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# Fire resistance of concrete slabs acting in compressive membrane action

No need for steel anymore?

11th of April 2017 – IStructE HQ London

STRUCTURES IN FIRE FORUM

# Introduction

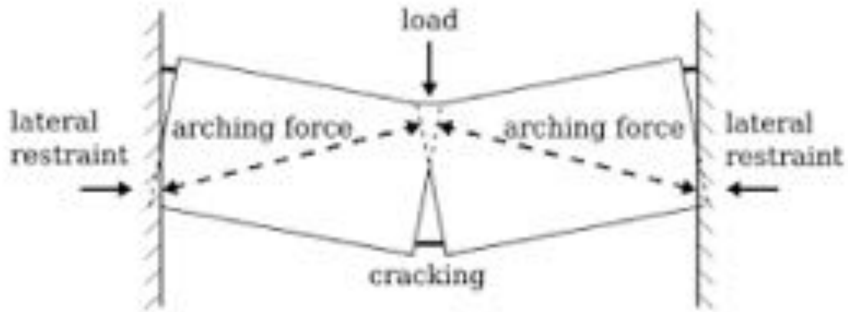
- Bearing capacity of (existing) slabs?

- Experiences in building renovations with a surprising low capacity following classic bending theory, mostly with rather thin slabs.
- No excessive deformations noted before => no tensile membrane action involved but probably compressive membrane action.

- What about fire resistance of such slabs?

# Compressive membrane action (CMA)

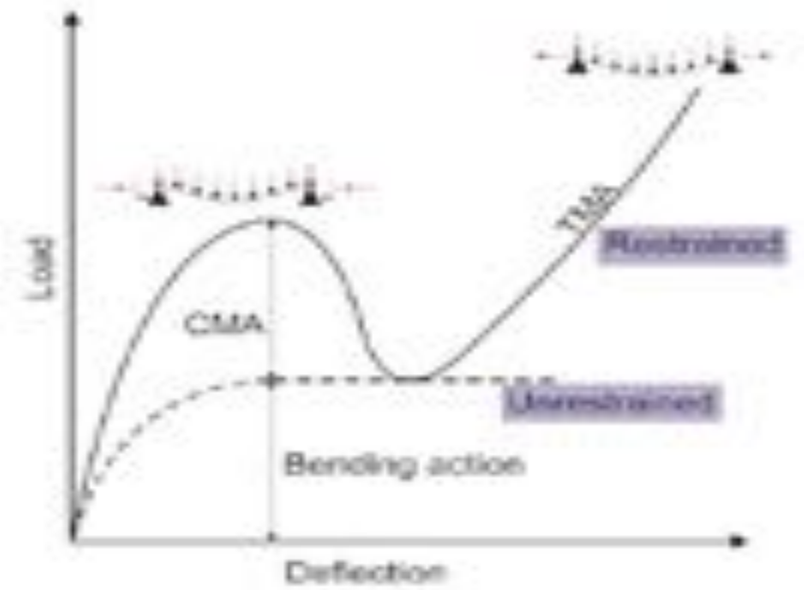
- The arch effect – compressive membrane action




- Lateral restraint capable to withstand compressive forces

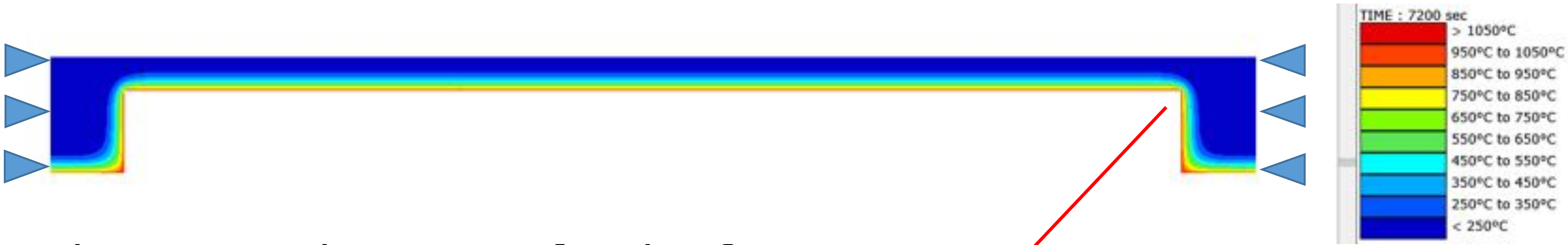
# Compressive membrane action (CMA)

