FIRE DESIGN OF HOLLOW CORE SLABS

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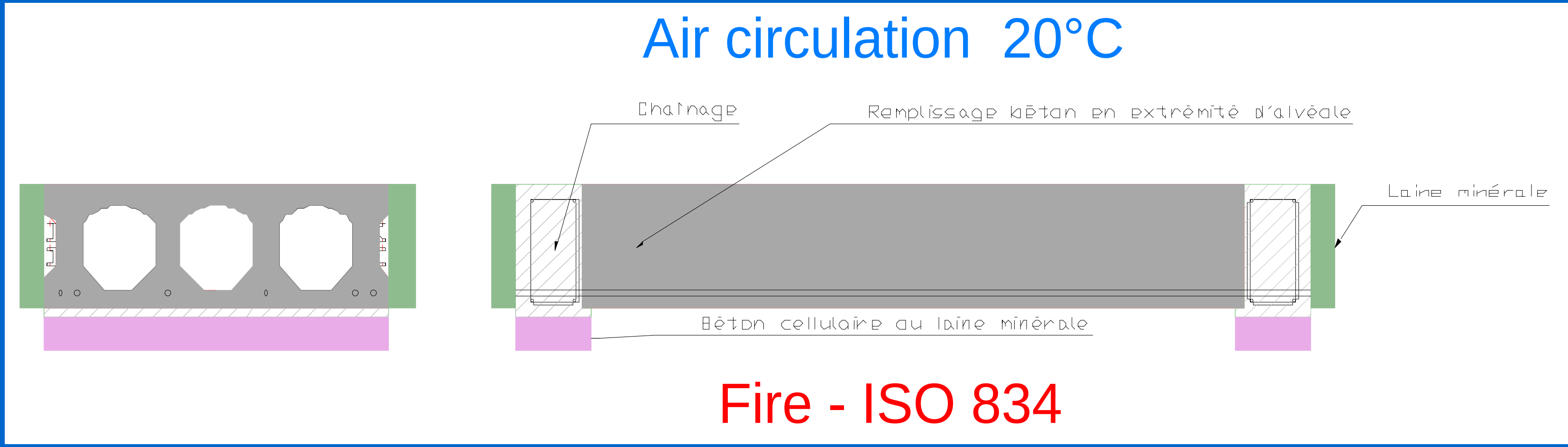
Plan

- **calculation of the temperature field on supports and at mid-span**

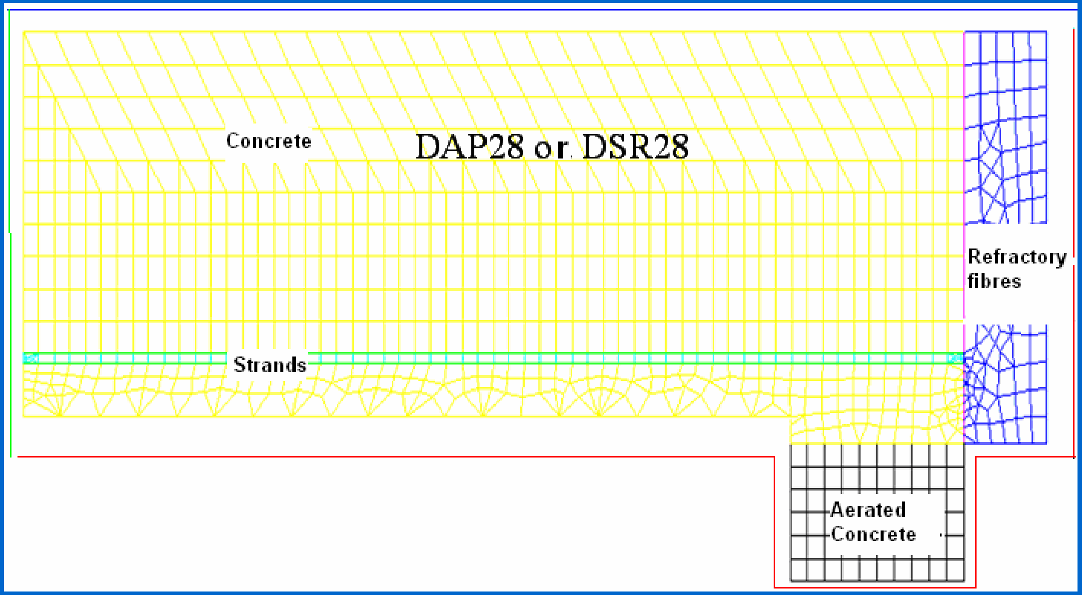
- **shear resistance model**
  - **Interaction model**
  - **Simplified shear-flexure model**
  - **Validation and results**

