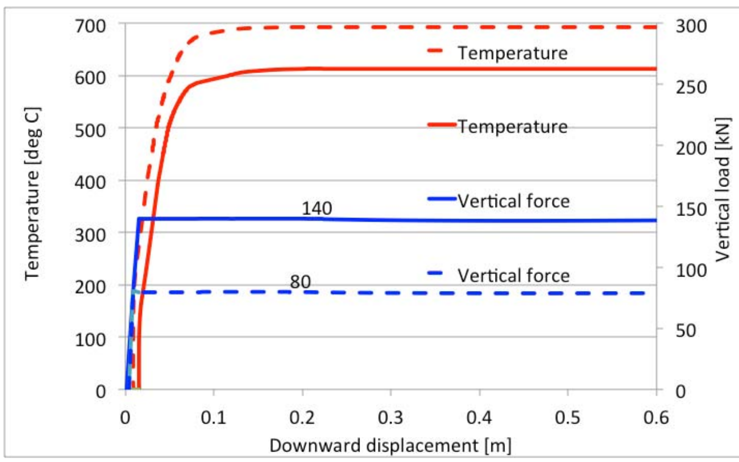


Figure 4.21 Temperature and vertical load histories. The square marks on the vertical load history indicate start and end of the load domain stages

Figure 4.21 shows the temperature evolution and deformation history for vertical force equal to 140 kN and 80 kN. The unloading is moderate, but it is not possible to restore equilibrium within the deformation range shown. Practically, the critical temperature is 613 deg C for 140 kN and 693 deg C for 80 kN.

Temperature domain

The downward displacement versus temperature is plotted in Figure 4.22. The collapse temperature for the three load levels is approximately 700 ^{0}C, 600 ^{0}C, 500 ^{0}C.

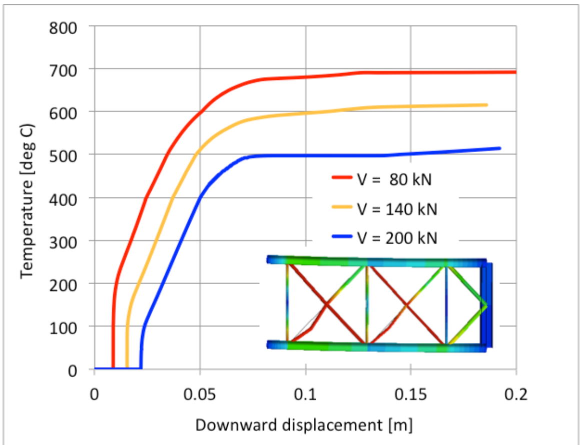


Figure 4.22 Downward displacement versus temperature for three load levels using the *temperature domain* method. Collapse mode for V = 140 kN is illustrated to the right.

Pushdown analysis

Figure 4.23 shows force-deformation relationships from Pusdown analysis for 50 ^{0}C temperature intervals from 400 ^{0}C to 700 ^{0}C and the peak resistance versus temperature