- CMA mechanism fails due to concrete crushing => TMA or failure

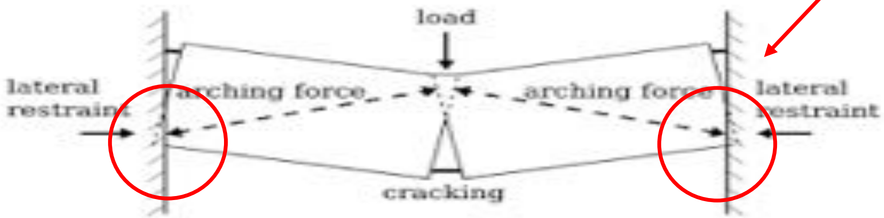


# CMA at elevated temperature

- Isothermal lines




- Behaviour at the origin of arches?

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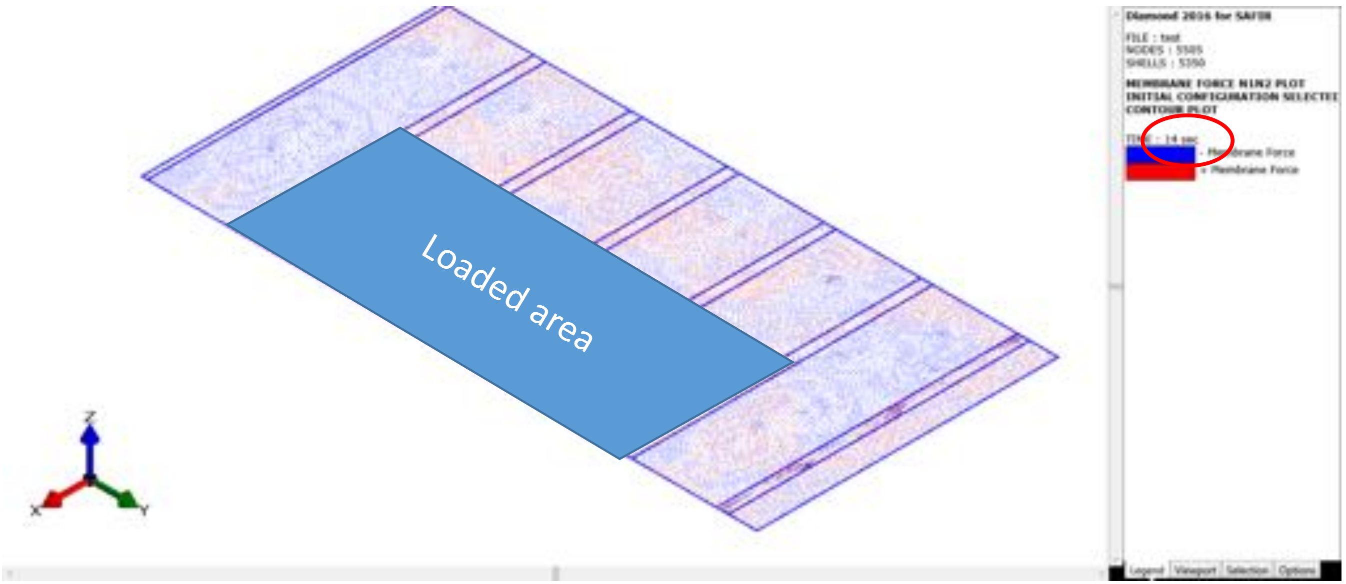
# CMA with FEM models?

- By the aid of plane elements with bending and membrane behaviour.
- Advanced material model (Explicit Transient Creep).
- At ambient conditions, time dependent load function (20s)




- Stops at 70%
- Doesn’t work

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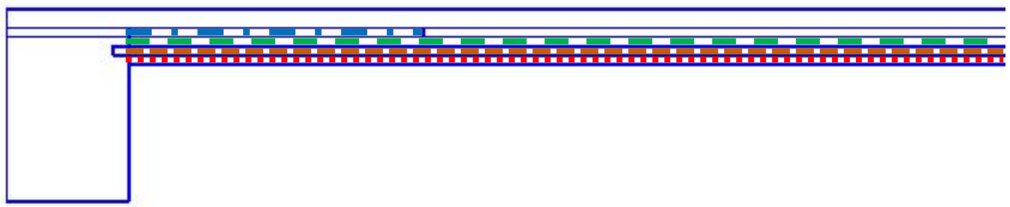
# CMA with FEM models?

