AARHUS UNIVERSITET

Eurocode 3 – Methodology, Principal Approaches and Algorithms

ADVANCED METHODS FOR CALCULATION OF STEEL STRUCTURES IN FIRE CONDITIONS

KULDEEP VIRDI

METHODOLOGIES IN EUROCODE 3 FOR ESTABLISHING FIRE RESISTANCE

EUROCODE STRATEGIES

Eurocode 3 allows three strategies for establishing fire resistance:

Time:	$t_{fi,d} > t_f$	Usually only *directly* feasible using advanced calculation models.
Load resistance:	$R_{fi,d,t} > E_{fi,d}$	Feasible by hand calculation. Aim is to find reduced resistance at design temperature
Temperature:	$\theta_{cr,d} > \theta_d$	Simplest EC3 method. Find critical temperature for loading and compare with design temperature

TIME DURATIONS

MINIMUM FIRE RESISTANCE PERIODS

Whatever the strategy for checking the fire resistance, the fire resistance has to be provided for a certain duration.

This duration needs to be sufficient for

- **allowing occupants to escape, and for**
- **allowing fire authorities to try and extinguish the fire**

The requirements for minimum duration of fire resistance are specified in local building regulations.

MINIMUM FIRE RESISTANCE PERIODS

For example, UK Building Regulations have the following requirements.

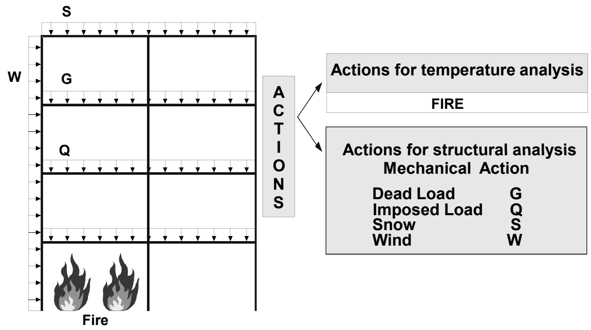
	Basement		Above Ground			
	> 10m	< 10m	< 5m	< 20m	< 30m	> 30m
Offices:						
Non-sprinkler	90	60	30	60	90	NO!
Sprinkler	60	60	30	30	60	120
Shops, Commercial:						
Non-sprinkler	90	60	60	60	90	NO!
Sprinkler	60	60	30	60	60	120
Car Parks:						
Open-sided			15	15	15	60
Other	90	60	30	60	90	120

To storey <u>floor</u> level

Times given in min

ACTIONS

ACTIONS



PARTIAL SAFETY FACTORS FOR LOAD RESISTANCE UNDER FIRE

EC3 PARTIAL SAFETY FACTORS

Ambient temperature strength design

γ_G	= 1.35	Permanent loads;
$\gamma_{Q,1}$	= 1.50	Variable loads

In Fire limit state

γ_{GA}	= 1.00	Permanent loads; accidental design situations
$\psi_{1,1}$	= 0.50	Combination factor; variable loads

MATERIAL PROPERTIES IN FIRE

FIRE ENGINEERING CALCULATIONS

The key factor in calculating the resistance to loads under fire is the degradation of the strength of the materials at high temperature.

Both steel and concrete undergo degradation of strength.

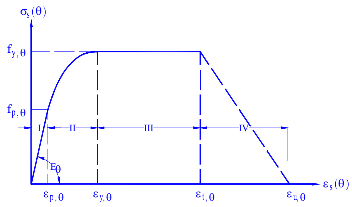
Experimentally obtained material stress-strain curves are used for this purpose. Eurocodes offer simplified models for stress-strain curves.

STEEL

EC3 STEEL STRESS-STRAIN CURVES

Parameters used to define the idealised steel stress-strain characteristics at high temperatures

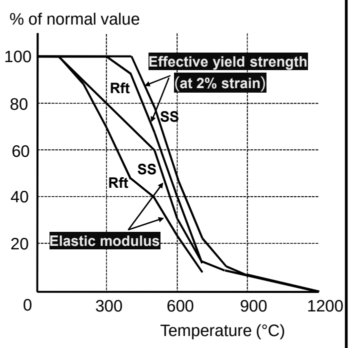
$f_{y\theta}$ – yield stress
E_{θ} – elastic modulus
$f_{p\theta}$ – proportionality limit stress
$\varepsilon_{t\theta}$ – limiting yield strain



EC3 STEEL STRESS-STRAIN CURVES

Reduction in yield strength and stiffness are very similar for various grades of structural steel and hot-rolled reinforcing bars (SS).

Cold-worked reinforcing bars S500 deteriorate more rapidly (Rft).



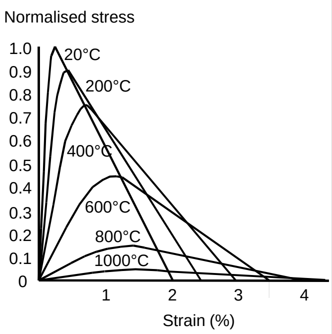
CONCRETE

CONCRETE STRESS-STRAIN CURVES

Concrete also loses strength and stiffness from 100°C upwards.

It does not regain strength on cooling.

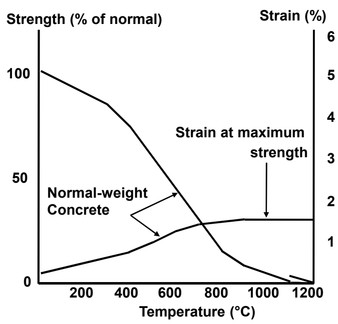
High temperature properties depend mainly on aggregate type used.



CONCRETE STRESS-STRAIN CURVES

Concrete strength reduction factors are:

Accurate for normal density concrete with siliceous aggregates.