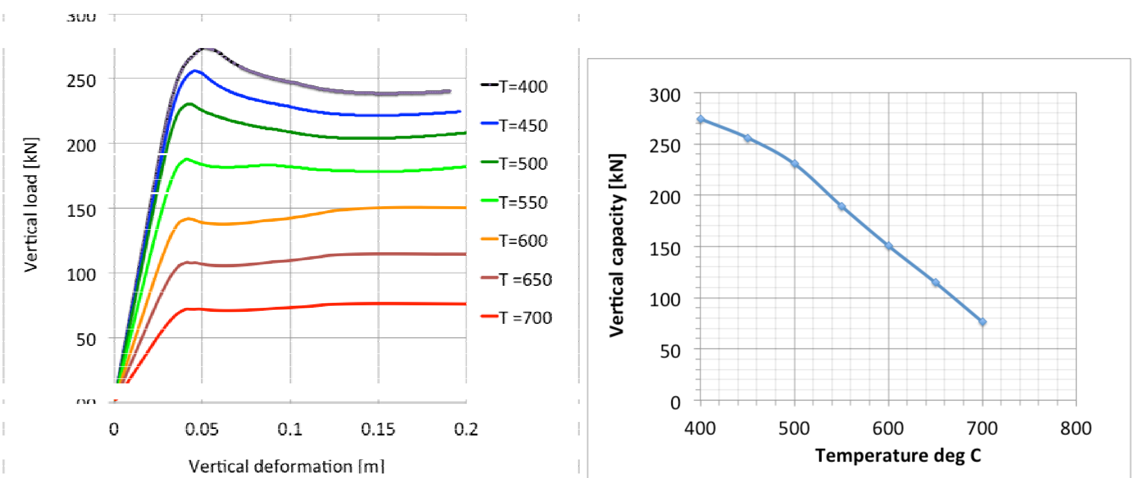


Figure 4.23 Force-deformation relationships from Pusdown analysis (left) and peak resistance versus temperature (right)

The resistance 400 ^{0}C is the same as that for normal te,perature. The critical tempretaure is approximately 535 ^{0}C for 200 kN, 615^{0}C for 140 kN and 695 ^{0}C for 80 kN. These results are very close to those obtained with the temperature-load domain method and confirms the validity of the Pushdown method.

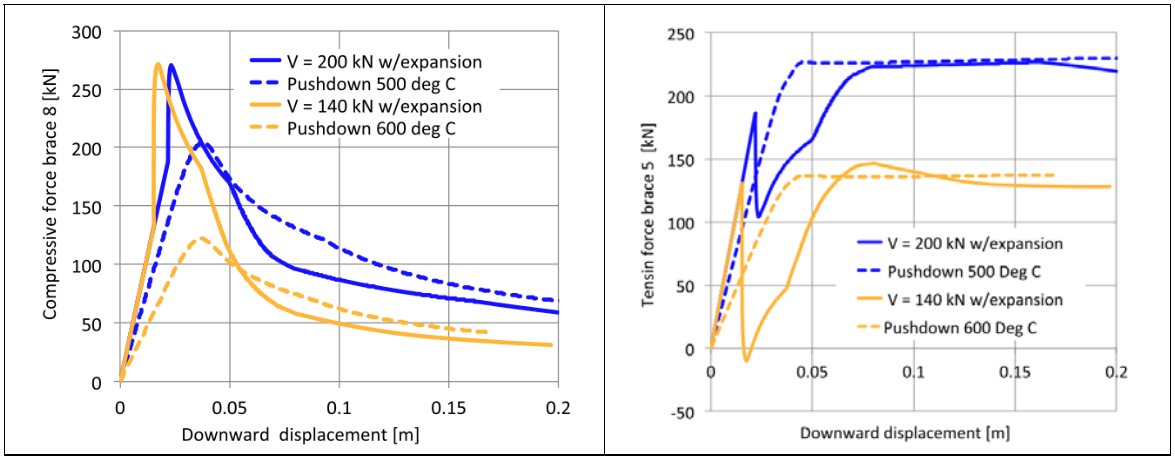


Figure 4.24 Force histories form temperature-load domain analysis and *Pushdown* analysis: Compressive brace #8 left figure, tension brace #5 right figure.

Figure 4.24 compares the force histories for brace # 8 (compression) and brace #5 (tension) for the *temperature-load domain* analysis and the *Pusdown* procedure.

In the *temperature domain* analysis the force in the compression brace increases fast due to thermal expansion when the temperature increases and the brace “buckles” for a low temperature (less than 100 ^{0}C. for V=200kN and slightly larger than 100 ^{0}C for V=140kN). This causes a pronounced change in the stiffness, but buckling is entirely displacement–controlled, so the brace continues to contribute substantially to the load carrying in the post-buckling range during further heating.