- Vertical section with superimposed layers of shell elements
- Advanced material model (Explicit Transient Creep).
- At ambient conditions, time dependent load function (20s)
- Rebar elements



- Adding elevated temperature profile pro layer is quiet simple

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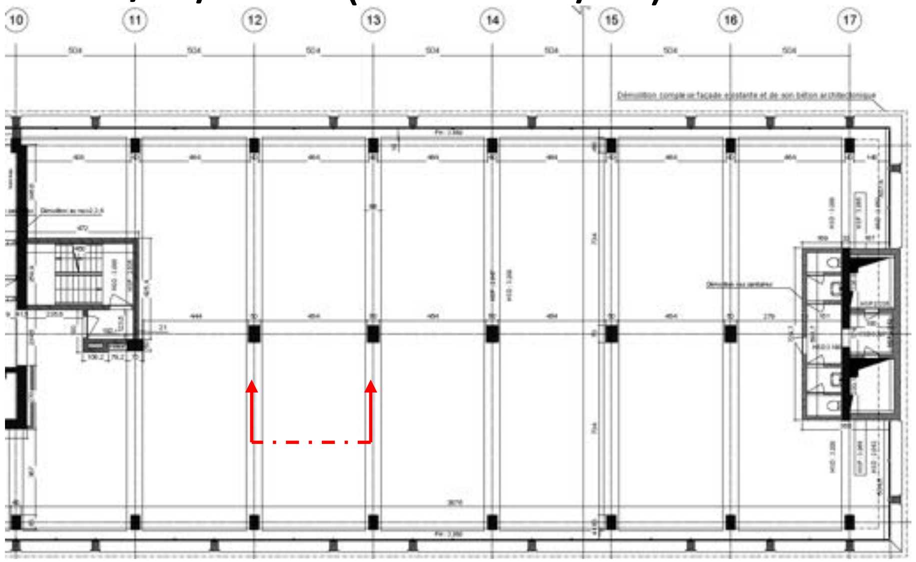
# Case study – building description

- Leopold tower in Brussels near the NATO (Evere).
- Office building in the past with screed and mobile load of 3 $kN/m^2$
- Slab of 0.14 m, 5.04 m span, width 16.5m, upper (curtailed) and lower principal reinforcement = Ø8/0.15 (335 $mm^2/m$)

Half floor:

Section

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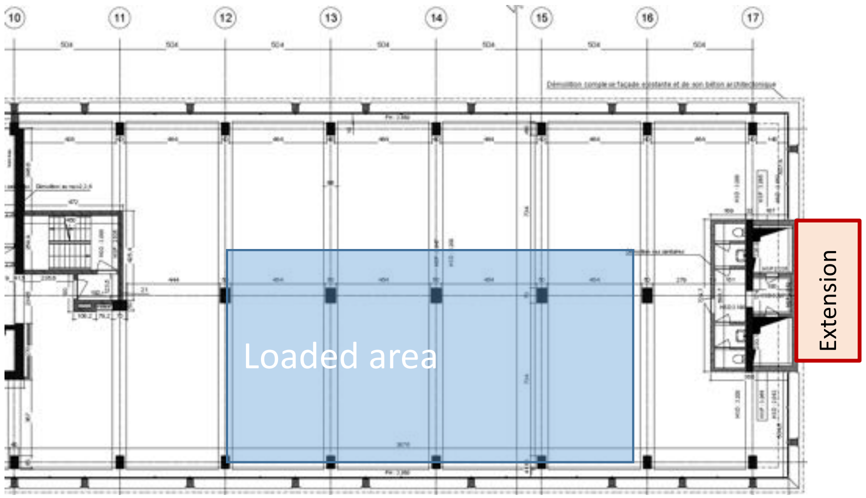
# Case study – building description

- Transformation into an apartment building with reduced mobile loads but increased super imposed dead loads.
- Out of bending theory; insufficient reinforcement even for the existing situation.
- But restraints available at the extremes => CMA possible?

- Due to high costs of external reinforcement + fire protection and possible benefits the owner agreed with a load test.

# Case study – Test set-up

- Load test done till ULS-values (for ambient conditions); 11.58 kN/m²
- Swimming pools of 0.8 m + DL.