Temperature along the strand: midspan and support

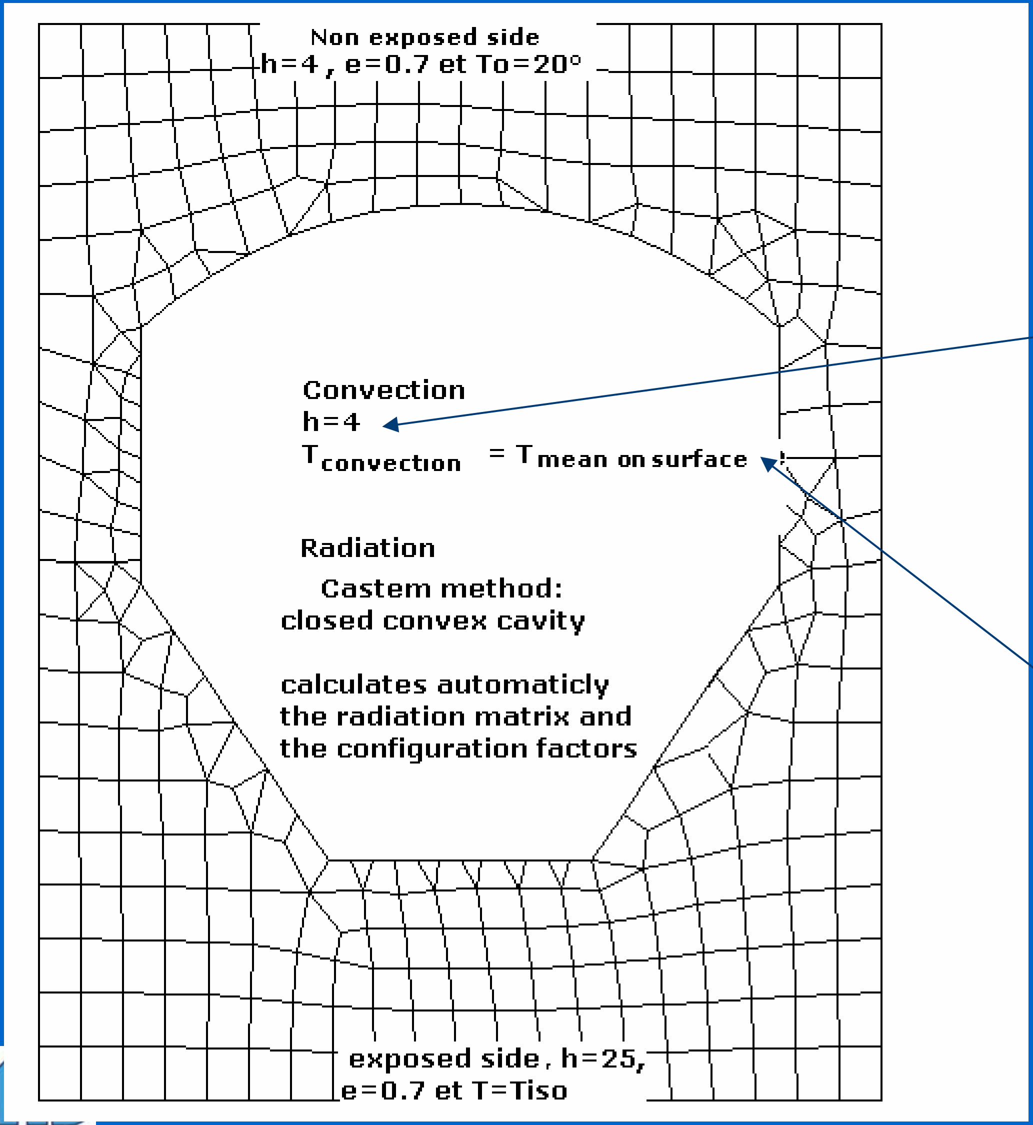


Fire test in CERIB
(up side down)

FEM thermal
model



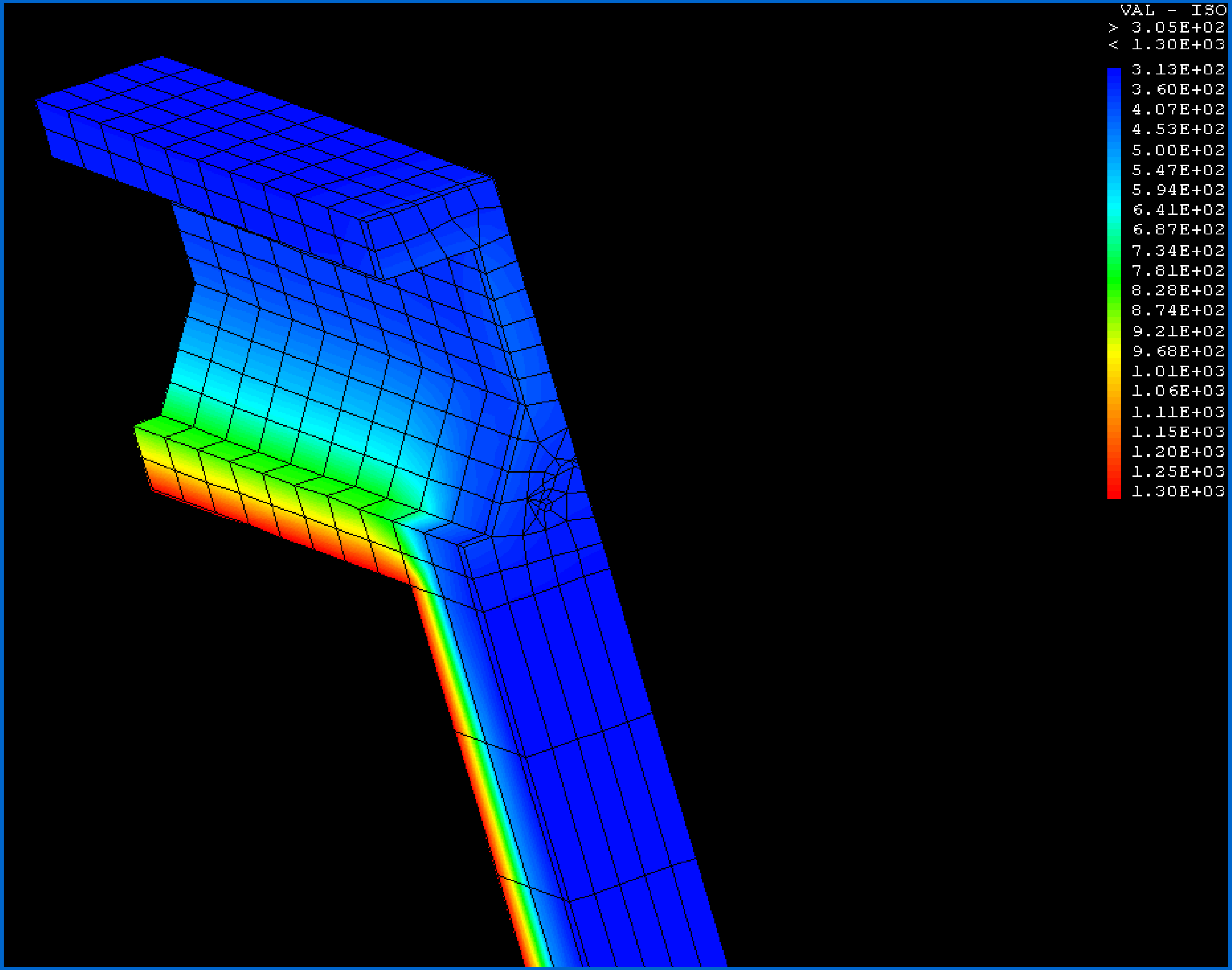
Boundary conditions for thermal model derived from EC2-1-2



The convective heat transfer coefficient in the cavity is the same that for non exposed surface