THERMAL EXPANSION

THERMAL EXPANSION

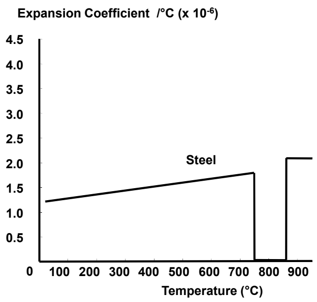
Thermal expansion is of concern only in Advanced Calculation Models

The basic effect of heating is thermal expansion.

Where, due to geometry or other factors, the expansion is constrained in some way, additional stresses are set up, which need to be added to mechanical stresses.

THERMAL EXPANSION DATA

Eurocode 3 specifies an accurate curve, with phase shift around 800°C.



'SIMPLE' METHOD 1
CRITICAL TEMPERATURE METHOD

EUROCODE STRATEGIES

Eurocode 3 allows three strategies for establishing fire resistance:

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Load resistance:	$R_{fi,d,t} > E_{fi,d}$	Feasible by hand calculation. Aim is to find reduced resistance at design temperature
Temperature:	$\theta_{cr,d} > \theta_d$	Simplest EC3 method. Find critical temperature for loading and compare with design temperature

EUROCODE STRATEGIES

Temperature: $\theta_{cr,d} > \theta_d$ → ***Find critical temperature for loading and compare with design temperature***

CRITICAL TEMPERATURE METHOD

[1] Calculate the loading on the structure at the fire limit state.

Analyse the structure to find moments and forces $E_{fi,d,t}$ at critical locations in the structure

This determines the 'Actions' on members

[2] Classify the section. The criterion is local buckling, which is characterised by the width/thickness ratio of elements

MEMBER CLASSIFICATION

For the following cases, the section classification used is the same as for ambient temperature.

- Compression members
- Simple beams supporting a concrete slab on the top flange
- All Class 3 and Class 4 sections

MEMBER CLASSIFICATION

The width/thickness ratios of flange and web elements should be less than the values shown in the table.

Element	Class 1	Class 2	Class 3
Flange	$c/t_f=10\varepsilon$	$c/t_f=11\varepsilon$	$c/t_f=15\varepsilon$
Compressed web	$d/t_w=72\varepsilon$	$d/t_w=83\varepsilon$	$d/t_w=124\varepsilon$
Web in bending	$d/t_w=33\varepsilon$	$d/t_w=38\varepsilon$	$d/t_w=42\varepsilon$

$$\varepsilon = \sqrt{\frac{235}{f_y}}$$

MEMBER CLASSIFICATION

For the following cases, the section classification used for the fire limit state is different from that at ambient temperature.

Tension members

Beams with exposure on all four sides

MEMBER CLASSIFICATION

For these members, the width/thickness ratios of the flange and the web elements should be less than the values shown in the same table but with:

$$\varepsilon = \sqrt{\frac{235}{f_y}\frac{k_{E,\theta}}{k_{y\theta}}} \quad = \quad 0.85\sqrt{\frac{235}{f_y}} \quad \text{(Approximately)}$$

$k_{E,\theta}$ = elastic modulus reduction factor for steel at temperature θ.

$k_{y\theta}$ = yield strength reduction factor for steel at temperature θ.

SIMPLE METHOD (1) - RESISTANCE

[3] Calculate the resistance of the cross-section at ambient temperature, but using the partial safety factors for the fire limit state, $R_{fi,d,20}$

[4] Calculate Utilisation Factor $\mu_0 = \frac{E_{fi,d,t}}{R_{fi,d,20}}$

$E_{fi,d,t}$ is the design loading of the member in fire, calculated in Step 1.

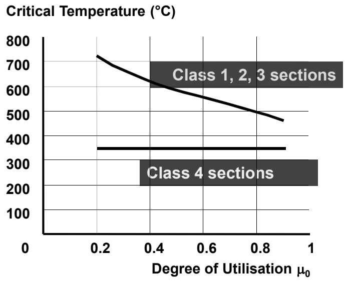
SIMPLE METHOD (1) - RESISTANCE

[5] Determine the Critical Temperature $\theta_{cr,d}$ using the Utilisation Factor

Non-slender sections (Classes 1, 2, 3) treated the same.

$$\theta_{cr} = 39.19 \ln\left[\frac{1}{0.9674\,\mu_0^{3.833}} - 1\right] + 482$$

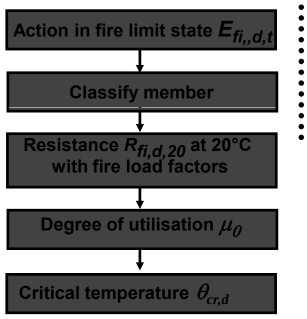
Slender (Class 4) sections treated conservatively (350°C).



SIMPLE METHOD (1)

FIRE RESISTANCE

STEEL TEMPERATURE



INCREASE OF TEMPERATURE IN UNPROTECTED MEMBERS

SIMPLE METHOD (1) - TEMPERATURE

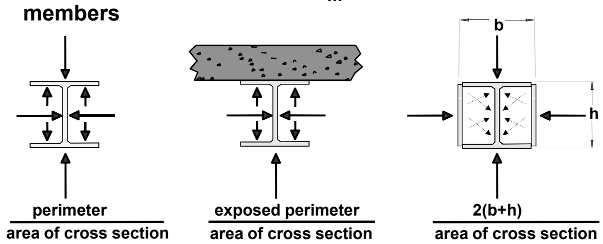
[6] Determine the Section Factor A_m/V (= H_p/A) needed for the calculation of the steel temperature

A_m = exposed surface area of member per unit length
= perimeter (H_p)

V = volume of member per unit length
= cross-section area (A)

SIMPLE METHOD (1) - TEMPERATURE

Definition of Section Factor A_m/V for unprotected members

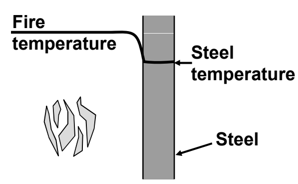


SIMPLE METHOD (1) - TEMPERATURE

[7] The temperature increase in steel is calculated for a small time step, assuming that all steel is at the same temperature

Temperature increase in time step Δt:

$$\Delta\theta_{a,t} = \frac{1}{c_a\,\rho_a}\frac{A_m}{V}\,h_{net,d}\,\Delta t$$



$h_{net,d}$ = Heat flux

c_a, ρ_a = specific heat and density of steel

HEAT FLUX

The heat flux has two parts - one due to radiation and the other due to convection.