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# Case study – Bearing capacity

- Elastic bending theory

$$M_{Rd} = (h - c_{centre} - \frac{x}{2})A_s f_{yd} = (h - c_{centre} - \frac{A_s f_{yd}}{2bf_{cd}})A_s f_{yd}$$

  - Span 13.17 kNm, hogging moments 11.52 kNm with sum 24.69 kNm
  - With $\Sigma M = wL^2/8$ => $w < 7.78$ kN/m² and needed + test 11.58 kN/m² ?
  - Only dead loads are representing already 6.18 kN/m² (factored)
- Plastic analysis with membrane action results in 0.35 m deformation!
- Measured deformations of only a few mm
- Only CMA can explain this behaviour

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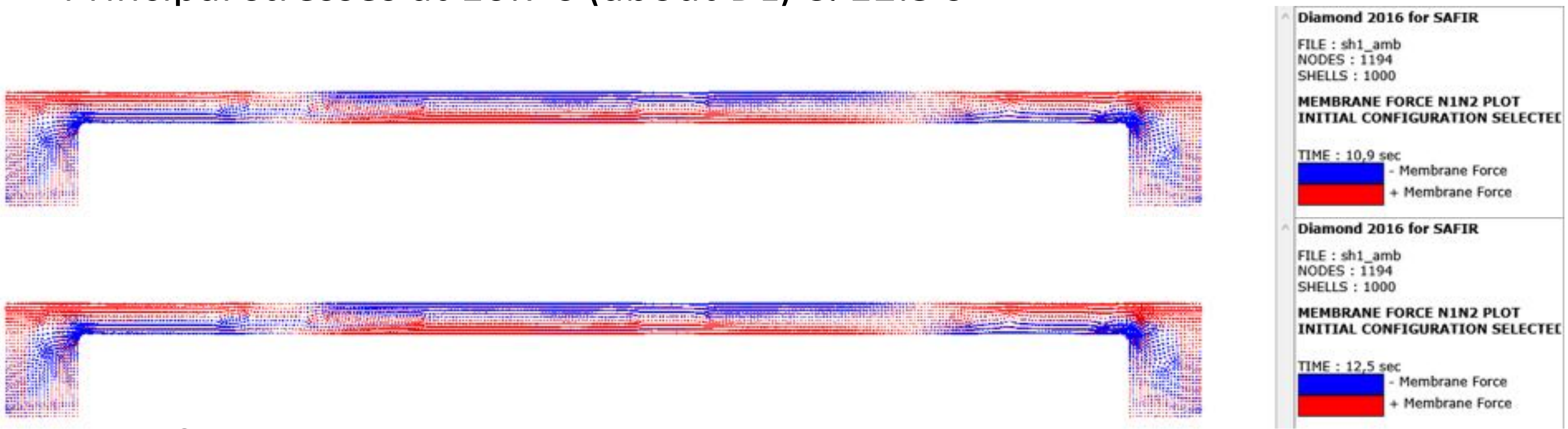
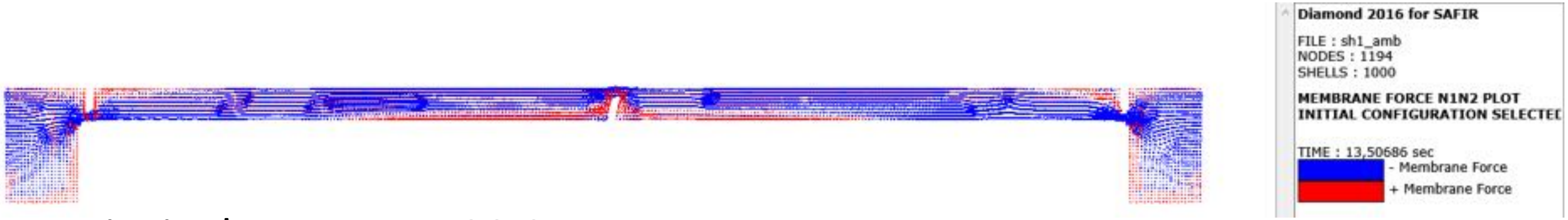
# Case study – FEM @ ambient temperature

- CMA simulation without reinforcement
- Full load available after 20 s with equal increments/time step
- Principal stresses at 10.7 s (about DL) & 12.5 s