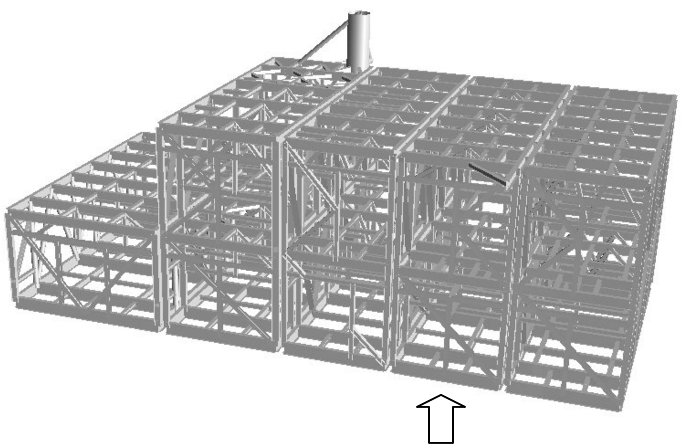
Collapse is triggered for a lateral displacement of 0.07 m for V=200kN and 0.1 m for V=140kN. The force in the compression brace is substantial in both cases; 100kN for V=200kN and 50kN for V=140kN.

The tensile force in brace # 5 follows closely the force in the compression brace. The increasing force in the compression brace cause intermediate unloading in brace # 5. Actually, for V=140 kN, brace #5 unloads completely (and even enters compression), but the tension force increases rapidly when the compression brace unloads due to “buckling”.

It is observed that the final force levels using the *Pushdown* procedure is very close the ones obtained with *temperature domain* method. The downward *displacement* is smaller because thermal expansion is neglected, but the ultimate resistance is virtually identical.

4.6 Topside Modules.

The *unprotected* topside modules are exposed to a huge pool fire, which exposes columns and beams heavily in the module indicated with the arrow. Figure 4-26 shows the temperature "footprint" computed by KFX-FAHTS at the time for failure, (5 ½ min).



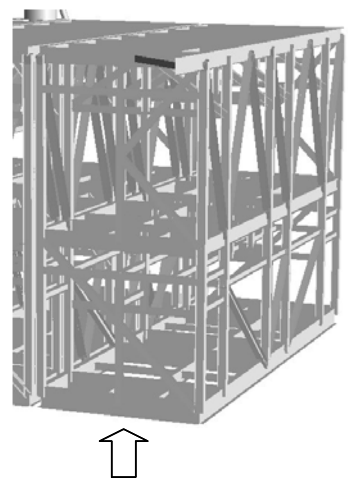


Figure 4-25 - Topside Modules.