Heat flux

$h_{net,d} = h_{net,r} + h_{net,c}$

HEAT FLUX - RADIATION

Heat flux due to radiation

$$h_{net,r} = 5.67 \times 10^{-8} \Phi \varepsilon_{res} \left[(\theta_r + 273)^4 - (\theta_m + 273)^4 \right]$$

Φ = configuration factor
(can be set to 1.0 in the absence of data)

θ_r, θ_m = environment and member surface temperatures

EMISSIVITY

ε_{res} = resultant emissivity
= $\varepsilon_f \times \varepsilon_m$
(can be taken as 0.5 in the absence of data)

ε_f = emissivity of the fire compartment
(can be taken as 0.8 in the absence of data)

ε_f = emissivity of the steel surface
(can be taken as 0.625 in the absence of data)

HEAT FLUX - CONVECTION

Heat flux due to convection

$h_{net,c} = \alpha_c (\theta_r - \theta_m)$

α_c = convective heat transfer coefficient
(NA value)
25W/m²°K for Standard or External Fire
50W/m²°K for Hydrocarbon Fire

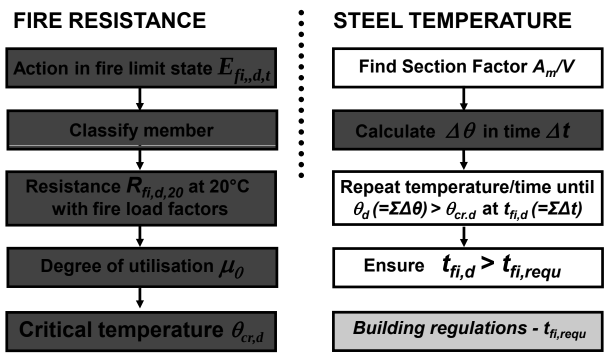
SIMPLE METHOD (1) - TEMPERATURE

[8] In the initial time steps, clearly θ_r is less than the critical temperature $\theta_{cr,d}$

The time required $t_{fi,d}$ for the steel temperature to exceed the critical temperature is evaluated by repeating step [7]

The time $t_{fi,d}$ is compared with time $t_{fi,req}$ specified in building regulations for fire safety. For safety, time $t_{fi,d}$ should, naturally, be greater than the time $t_{fi,req}$

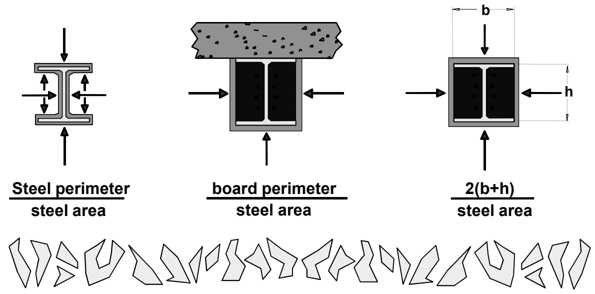
SIMPLE METHOD (1) - SUMMARY



INCREASE OF TEMPERATURE IN PROTECTED MEMBERS

PROTECTED STEEL

Definition of Section Factor A_p/V for protected members



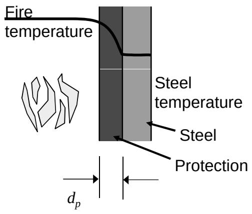
TEMPERATURE - PROTECTED STEEL

The temperature increase in steel is affected by the heat stored in the protection layer

Heat stored in protection layer

$$\phi = \frac{c_p \rho_p}{c_a \rho_a} d_p \frac{A_p}{V}$$

Temperature increase in time step Δt:



$$\Delta\theta_{a,t} = \frac{\lambda_p / d_p}{c_a \rho_a} \frac{A_p}{V} \left(\frac{1}{1+\phi/3} \right) (\theta_{g,t} - \theta_{a,t}) \Delta t - \left(e^{\phi/10} - 1 \right) \Delta\theta_{g,t}$$

OTHER PARAMETERS

θ_g, θ_a = environment and member surface temperatures

c_p, ρ_p = specific heat and density of protection material

d_p = thickness of protection material

λ_p = thermal conductivity of protection material

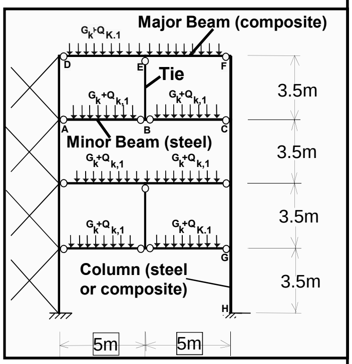
EXAMPLE UNPROTECTED TENSION MEMBER

WORKED EXAMPLE - TIE MEMBER

Fire resistance of a structural tie in a building frame

(Required time is 60min)

Ties - steel



TIE MEMBER - FIRE LIMIT STATE

Utilisation factor:

(4.2.4) μ_0 = $N_{fi,d,t} / N_{fi,20,Rd}$

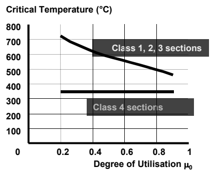
= 114.0 / 283.25

= 0.40

Critical temperature (Class 1 element):

(Table 4.1) θ_c = 619°C

(Graph alongside)



TIE MEMBER – STEP 1

Summary of First Step of Calculations

θ_m = 20.00 ° C for steel, initially

θ_r = 96.54 ° C for the room after 5 sec

Increase in temperature Δθ = 0.92 °C

New Steel Temperature, θ_m = 20.00+0.92 = 20.92 °C

TIE MEMBER – STEP 2

Summary of Second Step of Calculations

θ_m = 20.92 ° C for steel

θ_r = 146.95 ° C for the room after 10 sec

Increase in temperature Δθ = 1.57 °C

New Steel Temperature, θ_m = 20.92+1.57 = 22.49 °C

TIE MEMBER - FIRE LIMIT STATE

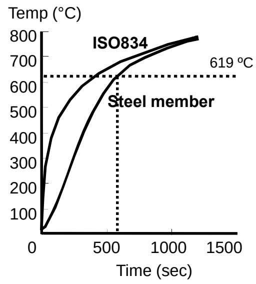
The calculations are repeated for each Δt = 5 sec period.