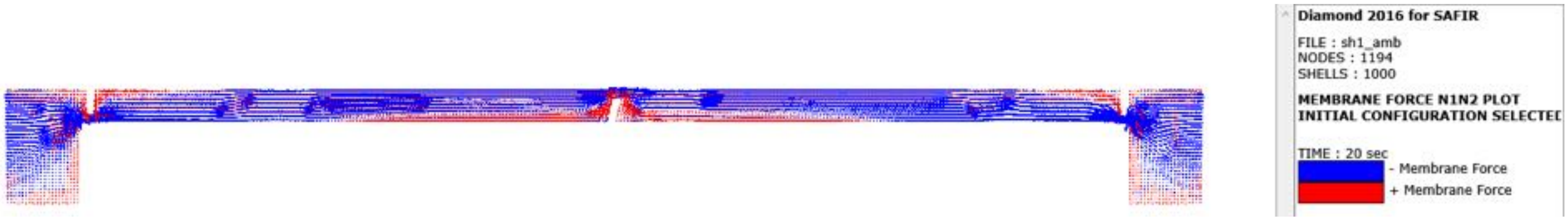


# Case study – FEM @ ambient temperature

- Principal stresses at 13.5 s




- Principal stresses at 20.0 s

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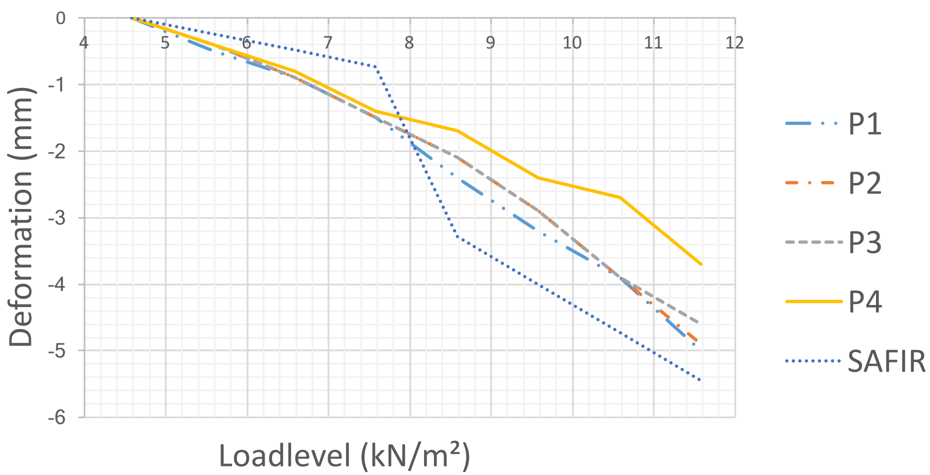
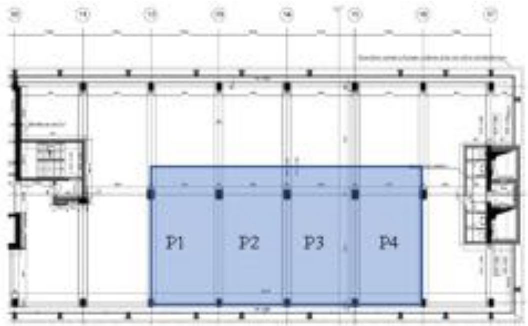
# Case study – FEM @ ambient temperature

- Measured deformations related to computed values

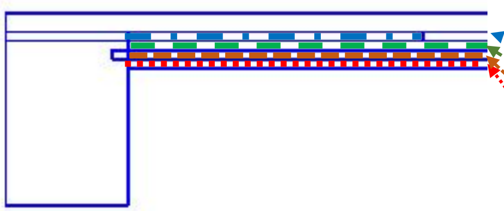


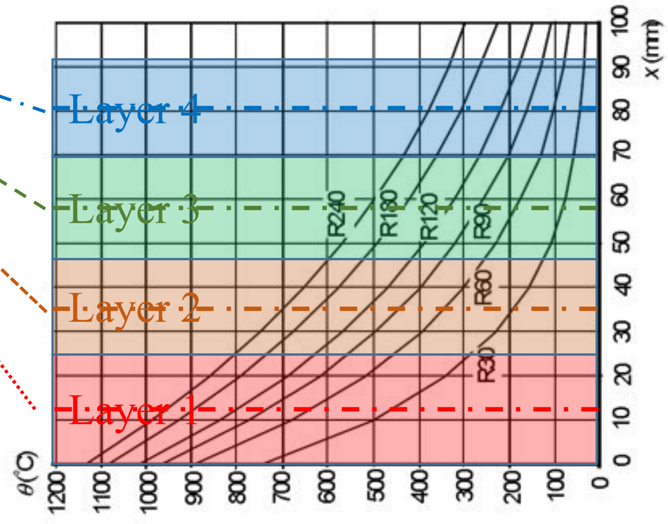




- Seems to correspond on a reasonable way ☺

# Case study – FEM @ elevated temperature

- Temperature profiles out of NBN EN 1992-1-2






- Reduced load 8.08 kN/m²
  (due to fire conditions)