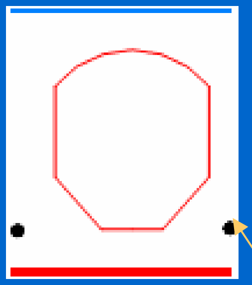
The heat capacity of air is neglected

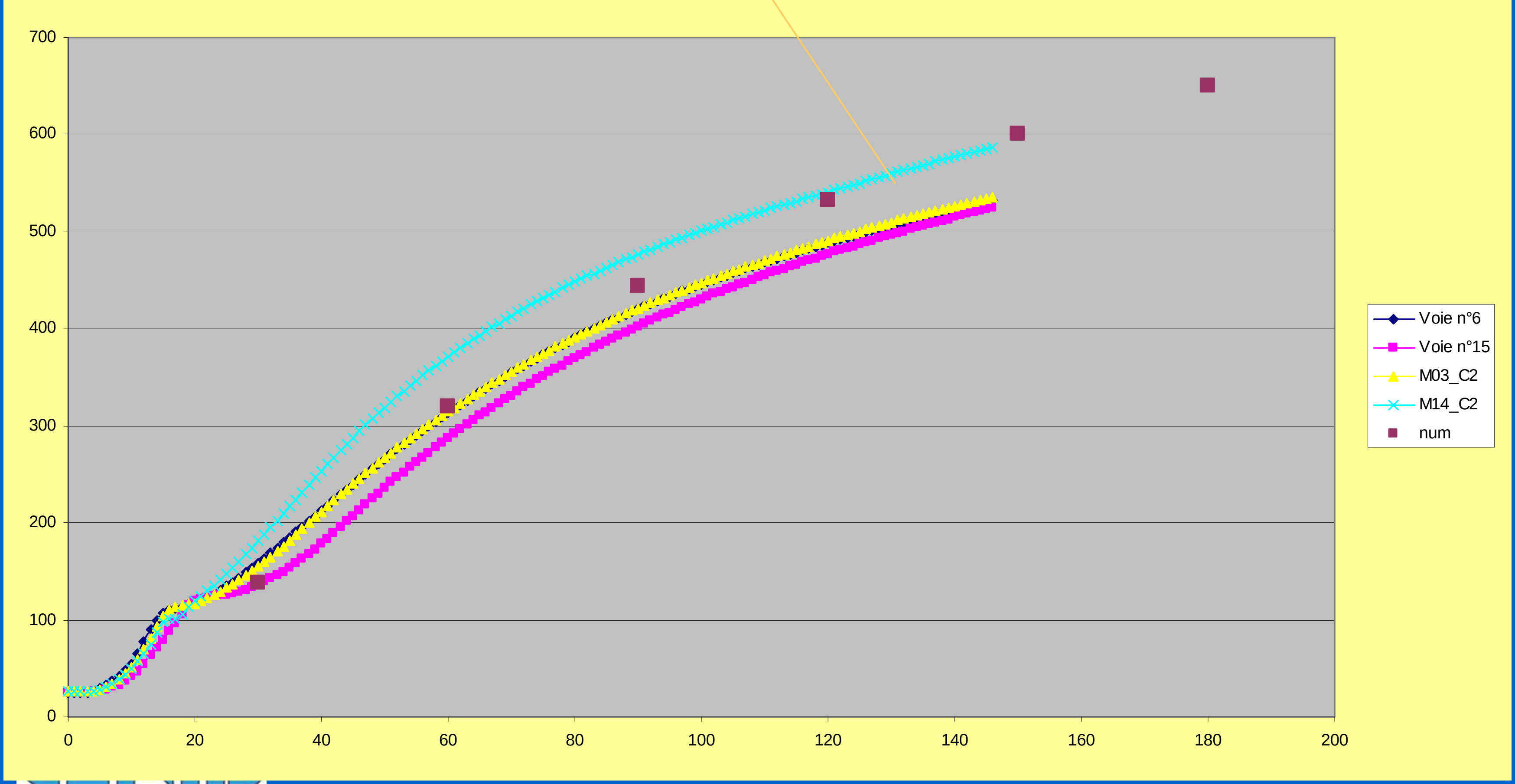
Thermal calculation



3D-temperature (°K) after 2h of Iso-834 fire curve

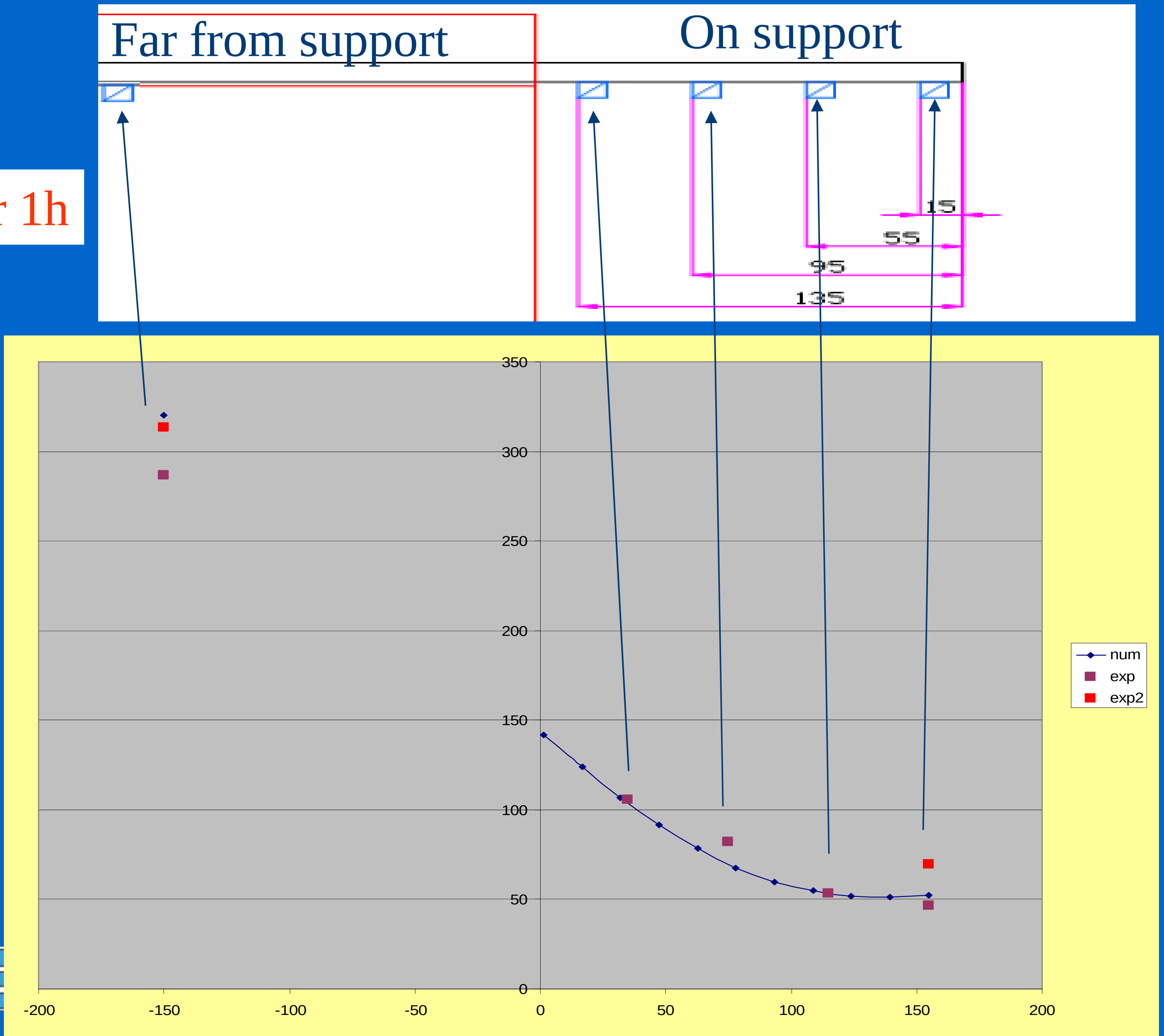
Strands temperatures





Temperature along the strand

After 1h



Calculation method

for shear strength at high temperature

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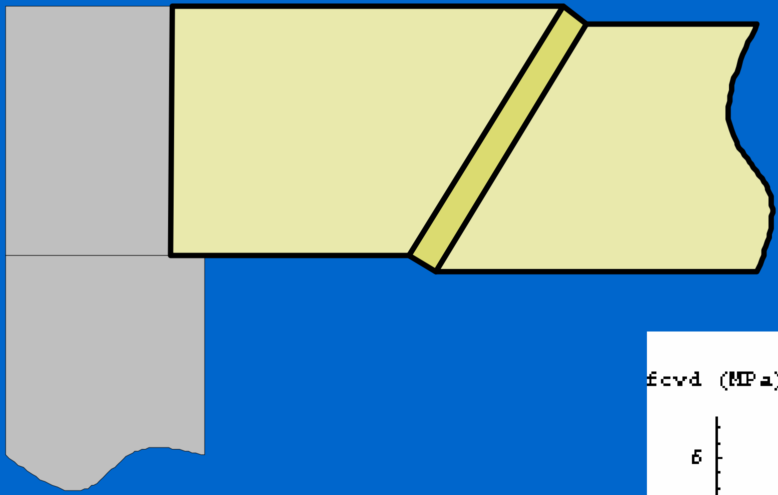