It can be shown that the time required for the steel tie to reach the critical temperature of 619ºC is 9min 40 sec

Clearly, this is less than the required duration of 60 min.

(Not OK)

The tie will have to be protected.



EXAMPLE
PROTECTED TENSION MEMBER

TIE MEMBER - FIRE PROTECTION

It is given that 60 minutes' fire protection required.

Protection against fire will be assessed for encasing it with 20mm thick Gypsum board.

Thermal properties of Gypsum are:

Density	ρ_p	=	800 kg/m^3
Specific heat	c_p	=	1700 J/kg°K
Thermal conductivity	λ_p	=	0.2 W/m°K
Section factor	A_p/V	=	300.97 m^{-1}

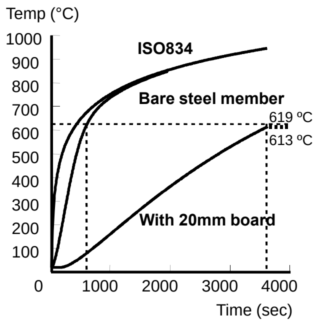
TIE MEMBER - FIRE PROTECTION

Steel temperature is again calculated for increments of time of Δt = 5 sec.

At 60 min, the steel temperature obtained is 613°C, which is just below the critical temperature of 619°C.

Thus, 20mm gypsum boarding provides the required 60 min fire protection.

(OK)



SUMMARY

SUMMARY SO FAR

Eurocode 3 allows three strategies for establishing fire safety of protected and unprotected steel structures.

The simple method, based on critical temperatures, has been described in detail.

METHOD 2
STRENGTH UNDER FIRE

EUROCODE STRATEGIES

Load resistance: $R_{fi,d,t} > E_{fi,d}$ → ***Feasible by hand calculation. Aim is to find reduced resistance at design temperature***

UNRESTRAINED BEAMS

UNRESTRAINED BEAMS

The lateral-torsional buckling moment capacity is checked for the maximum temperature $\theta_{a,com}$ at the compression flange:

$$M_{b,fi,t,Rd} = W_{pl,y}\ k_{y,\theta,com}\ f_y\ \chi_{LT.fi}\ \frac{1}{\gamma_{M.fi}}$$

$W_{pl,y}$ is the plastic section modulus of the cross-section *(assuming Class 1 or Class 2 section)*.

The reduced yield strength of the compression flange is defined as $k_{y,\theta,com} f_y$ at $\theta_{a,com}$

UNRESTRAINED BEAMS

$\chi_{LT,fi}$ is the lateral-torsional buckling strength reduction factor in fire design situation.

The strength reduction factor $\chi_{LT.fi}$ for flexural buckling is calculated using normalised slenderness :

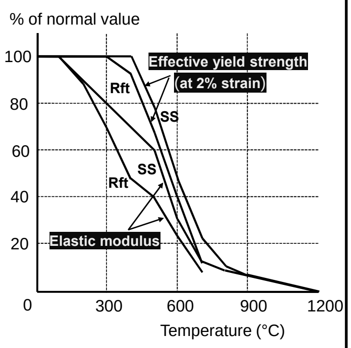
$$\bar{\lambda}_{LT,\theta,com} = \bar{\lambda}_{LT} \sqrt{k_{y,\theta,com} / k_{E,\theta,com}}$$

Lateral-torsional buckling need not be considered if:

$$\bar{\lambda}_{LT,\theta,com} < 0.4$$

EC3 STEEL STRESS-STRAIN CURVES

Graphs to be used for $k_{E,\theta}$ *and* $k_{y\theta}$



RESTRAINED BEAMS

RESTRAINED BEAMS

Restrained beams do not exhibit lateral-torsional buckling. Moment capacity is found by using:

$$\chi_{LT.fi} = 1$$

COLUMNS

COMPRESSION MEMBERS

This information applies to Class 1, 2 or 3 sections. The axial buckling resistance is checked for the maximum temperature $\theta_{a,com}$ in the steel:

$$N_{b,fi,t,Rd} = A\, k_{y,\theta,max}\, f_y\, \chi_{fi}\, \frac{1}{\gamma_{M.fi}}$$

χ_{fi} is the axial buckling strength reduction factor in fire design situation, obtained from curve 'c'.

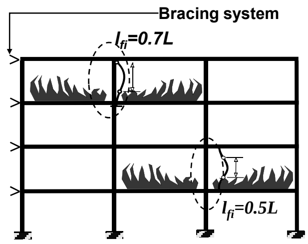
The reduced yield strength of the compression flange is defined as $k_{y,\theta,com} f_y$ at $\theta_{a,com}$. *The c*orrection factor of 1.2 simply allows for uncertainties.

COMPRESSION MEMBERS

The strength reduction factor $\chi_{LT,fi}$ for axial buckling is calculated using normalised slenderness :

$$\bar{\lambda}_{\theta,max} = \bar{\lambda}\sqrt{k_{y,\theta,max} / k_{E,\theta,max}}$$

The effective length for buckling in the fire design situation may be adopted as shown in the diagram



SUMMARY

SUMMARY SO FAR

Eurocode 3 allows three strategies for establishing fire safety of protected and unprotected steel structures.

Formulae to be used for the 'simple' method based on calculation of strength for a given temperatures have been briefly mentioned.