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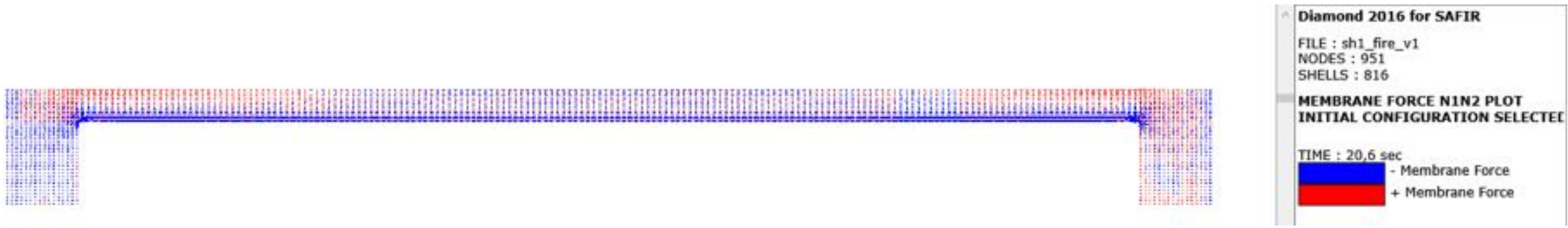
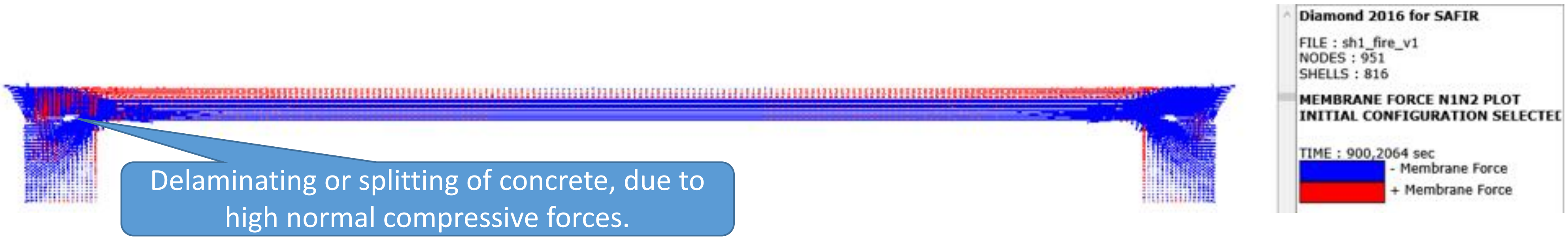
# Case study – FEM @ elevated temperature

- Principal stresses at 20.0 s with layered shell elements




- Principal stresses at 900 s (delaminating starts)

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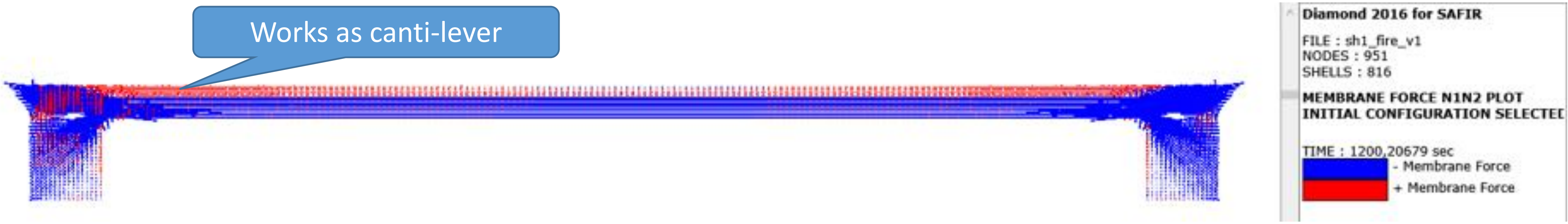
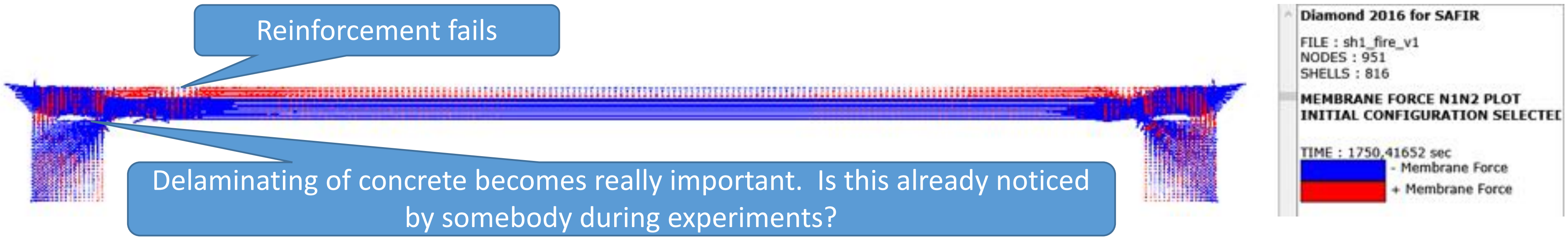
# Case study – FEM @ elevated temperature