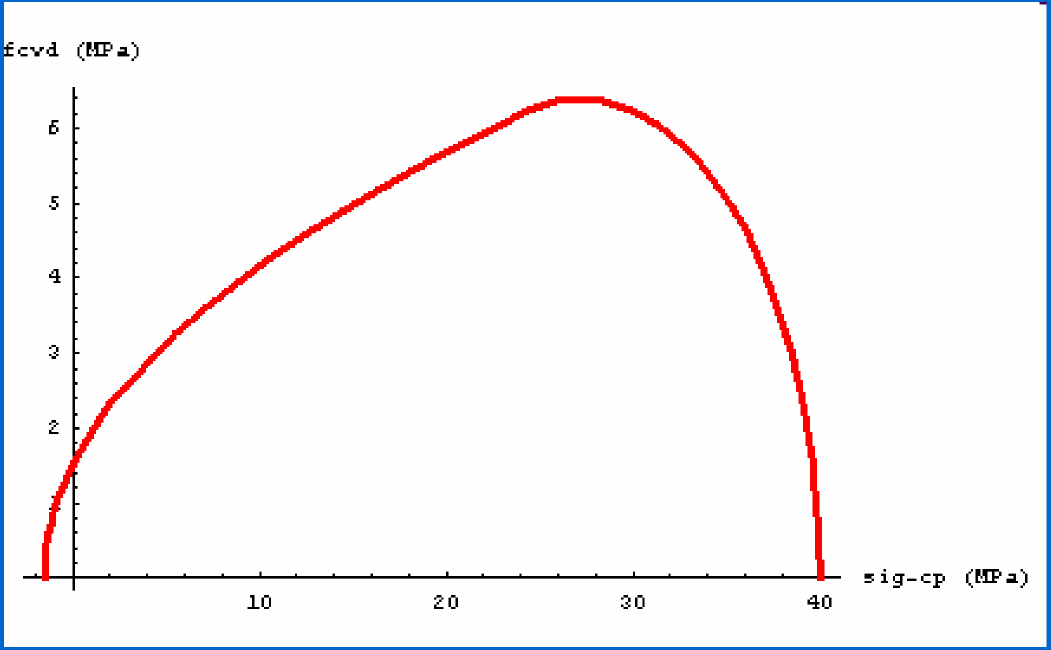
Interaction model

Calculation of a « upper bound » value of the shear strength :
Integration of the shear strength along the assumed failure surface



$$V_{R,\max} = \iint f_{cvd}$$



Shear strength – normal stress interaction curve given by EC2-1-1 §12

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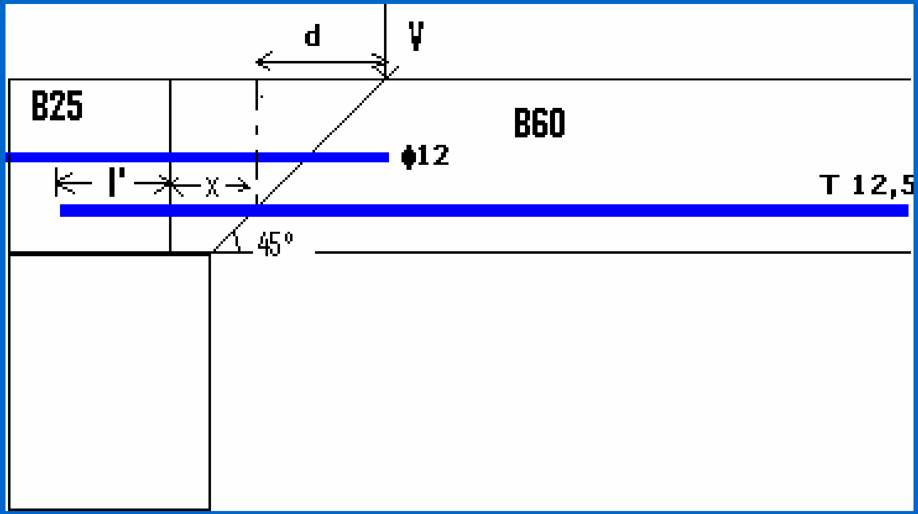
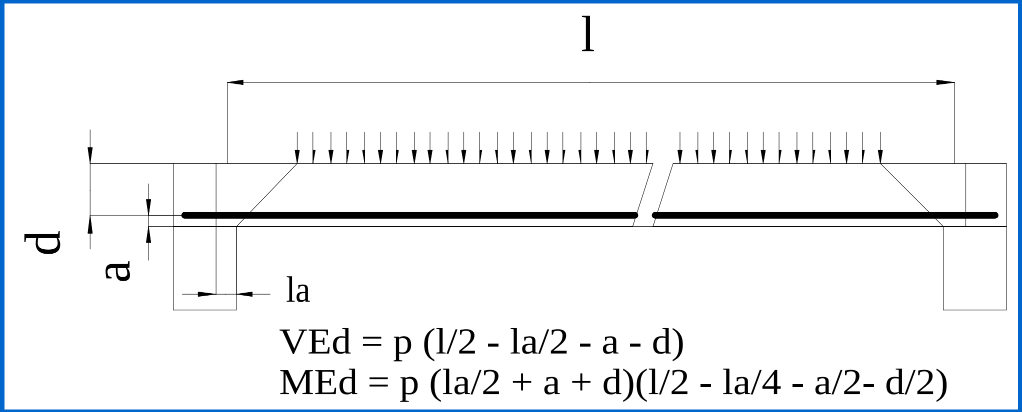
Interaction model