EXAMPLE – COMPOSITE FLOOR

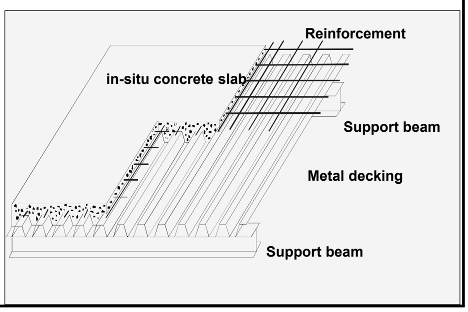
COMPOSITE FLOORS

A composite slab comprises three main components

Profiled steel decking

Some reinforcement

Cast-in-situ concrete



COMPOSITE FLOORS

Composite floor slabs offer advantages over other systems

- speed and simplicity of construction
- safe working platform protecting workers below
- lighter than traditional concrete flooring
- often used with lightweight concrete, which further reduces the dead load

STRENGTH CRITERION "R"

MOMENT CAPACITY METHOD

The temperature distribution through the beam at the appropriate fire resistance period is required.

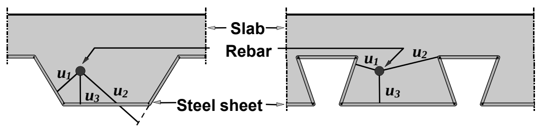
Using the temperature data, the modified strength properties of the materials in the section at these elevated temperatures are determined. Use is made of strength reduction factors.

The reduced strengths of the materials are then used to determine the moment capacity of the member at critical sections.

STEEL TEMPERATURE

In EC4, the "effective distance parameter z" is calculated from the formula:

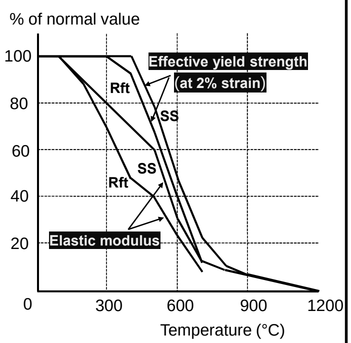
$$\frac{1}{z} = \frac{1}{\sqrt{u_1}} + \frac{1}{\sqrt{u_2}} + \frac{1}{\sqrt{u_3}}$$



STEEL STRENGTH

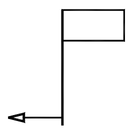
The strength of the steel reinforcement is determined using the appropriate material strength reduction factor $k_{y\theta}$

(Rft for reinforcement)



SAGGING MOMENT RESISTANCE

The sagging moment capacity is determined using stress blocks, just as for reinforced concrete beam or slab sections

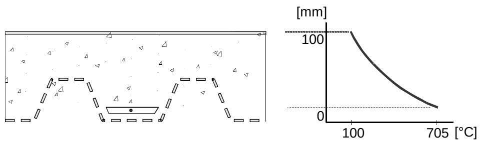


NEGATIVE MOMENT OF RESISTANCE

This is calculated by Numerical Integration

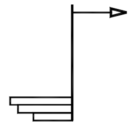
The ribbed zone is divided into thin slices.

The temperature is determined at the centroid of each trapezium.



HOGGING MOMENT RESISTANCE

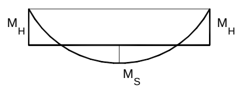
The hogging moment capacity is determined using stress blocks, just as for reinforced concrete beam or slab sections, except that in this case there exist a number of thin stress blocks with varying concrete strength.



Moment capacity is calculated by taking moments about the steel reinforcement

MOMENT CHECK – INTERNAL SPAN

For an internal span it may be assumed that the span is continuous over beams.

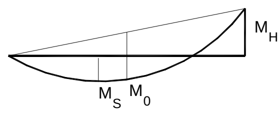


Thus $M_H + M_S \geq M_0$

Where, M_0 is the free bending moment (factored for the fire limit state).

MOMENT CHECK – EXTERNAL SPAN

For an external span it may be assumed that the span is continuous over the internal beam only. Thus



Where, M_0 is the free bending moment (factored for the fire limit state).

METHOD 3
ADVANCED CALCULATION MODELS

ADVANCED CALCULATION MODELS

Eurocode 3 has one page and a half on this method, with no equations or formulae.

The principles to be followed are specified.

The user can choose their own method, as long as the fundamental behaviour is modelled leading to a reliable approximation of the expected behaviour under fire conditions.

Separate calculations for thermal and mechanical response are necessary.

ADVANCED CALCULATION MODELS

Any heating curve (fire) may be used.

Variation of thermal properties should be as given in the standard.

Thermal response should follow established theory of heat transfer.

Mechanical response should follow established theory of structural mechanics.

ADVANCED CALCULATION MODELS

Effect of mechanical actions, geometrical imperfections and thermal actions should all be combined.

Verification of calculation should be made on the basis of relevant test results.

OUTLINE

Advanced Calculation Models for
- Analysis for Temperature Distribution
- Analysis for Strength

Computer Modelling for Temperature Distribution
- Program TASEFplus

Computer Modelling for Strength
- Program COMPSEFplus

Conclusion

ADVANCED CALCULATION MODELS
HEAT FLOW ANALYSIS

HEAT FLOW ANALYSIS

Consideration of heat conduction, convection and radiation

Use of accurate material properties

Appropriate modelling of boundary conditions

HEAT FLOW ANALYSIS

The basic heat conduction equation in two dimensions is:

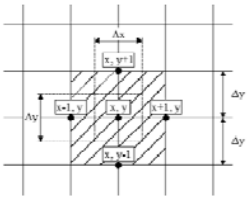
$$\frac{\partial^2 T}{\partial x^2} + \frac{\partial^2 T}{\partial y^2} = \frac{\rho c}{k}\frac{\partial T}{\partial t}$$

The equation can be solved by the Finite Element Method or by the Finite Difference Method

HEAT FLOW ANALYSIS

The finite difference form of the Heat Flow equation at a typical point:

$$T_{x,y}^{p+1} = \frac{\alpha \Delta t}{(\Delta x)^2}\left(T_{x+1,y}^{p} + T_{x-1,y}^{p} + T_{x,y+1}^{p} + T_{x,y-1}^{p}\right) + \left[1 - \frac{4\alpha \Delta t}{(\Delta x)^2}\right] T_{x,y}^{p}$$



(Explicit form)

(Depends upon Critical Time Increment ***Δt***)

HEAT FLOW ANALYSIS

Several Computer programs are available for doing this task.