- Principal stresses at 1200 s




- Principal stresses at 1750 s (cracks on top and termination/failure)

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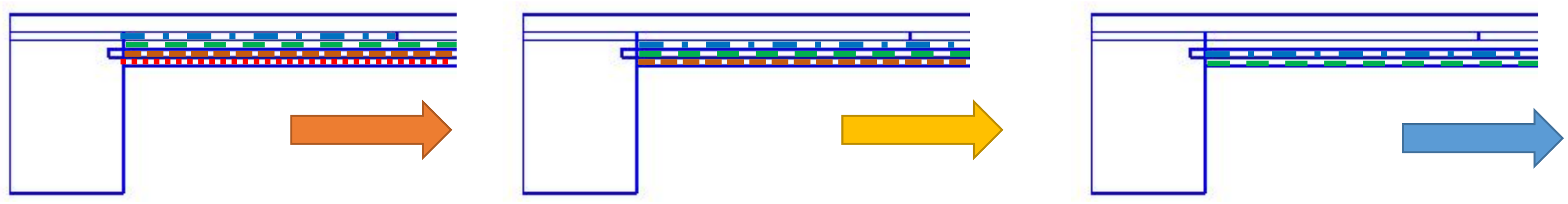
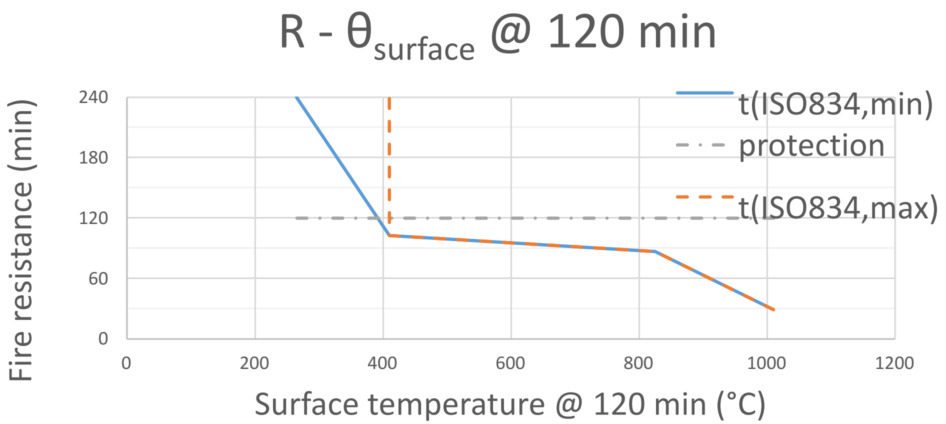
# Case study – Failure time with protection

- Temperature profiles out of NBN EN 1992-1-2



- Protection time

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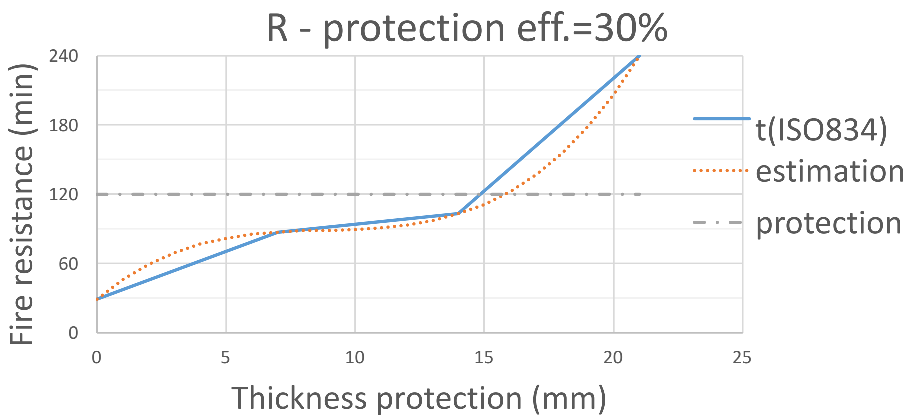
# Case study – Failure time with protection