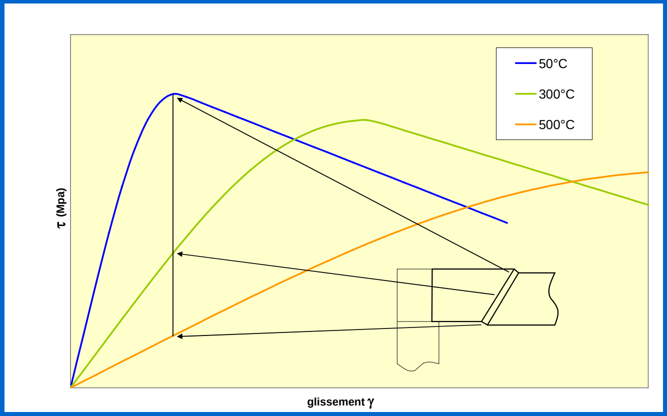




- Assume a 45° failure surface
- Calculate the bond/anchorage capacity of reinforcement
- Calculate the longitudinal stress field equilibrating M_{Ed} and thermal strain (1st step with a reduced load, include shift rule)
- Calculate $V_{R,fi}$ by integration of the interaction curve
- Adjustment of the load level: $V_{Edfi} = V_{R,fi}$
- Iterations until convergence

Interaction model

Compatibility of shear strain : Kinematic factor



$$V_{R,\max} = \iint \frac{f_{cvd,i}}{k_{cvd,i}} ds_i$$

$$k_{cvd,i} = \frac{\gamma_i}{\gamma_{\min}} = \frac{\frac{f_{cvd,i}}{G_i}}{\gamma_{\min}}$$

$$V_{R,\max} = \iint \frac{f_{cvd,i}}{1.5} ds_i$$

Assumption: shear stress / shear strain relationship at high temperature proportional to axial stress / strain relationship

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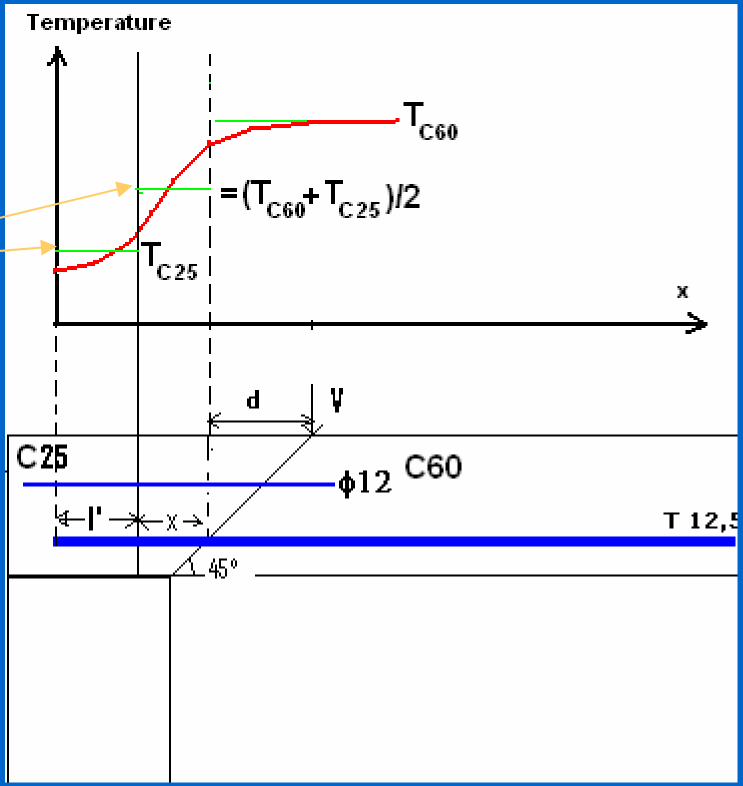
Interaction model

Bond and achorage of reinforcement

$$F_{R,a,fi} = F_{R,a,fi,p} + F_{R,a,fi,s}$$

$$= A_p \sigma_{ra,fi} + A_s f_{yk,fi}$$

$$= A_p \left(\frac{l'.f'_{bpd,fi}}{\alpha_2 \phi} + \frac{x.f_{bpd,fi}}{\alpha_2 \phi} \right) + A_s f_{yk}$$



ϕ_{12} fully anchored and $K_s(\theta)=1$

$$f'_{bpd,fi} = \eta_{p2}\,\eta_1 \frac{0{,}7 f_{ctm,C50}\, k_{c,t}\left(\frac{T_{C60}+T_{C25}}{2}\right)}{\gamma_{c,fi}}$$

$$f_{bpd,fi} = \eta_{p2}\,\eta_1 \frac{0{,}7 f_{ctm,C25}\, k_{c,t}\left(T_{C25}\right)}{\gamma_{c,fi}}$$

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A proposal of simplified calculation method for shear strength

Shear flexure simplified model

$$V_{Rd,c,fi} = \left(\left(C_{Rd,c}\, k\, (100\, \rho_{l,fi}\, \overline{f_{c,fi}}\right)^{1/3} + k_1\, \sigma_{cp,fi}\right) b_w\, d$$

ratio of longitudinal reinforcements brought back to the <u>minimal</u> section

$$\rho_{l,fi} = \frac{\sum F_{R,a,fi}}{500} \frac{1}{b_w\, d}$$

b_w web minimal width

average concrete stress due to prestressing on the considered section

$$\sigma_{cp,fi} = \min(k_p(\theta)\sigma_{cp,20°C}; \frac{F_{R,a,p,fi}}{A_c})$$

$\overline{f_{c,fi}}$ mean compressive concrete strength on the total section, i.e. temperature at mid heigth of the slab