SAFIR
VULCAN
ANSYS
ABAQUS

Plus many others

PROGRAM TASEF

PROGRAM TASEF

TASEF is a program written in FORTRAN and has a line-by-line data input. It was developed by Ulf Wickström from Sweden.

Program TASEF was first published in 1979 and was later enhanced in 1990.

It handles heat flow in two-dimensional and axi-symmetric cross-sections exposed to fire temperatures.

PROGRAM TASEF

A variety of boundary conditions can be applied to surfaces of the section. These include not only surfaces subjected to heat flux as in a fire but also those in which there is no gain or loss of heat and there is no change in entropy (Adiabatic).

Surfaces can also be without exposure to external heat.

TASEF can also handle multiple fires in the same problem.

PROGRAM TASEF

Material properties required are specific heat conductivity and specific heat capacity of the material.

Latent heat of water is used to handle problems involving moist materials such as concrete.

PROGRAM TASEFplus

PROGRAM TASEFplus

TASEFplus, written in Visual Basic, is a pre-processor and a post-processor for TASEF. It was developed by the author while at City University London.

TASEFplus includes material properties as specified in Eurocodes.

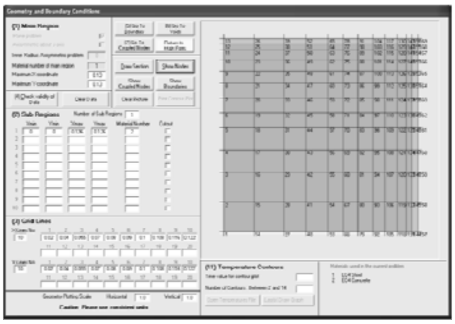
The next few slides show the key features of TASEFplus. All the input can be completed with visual interface for data integrity.

PROGRAM TASEFplus

The Geometry is defined by specifying:

Outer dimensions,
Sub-regions, which can be voids, and
A few grid-lines for making the Finite Element mesh.

PROGRAM TASEFplus



PROGRAM TASEFplus

The Boundary Conditions are defined by specifying:

Node groups

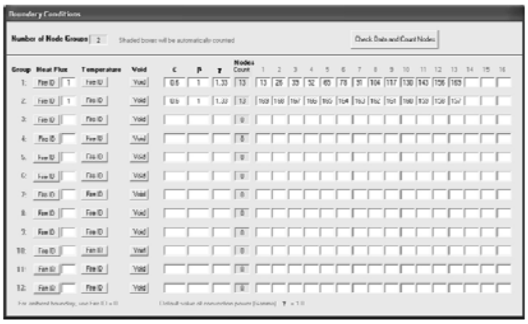
For each Node Group:
The type of boundary condition that applies
Fire with Heat Flux,
Adiabatic (Fire without Heat Flux), and
Ambient (No Fire).

PROGRAM TASEFplus

Internal voids can be specified.

For example, 'Hat Sections' used in Scandinavia can be modelled.

PROGRAM TASEFplus

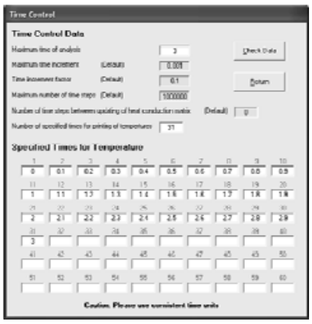


PROGRAM TASEFplus

The temperature-time history specifies the times at which the calculations are output.

In fact, the time domain has an explicit idealisation, so the calculations are carried out for a 'critical' time step for accuracy and convergence.

PROGRAM TASEFplus

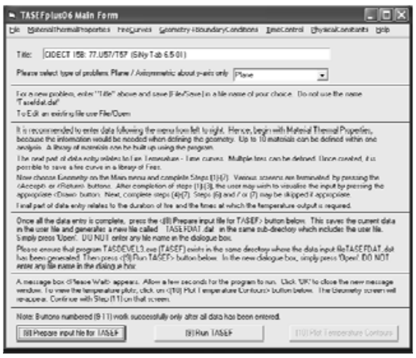


PROGRAM TASEFplus

All the instructions required are given on the front page of the program.

(The program also comes with a user manual)

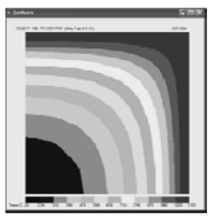
PROGRAM TASEFplus



PROGRAM TASEFplus

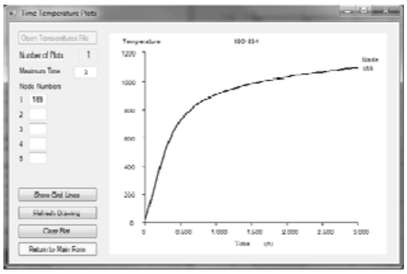
The output is in the form of a fully annotated ext file.

In addition, colour contours of temperature distribution can be plotted.



PROGRAM TASEFplus

The temperature history of one or more selected nodes can be plotted.



ADVANCED CALCULATION MODELS

STRENGTH ANALYSIS FOR FIRE

STRENGTH ANALYSIS FOR FIRE

Use of accurate material properties, including thermal expansion, at high temperatures

For columns, second-order effects and imperfections need to be included

Requires computer software for any practical calculations.