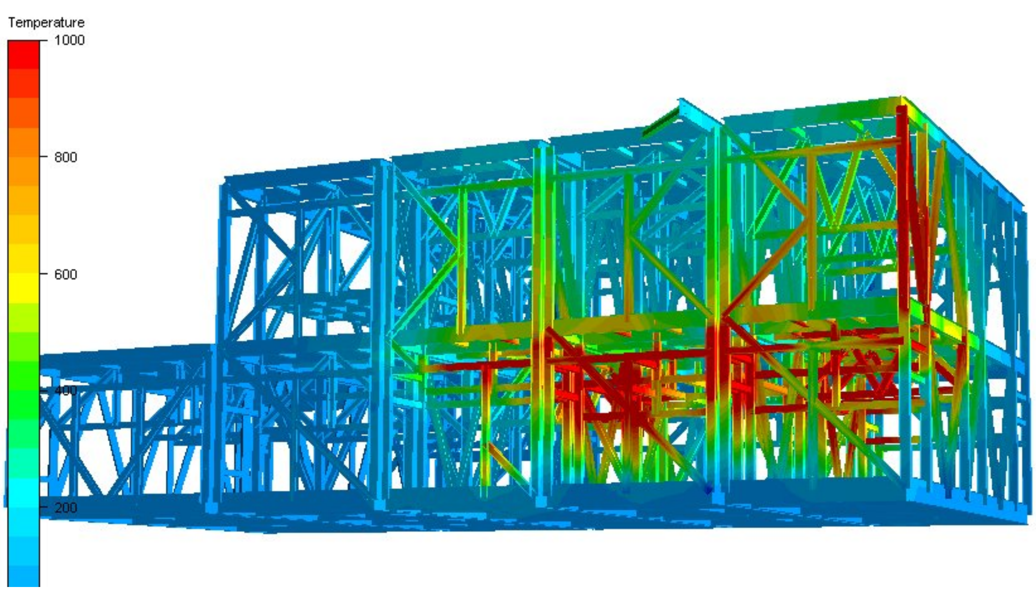


Figure 4-26 – Temperature Field at time for collapse.

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Figure 4-27 shows the response after 5 ½ minutes when the outer columns buckle and the module tilts forward. The thermal expansion (coefficient = 1.2 10^{-5}) lifts the module slightly before the sudden change to negative vertical displacement.

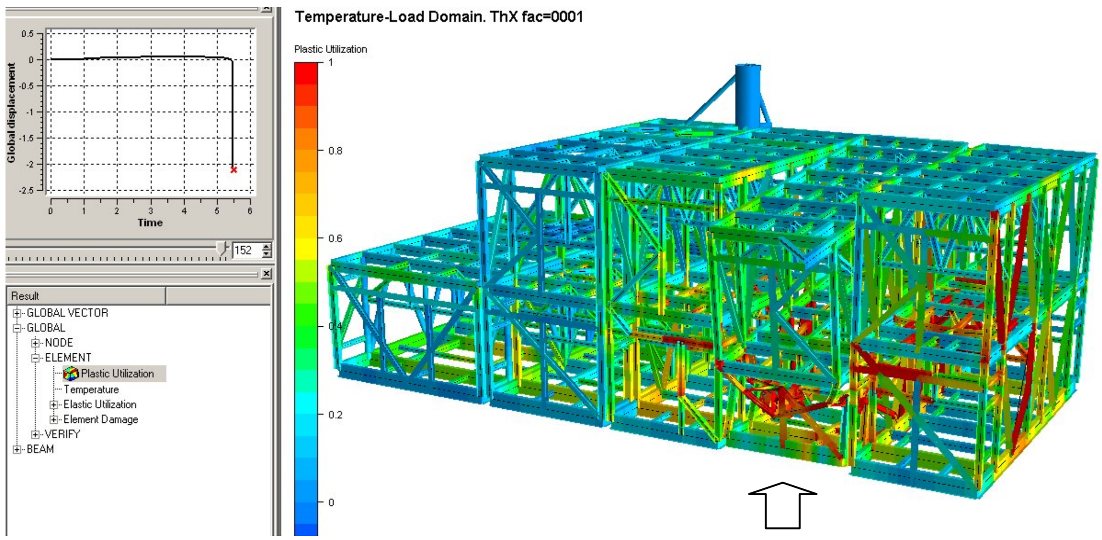


Figure 4-27 - Response at time for failure.