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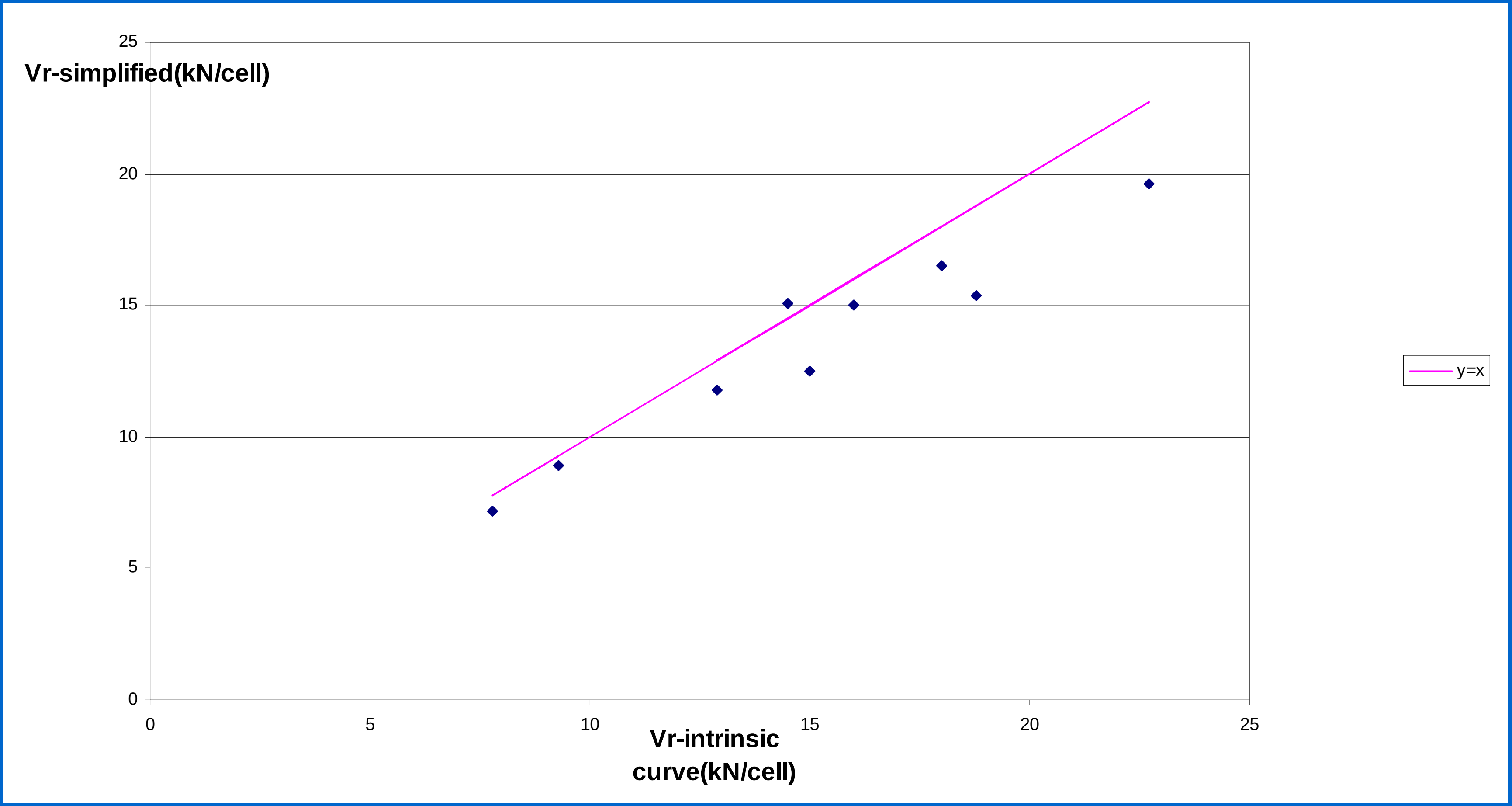
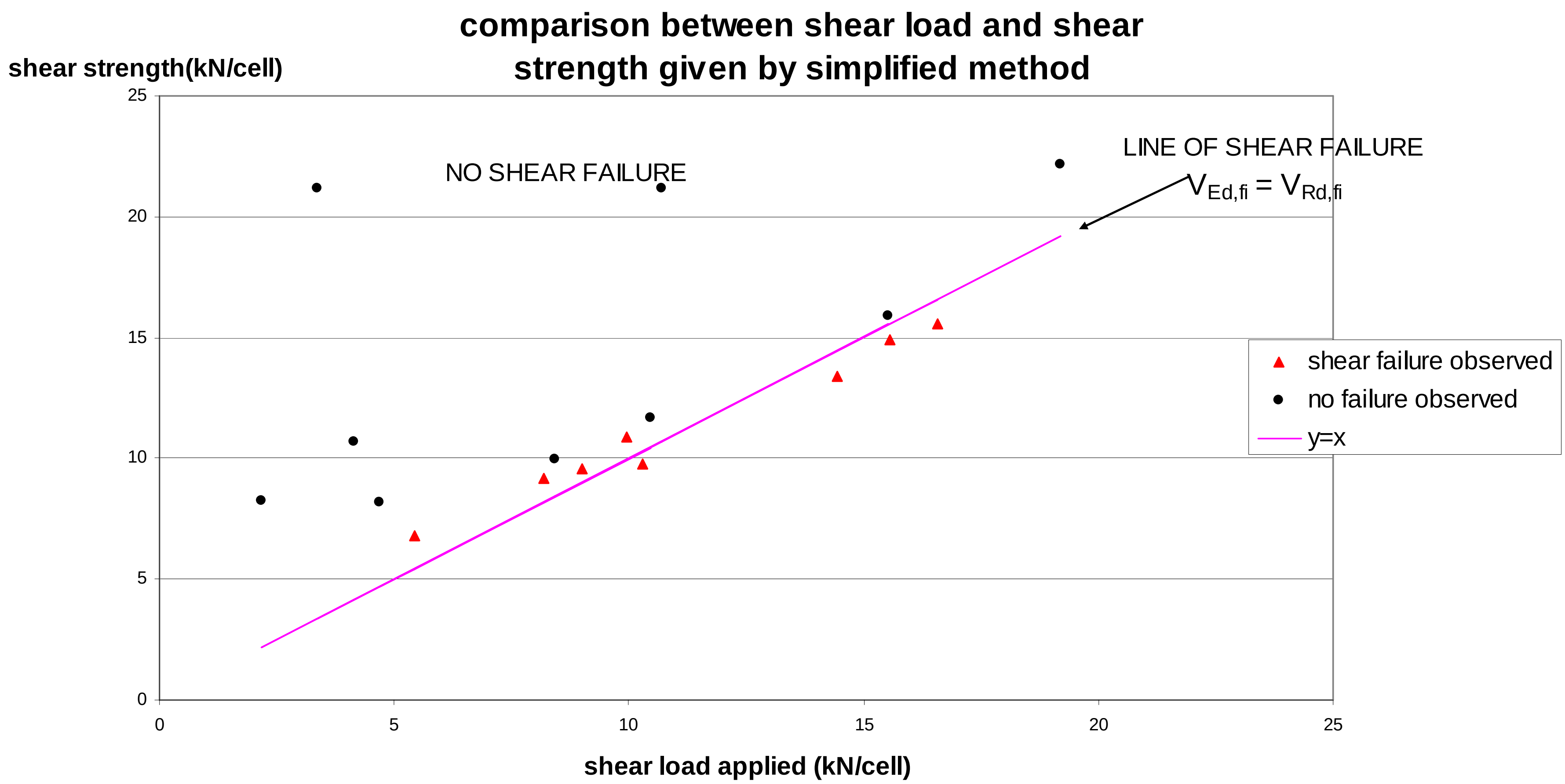
Comparison between shear-flexure simplified model and interaction model