Permitted by Eurocodes

STRENGTH ANALYSIS FOR FIRE

Advanced Calculation Models allow proper modelling of cross-section geometry, including benefits from concrete in floor slab acting as heat sink

Numerical analysis can be done by using the Finite Element Method or the Finite Difference Method

FINITE ELEMENT METHOD

FINITE ELEMENT METHOD

Most general-purpose Finite Element Programs are not directly useable for strength analysis of structures exposed to fire.

There is a very small number of Finite Element programs specifically developed for fire analysis.

FINITE ELEMENT METHOD

Most Finite Element programs have a steep learning curve.

Practising engineers find the time required to create the data model for such programs unacceptably long.

FINITE DIFFERENCE METHOD

FINITE DIFFERENCE METHOD

Finite Difference based computer programs are only able to solve specific types of problems, for example, columns or plates.

For their specific application, such programs are fast and the learning curve is very short, principally because the amount of data required is not large.

COLUMN ANALYSIS FOR FIRE

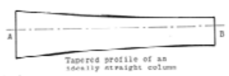
FINITE DIFFERENCE METHOD

Finite Difference based computer programs are only able to solve specific types of problems, for example, columns or plates.

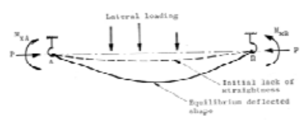
For their specific application, such programs are fast and the learning curve is very short, principally because the amount of data required is not large.

COLUMN MODEL

Non-uniform profile along the length



Temperature dependent material properties



Lateral loading

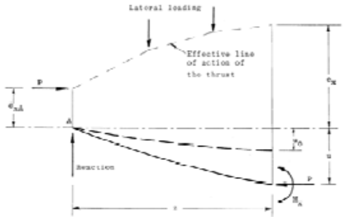
Initial imperfections.

COLUMN MODEL

Column Equilibrium equation

$M_x = P (e_x + u)$

This is a non-linear equation.



CALCULATION MODEL

Solution of two sub-problems

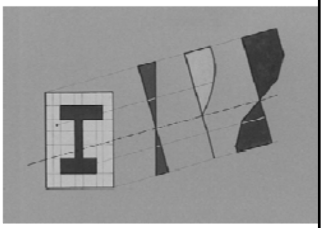
Internal Equilibrium
Evaluation of stress resultants using numerical integration (finding P and M)

External Equilibrium
Calculation of deflections using finite differences and second-order iteration (finding u)

$[M_x = P (e_x + u)]$

INTERNAL EQUILIBRIUM

Stresses need to be integrated over appropriate areas, using non-linear temperature dependent stress-strain relations, to satisfy internal equilibrium



Numerical Methods such as Gauss Integration are used

EXTERNAL EQUILIBRIUM

The equilibrium deflected shape is determined by the finite difference method combined with the Newton-Raphson method of iteration.

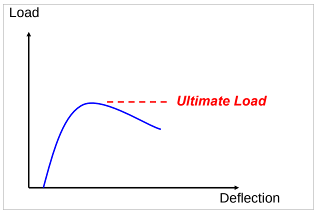
Thus, starting with an approximate solution $\{ u^k \}$ for the deflections at the finite difference stations, a better solution is obtained by:

$$\{ u^{k+1} \} = \{ u^k \} - [I - K]^{-1} \{ u^k - U^k \}$$

[K] is determined numerically

STABILITY ANALYSIS

The method described is applied repeatedly, starting with a small applied load and solving for the deflected shape, and then increasing the load until no convergence for the deflected shape is obtainable.



The maximum load for which convergence is obtained is taken as the ultimate strength of the column.

PROGRAM COMPSEFplus

PROGRAM COMPSEFplus

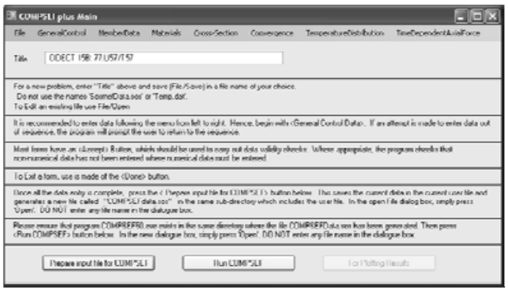
COMPSEF is a program written also in FORTRAN. COMPSEFplus, written in Visual Basic, is a pre-processor for COMPSEF.

Both programs were developed by the author while at City University London.

The next few slides show the key features of the program. All the input can be completed in a very short time.

PROGRAM COMPSEFplus

As for TASEFplus, all the instructions necessary to analyse a problem appear on the first screen.



PROGRAM COMPSEFplus

The next screen defines what kind of analysis is being performed, whether it is uniaxial or biaxial bending.

A beam is a column with zero axial load.

PROGRAM COMPSEFplus

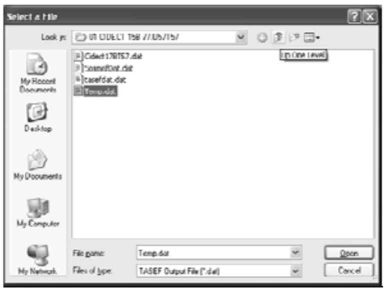
The subsequent screen defines the column length and few other parameters, which are often default values.

PROGRAM COMPSEFplus

Geometry is mapped by defining quadrilaterals and the applicable material.

PROGRAM COMPSEFplus

Temperature output file from TASEF is read directly.



PROGRAM COMPSEFplus

Even though the geometry may be defined in different ways, full mapping of TASEF geometry to COMPSEF geometry is automatic.

The output is fully annotated text file with deflections of the structure.