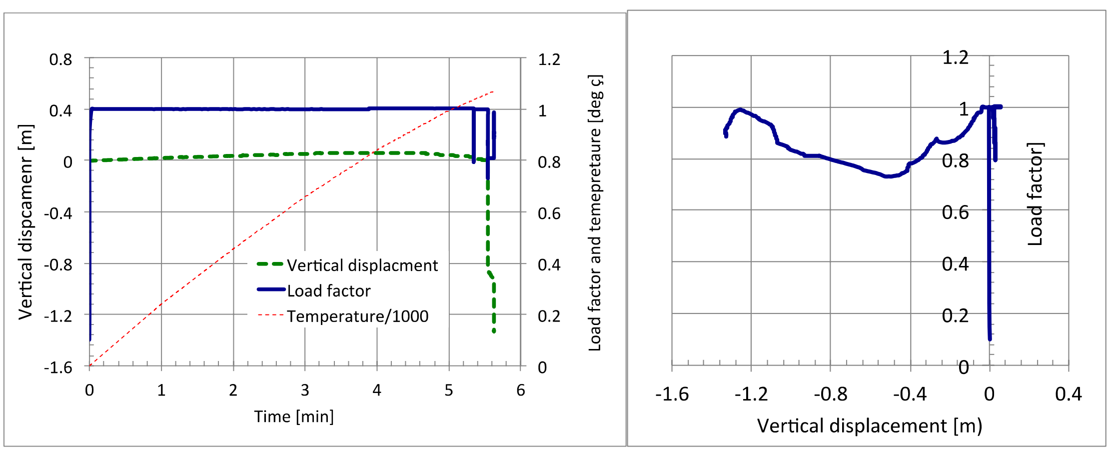
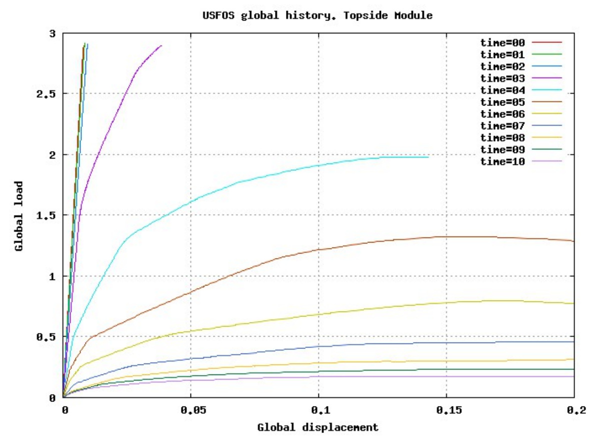


Figure 4.28 – Time histories (left) and Load factor versus vertical displacment (right)

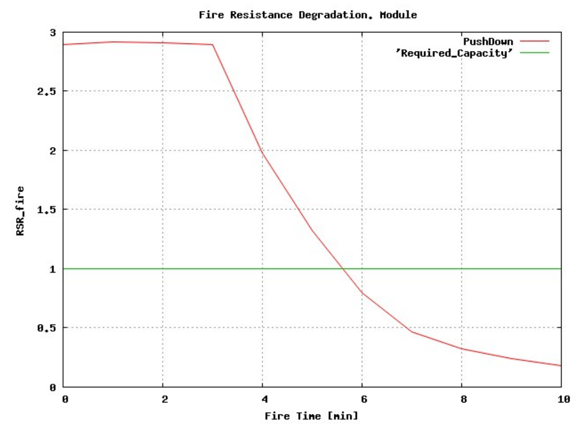
Figure 4.28 shows also the loaf factor versus time. Very close to collapse it is necessary to perform intermediate unloading to facilitate redistribution of the load carrying. However, shortly afterwards the structure has degraded too much and collapse is triggered. This is evidence by the plot of the load factor versus vertical displacement. It is interesting to see that the structure is *almost* capable of restoring equilibrium for a vertical displacement of -1.3 m, but then collapse is inevitable.

The same modules are then computed using the pushdown procedure. The ultimate strength is found for times 1 to 10 minutes. Figure 4-29 presents all global history plots (to the left) and the corresponding Fire Degradation Curve (right). For time 5 ½ minutes, the red (capacity) curve crosses the green (requirement) line.

Both methods predict the collapse time to 5 ½ minutes.



Global Histories for different fire times



Fire Degradation Curve

Figure 4-29 - Pushdown procedure. Global histories (left) and Fire egradation curve (right)

The modules are then checked for varying thermal expansion coefficient, where the original thermal coefficient is divided by 10, 100 and 1000.

The Temperature-Load domain procedure is used.

Figure 4-30 compares the global response for the four different cases. The red curve (1:1) represents the case with thermal expansion = $1.2\ 10^{-5}$.

The 1/10 means coefficient=$1.2\ 10^{-5}$ / 10 = $1.2\ 10^{-6}$, etc.