Comparison with available test results



Data base (shear failure tests):

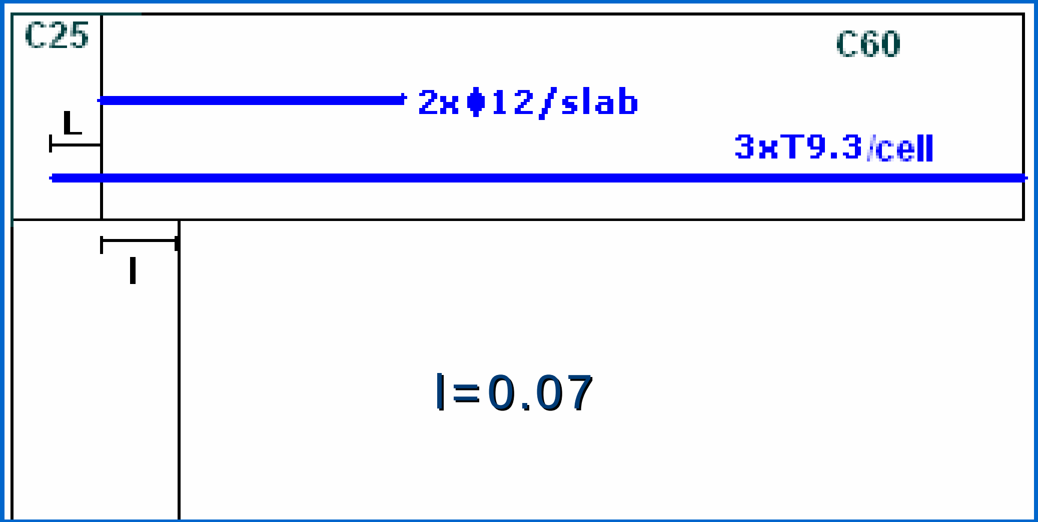
-Arnold Van Acker: 5

-Tauno Hietanen: 4

Characteristics of tests with shear failure

Test report.	Year	Origin	Slab height mm	Prestressing strands	Span x width m²	HA12	Self Weight kN/m²	Vtest/dalle kN	Test duration	Vr/dalle kN
Dk-Betonelement foreningen	2005	Denmark	265	2T12.5 a=40	2,94 x 2,4	yes	3,65	89.48	45'	90.55
FEBE-RUG 9158-a	1998	Belgium	265 +30	2T12.5 a=50	2,9 x 2,4	yes	4,55	85.54	120'	83.1
Belgium 1971	1971	Belgium	265	T12.5 a=31	2,9 x 1,2	no	3,86	54.72	33'	52.16
Denmark 1998	1998	Denmark	185	T9.3 a=31	6,2 x 2,4	no	2,62	43.5	22'	47.8
VTT/PAL 4350	1984	Finland	265	T9.3 a=60	5,08 x 2,4	no	3,86	25.11	130'	27.43
VTT/PAL 2480/82	1982	Finland	265	T12.5 a=65	3,9 x 2,4	no	3,6	44.28	63'	43.68
VTT/PAL 4248/84	1984	Finland	265	T12.5 a=64	5,185 x 2,4	no	3,6	48.65	49'	48.54
VTT/PAL 566d/85	1985	Finland	265	T12.5 a=57	5,165 x 2,4	yes	3,6	55.56	77'	54.86
VTT/PAL 90228/89	1989	Finland	265	T9.3 a=62 T12.5 a=92	5,165 x 2,4	yes	3,6	77.44	27'	79.32

Influence of longitudinal bars and protruding strands



HC : 270 mm

Span : 12.5 m

R120

Configuration	Vr (kN/cell) intrinsic curve	Vr(kN/cell) Simplified method
No ϕ12 L=0	11.2	12.1
ϕ12 L=0	16.7	15.1
ϕ12 L=0.1	19.8	17.8

Concluding remarks on the shear resistance models

Both formulas seems to give a correct answer compared to test results of HC slabs

The “interaction method” allows to take into account the inter-action between the slabs ant supports (compression for floors with low slenderness ratio – tension for floors with cable effect), this approach could be developed for a global study (fire development, behaviour of the structure)

It is obvious that “shear flexure” simplified method is too conservative for short time fire exposure, and probably for thick HC

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Tentative review of of tabulated data

Standard fire resistance	Shear capacity $V_{Rd,fi}/V_{Rd,cold}$ (%) Slab thickness (mm)				
	160	200	265	320	400
R30	73		70	71	76
R60	66	68	56 / 62	55 / 63	52 / 70
R90	60*	63	53	50	48 / 68
R120	55*	*	50	47	46
R180	44*	*	*		42

In blue : « shear-flexure » in red : « interaction »