PROGRAM COMPSEFplus

```
Axial load              = -587000.000000
Time                    =      49.050000
Time increment          =       0.050000
Axial displacement      =       1.378246
X convergence norm      =       0.000000
Y convergence norm      =   -1536.013848

----------------------------------------
Station   X-Deflection   Y-Deflection
-------   ------------   ------------
    1        0.000000       0.00
    2        0.000000      -1.19

   15        0.000000    -101.19
   16        0.000000    -102.22
   17        0.000000    -101.09
-----------------------------------------
 FAILURE TIME            =      49.05
-----------------------------------------
```

APPLICATION TO CONCRETE FILLED STEEL TUBES

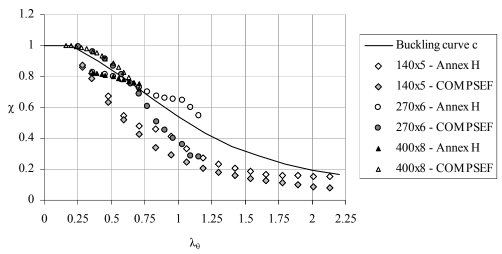
APPLICATIONS

If one looks at Eurocode 4, the design method for concrete filled tubes in Annex H is less than satisfactory.

This has been demonstrated by running COMPSEFplus.

Results from Annex H are higher than those given by Advanced Calculation Models. While this can mean economical designs using Annex U, it could also mean unsafe designs

APPLICATIONS



REFERENCE

Nyman, S and Virdi, K S
Fire Response Of Concrete Filled Hollow Steel Sections
EUROSTEEL 2011, August 31 - September 2, 2011, Budapest, Hungary

APPLICATION
SHADOW EFFECT

SHADOW EFFECT

In fire engineering, it is recognised that the radiative heat exchange in unprotected steel I-sections is reduced due to geometric effects.

Most computations are based on the assumption that an I-section receives heat from convection and radiation uniformly over the entire surface area.

Since radiation is directional, the reduction in the heat transferred actually occurs because flanges cast a 'shadow' on the rest of the section.

SHADOW EFFECT

Ignoring the shadow effect leads to conservative results.

However, one consequence is that a given design may thus become uneconomic.

Based on some of the work of co-author Ulf Wickström, a much simplified approach appears in the fire engineering part of Eurocode 3 for steel structures.

SHADOW EFFECT

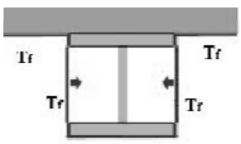
As an example of the Advanced Calculation Model, temperature distributions in steel beams with or without considering the shadow effect are evaluated in a more rigorous manner.

The effect of reduced temperatures obtained on the fire duration from TASEF is later evaluated using the finite difference based program COMPSEF.

MODELLING SHADOW EFFECT

The shadow effect is considered by introducing an artificial boundary on the 'open' sides of the I section, that is, in the space between the flanges

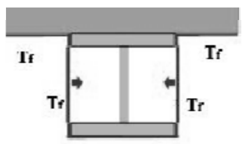
The introduced boundaries follow the same fire curve as the rest of the section.



MODELLING SHADOW EFFECT

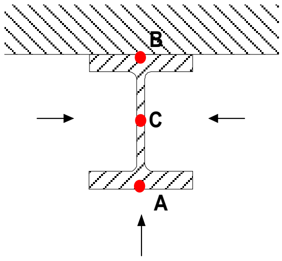
The key parameter of this artificial boundary is that it is specified to be adiabatic.

Thus, while following the fire curve, it does not cause radiation to pass through, thus introducing a shadow.



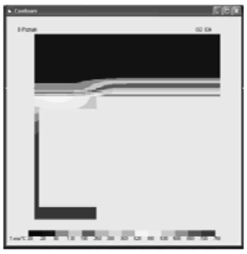
TEMPERATURE DISTRIBUTIONS AT 30 min

Points selected for comparison of temperatures

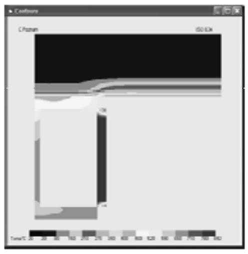


TEMPERATURE DISTRIBUTIONS AT 30 min

No Shadow Effect

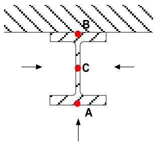


With Shadow Effect



MODELLING SHADOW EFFECT

Position	Eurocode 3	No Shadow Effect	With Shadow Effect
A	827`	709	661
B	827`	544	510
C	827	714	666



Rigorous analysis gives lower temperatures than EC3.

Consideration of Shadow effect gives further reduction in temperatures.

MODELLING SHADOW EFFECT

The span of the beam is taken as 3m. The beam is subjected to a uniformly distributed gravity load of 300kN/m.

As there is no composite action between the beam and the concrete above, this analysis considers the steel section alone.

MODELLING SHADOW EFFECT

Results from COMPSEF are given below.

No Shadow Effect	With Shadow Effect
30.0min	35.4min

This is a significant gain in fire duration for a bare steel section.

CONCLUSIONS

CONCLUSIONS

Eurocode 3 methodologies have been described.

The basis of "Advanced Calculation Models" as permitted in Eurocodes was covered.

Use of Advanced Calculation Models requires computer programs.

CONCLUSIONS

Two programs - one for heat transfer and one for mechanical response - have been described.

These programs are not 'general purpose', but dedicated to solving specific range of problems.

Consequenly, what they can do, can be done with limited input, results are fast to obtain, and the 'learning curve' is not too steep.

CONCLUSIONS

The programs can be used for design, research, and for teaching.

These programs have indeed been used in Fire Engineering modules for Master's programmes at City University and at Luleå University of Technology in Sweden.

CONCLUSIONS

One application has described how the shadow effect can be taken into account in determining the temperature distributions in a steel beam exposed to fire.

The reduction in temperatures obtained by an advanced calculation method, using the program TASEF, is first due to the transmission of heat into the concrete slab supported by the beam, a feature not taken into account in Eurocode 3.

CONCLUSIONS

Further significant reductions in temperatures are obtained by considering the shadow effect.

The resulting improved structural performance, calculated by the finite difference based program COMPSEF, is reflected in the increase in time to failure.

This difference could be significant in many practical situations.

THANK YOU