The four curves are not identical*), but give same conclusion with respect to collapse time, which is 5 ½ minutes.

**) It should be emphasized that the general uncertainties in connection with the fire accidents are relatively large, (i.e.: the extent and duration), and substantial larger then the minor deviations observed in the curves for structural response.*

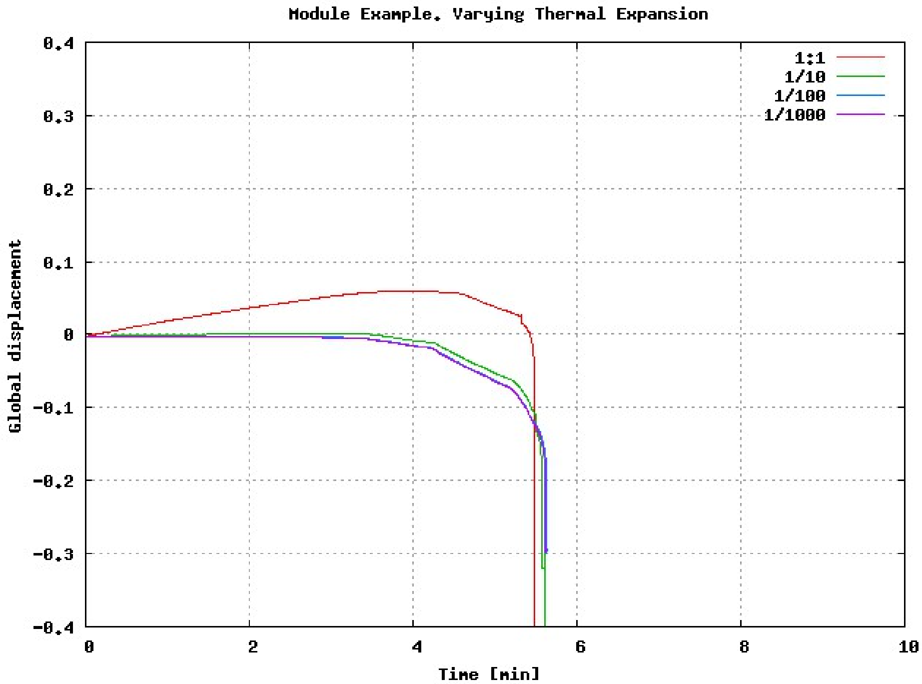


Figure 4-30 - Global History for varying thermal expansion coefficients. Temp-Load Domain.

5 Input to USFOS

The pushdown analysis input consists of a structural model, a temperature load file ("beltemp-input") and the USFOS control file. The USFOS control file is defined in Figure 5-1 and the "beltemp" file is defined in principle in Figure 5-2.

```
HEAD       Usfos  PushDown Input Example
                   USFOS AS
                    2014
'
'          KeyWord    Value
PushDown  LoadCase     7           ! Use Fire Load Case 7
'
SteelTDep                          ! Eurocode Efficient Yield
'
'         Curve  LoadType  LoadCase
CINIDEF  Fire   BaseShear    3     ! EuroCode Curve C
'
'         nloads   npostp   mxpstp   mxpdis
CUSFOS      10       200      1.00     1.0
'
'         lcomb    lfact    mxld     nstep     minstp
             1      0.01     3.0     2000      0.001  ! Gravity Loads
'
'       n   Node Dof Fac
CNODES 1    3   3   -1             ! Definition of Global Displacement
```

Figure 5-1 – Complete Usfos control file for PushDown analysis

Command	Description	Comments
PushDown	Modifies every element's material properties based on the actual temperature and degradation function. Yield strength and E-mod are modified.	Ensures that the cross section capacity becomes correct for the actual temperature distribution.
SteelTDep	Activates the steel material degradation curve, "Effective Yield" according to Eurocode-3. The E-module is also updated.	Other degradation curves could also be used: AlumTDep and UserTDep.
CINIDEF	Applies member imperfection, Eurocode-3, curve C. In the example, mechanical load case 3 is used to define the orientation of the imperfection	Gives a relatively large imperfection. In the order of 0.5%. Impacts the column buckling capacity. Represents the thermal expansion effects.
CUSFOS	The usual load control. Only the function loads to be carried are included	Since the structure could be very soft and weak due to degraded materials, the load steps should be small.
CNODES	The usual definition of "Global Displacement"	The vertical deformation (dof=3) is normally the most interesting. Monitoring several points on the structure are recommended.