* Does not satisfy the insulation criterion

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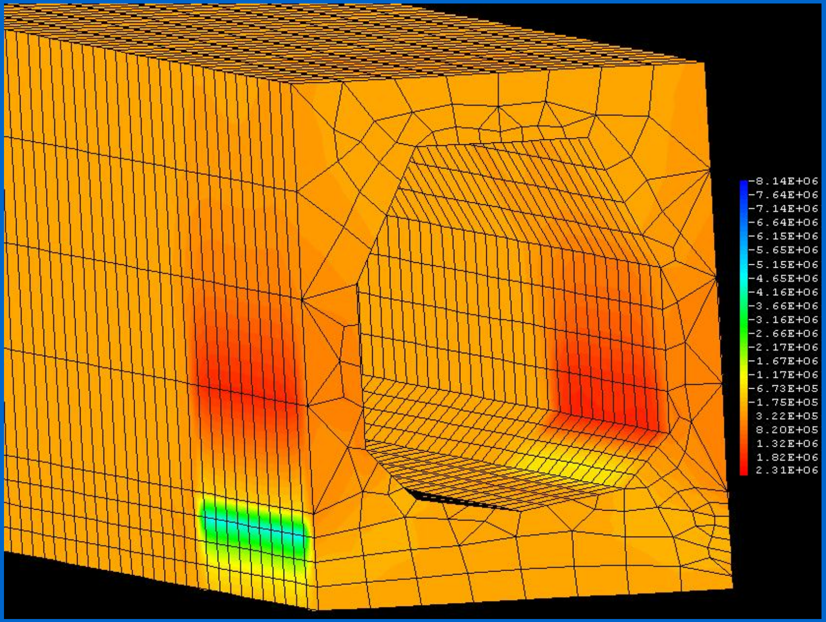
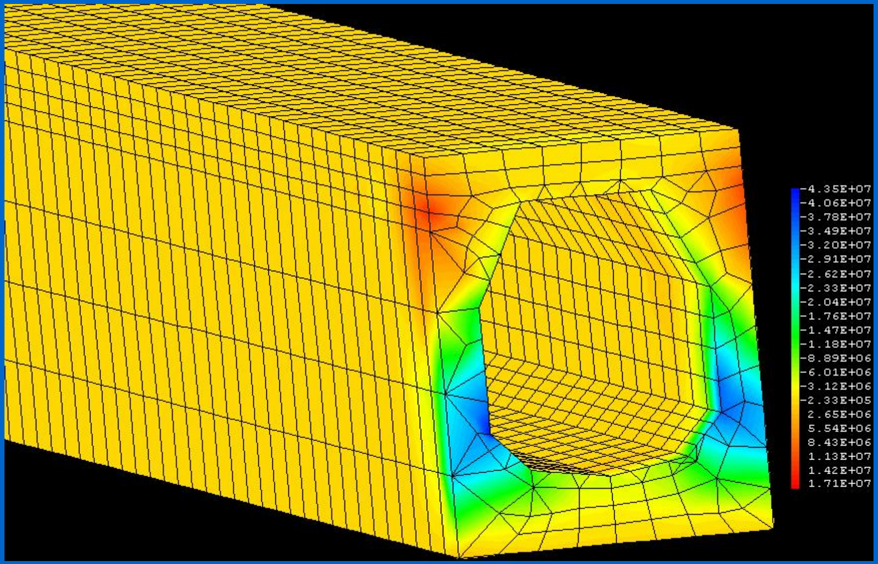
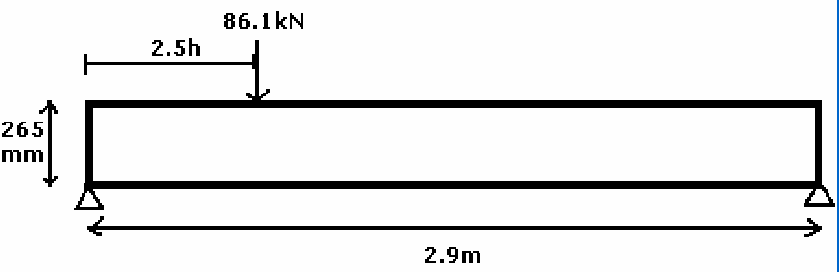




Shear by prestressing

Shear by thermal strain

Dk-Betonelement (Denmark)



$$\Rightarrow V_{Ed,fi} = 73.3 kN/m \Rightarrow V_{Ed,fi/cell} = 16.56 kN/cell$$

Effort anchored by reinforcement

$$F_{R,a,fi} = F_{R,a,fi,p} + F_{R,a,fi,s} = A_p \sigma_{ra,fi} + A_s f_{yk,fi}$$

$$= A_p \left(\frac{l'.f'_{bpd,fi}}{\alpha_2 \phi} + \frac{x.f_{bpd,fi}}{\alpha_2 \phi} \right) + A_s f_{yk,fi}$$

$$f_{ck} = 60\ MPa \Rightarrow f_{ctm} = 4.4\ MPa$$

$$l' = 0; a = 40\ mm$$

$$x = 70\ mm + a = 110\ mm$$

$$T_s = 250\ °C \Rightarrow k_{ct}(T_s) = 0.7$$

$$f_{bpd,fi} = \eta_{p2} \eta_1 \frac{0,7 f_{ctm} k_{c,t}(T_s)}{\gamma_{c,fi}}$$

$$= 1.2 \frac{0.7 \times 4.4 \times 0.7}{1} = 2.59 MPa$$

$$f_{yk,fi}(\phi 12) = \frac{f_{yk} \times k_s(T_{\phi 12})}{\gamma_{s,fi}} = f_{yk} = 500 MPa$$

$$\left\{ \begin{array}{l} 2 \times T12.5; \alpha_2 = 0.19; A_p = 186\ mm^2 \\ A_s = 90.4\ mm^2 \end{array} \right\} \Rightarrow F_{ra,fi} = 67.5 kN$$

Dk-Betonelement (Denmark)

ratio of longitudinal reinforcements brought back to the <u>minimal</u> section

$$\left\{\begin{array}{l} b_\omega = 41\,mm \\ d = h - 40 = 225\ mm \end{array}\right\} \Rightarrow \rho_{l,fi} = \frac{F_{ra,fi}/500}{b_\omega d} = 1.46\,\%$$

average concrete stress at x+d (shift rule):

$$\sigma_{cp,fi} = \frac{F_{R,a,fi,p}(x+d)}{A_c} = 2.2MPa$$

Mean compressive concrete strength

$$\overline{f_{c,fi}} = \frac{f_{ck}k_c(\overline{\theta})}{\gamma_{c,fi}} = 60 \times 0.98 = 58.8MPa$$

$$V_{Rd,c,fi} = \left(\left(C_{Rd,c}\,k\,(100\,\rho_{l,fi}\,\overline{f_{c,fi}}\right)^{1/3} + k_1\,\sigma_{cp,fi}\right)b_w\,d = 16.77kN$$

$$C_{Rd,c} = 0.18$$

$$k = 1 + \sqrt{\frac{200}{d}} = 1.94$$

$$k_1 = 0.15$$

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