- Protection material; presume the same efficiency as one layer of 23 mm of concrete:

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# Simplifications – engineering judgement

- Let's try the simple way:
  1. At ambient temperature, the horizontal reaction force is computed as: $H_{Sd} = M_{Sd}/z = p_{Sd}.L^2/(8.z)$ with z = the lever arm or arch camber, $p_{sd}$ = design load and L the span.
  2. The depth of the compression area is: $x = H_{Sd}/(b.f_{cd})$ with b the slab width and $f_{cd}$ the design concrete strength in compression.
  3. The lever arm is modified to account for the depth x: $z = h - x/2$ with h the slab height; some iterations may be required from 1 to 3 to find the final value of the lever arm.

# Simplifications – engineering judgement

- **Let's try the simple way**:
  1. In case of fire, the applied load is lower than the design load at ambient temperature; hence the horizontal reaction force is reduced as: $H_{fi} = p_{fi}.L^2/(8.z_{fi})$ with $z_{fi}$ = lever arm and $p_{fi}$ = load in case of fire.
  2. The depth of the compression area in case of fire is: $x_{fi} = H_{fi}/(b.f_{\theta})$. Looking to *Fig. 1a*, the temperature θ of the lower part of the slab will be important at the origins of the arch.
  3. The lever arm is modified to account for the depth $x_{fi}$: $z_{fi} = h - (x_{fi}/2)$; some iterations may be required from 1 to 3 to find the final value of the lever arm in case of fire.
  4. Finally, the verification of structural safety in the fire situation is performed as: as long as $H_{fi}<H_{Sd}$, the slab is able to sustain the load in the fire situation.

# Simplifications – engineering judgement

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4. Finally, the verification of structural safety in the fire situation is performed as: as long as $H_{fi}<H_{Sd}$, the slab is able to sustain the load in the fire situation.

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# Simplifications – engineering judgement

- Case study => doubtful result ?

1. $H_{Sd} = M_{Sd}/z = 11.58.5.04^2/(8 \cdot 0.133) = 276.46$ kN/m with $z = 0.95.0.140 = 0.133$
2. $x = 276.46/(0.85 \cdot 30/1.5) = 16.26$ mm, hence take 16 mm.
3. $z = 0.140 – 0.016/2 = 0.132$ m which can be considered as converged, taking into account 1 % deviation compared with 0.133.
4. $H_{fi} = 8.08.5.04^2/(8 \cdot z_{fi}) \leq 276.46$ kN/m which leads to $z_{fi} \geq 0.093$ m
5. $z_{fi} = 0.140 – (x_{fi}/2) \geq 0.093$ or $x_{fi} \leq 0.093$ m
6. $x_{fi} = 276.46/(f_{c\theta}/f_{ck} \cdot 30) \leq 93$ mm, the maximum allowable reduction to respect this equilibrium is obtained if $f_{c\theta}/f_{ck} \geq 0.10$ and can be applied at the origin of the arch. Take this equal to the one corresponding to the temperature in the ultimate fiber of the layer as a rough and save simplification. Following table 3.1 of EC2-1-2 [4] this means 871 °C would be acceptable, which appears at R57 < R120 required but > R29 following SAFIR analysis

# Further developments

- Punching problem:
  - Has the same failure behaviour
  - Is related to the same mechanism
  - But we even don’t understand and agree about at ambient conditions.

  …
- …

# Conclusions

- CMA can play a major role in restrained structures.  With advanced FEM-analysis and suitable material models this can be simulated with a reasonable agreement for the vertical deformations.
- Modelling at elevated temperatures is even more challenging, however maximum surface temperatures could be derived for a practical case study and subsequently the needed fire protection.
- Our understanding of CMA is still to limited to built up simple engineering models.

# Thanks for the kind attention

## Q&A ?

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