Table 5-1 - Usfos control file commands.

The BELTEMP record defines the temperature development of every element, both mean temperature and gradients over the cross section. These temperature load files are computed prior to the USFOS analysis, and one temperature file represents one fire scenario. The temperatures could be computed "by hand" or using tools like FAHTS.

The increments, i.e.: The changes from previous cases are specified. The load case IDs have to be defined in *increasing order*, where the first load case (here set to 4) defines the first fire time, and the next load case (here 5) defines the next fire time.

The command LCASETIM is just information about the fire time corresponding to the actual load case. In the example, load case 4 represents the temperature after 1 minute, load case 5 represents time=2 minutes, etc.

The temperature rise in a real steel structure is a relatively slow process, and storing temperature results for every minute gives high precision also for transient fires.

Minute is the most used time-unit by the safety engineers and it is recommended to use this unit in all result presentation.

```
'             load case    time
 LCASETIM         4         1.0
'
'             load case   element     Mean       Gradient    Gradient
'                no       number   temperature     Y-dir       Z-dir
'                                  (increment) (increment) (increment)
 BELTEMP          4          12      127.560      85.954       0.000
 BELTEMP          4          23      127.560      83.585       0.000
''
 LCASETIM         5         2.0
 BELTEMP          5          12      106.648      87.733       0.000
 BELTEMP          5          23      106.648      78.111       0.000
''
 LCASETIM         6         3.0
 BELTEMP          6          12      104.760      67.906       0.000
 BELTEMP          6          23      104.760      69.694       0.000
''
 LCASETIM         7         4.0
 BELTEMP          7          12      101.471      56.187       0.000
 BELTEMP          7          23      101.471      56.514       0.000
```

Figure 5-2 - Usfos temperature input file

6 Problems where Thermal Expansion are important.

Above, it has been demonstrated that the “zero-sum” forces created by thermal expansion have no practical impact on the ultimate strength during fire.

However, this does not mean that thermal induced stresses could be disregarded in general.

The main reason for the low importance for the ultimate strength problems is the fact that when the steel yields and get plastic deformations, these locked in forces will be released. If steel is heated up 800°C, the thermal elongation becomes in the order of 1%. If the component is not free to expand, the plastic *compression* strain becomes 1%, which is a “small” plastic strain in connection with ultimate strength analysis.

6.1 Elastic stresses

However, if the structure behaves elastically the thermal induced stresses could be significant. For example: If a structure is exposed to repeated heating, (for example a flare boom), the thermal induced stresses in the exposed components could experience large stress ranges with possible fatigue problems.

The temperature-load domain procedure should be used to compute the elastic stresses.

6.2 Elastic deformations

If the elastic deformations caused by for example uneven temperatures over the cross section should be computed, the temperature-load domain procedure has to be used.

7 Summary

The “RSR_fire(t)” degradation curves describing structural degradation versus fire time contain the essence of many simulations. Information about the initial reserve capacity (or safety margin) and how the different fire scenarios (accidents) change the performance over time is crucial for the safety engineers.

The conventional (fire)time domain method is able to predict collapse (or not), but gives little information about how the performance change over time, and does not express the reserves.

In order to design these RSR_fire(t) curves, a series “pushdown” analyses are performed, for example one per minute fire time. In the pushdown analysis, the cross section capacity for the actual temperature field of every member is computed. The thermal induced deformations (thermal expansion) is included in terms of the initial deformations defined in Eurocode-3 curve C.