Why don't more steel-faced sandwich panelled ceilings collapse in fire?

A presentation to Structures in Fire Forum, 12 April 2016, at the IStructE HQ, Bastwick Street, London.

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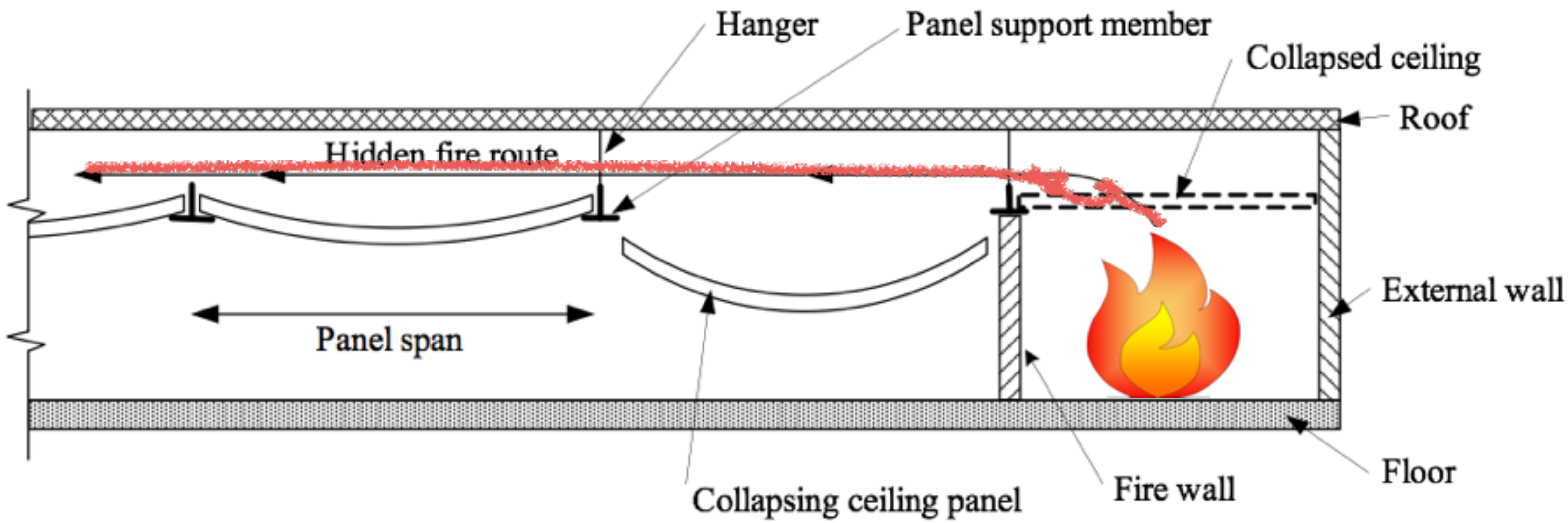
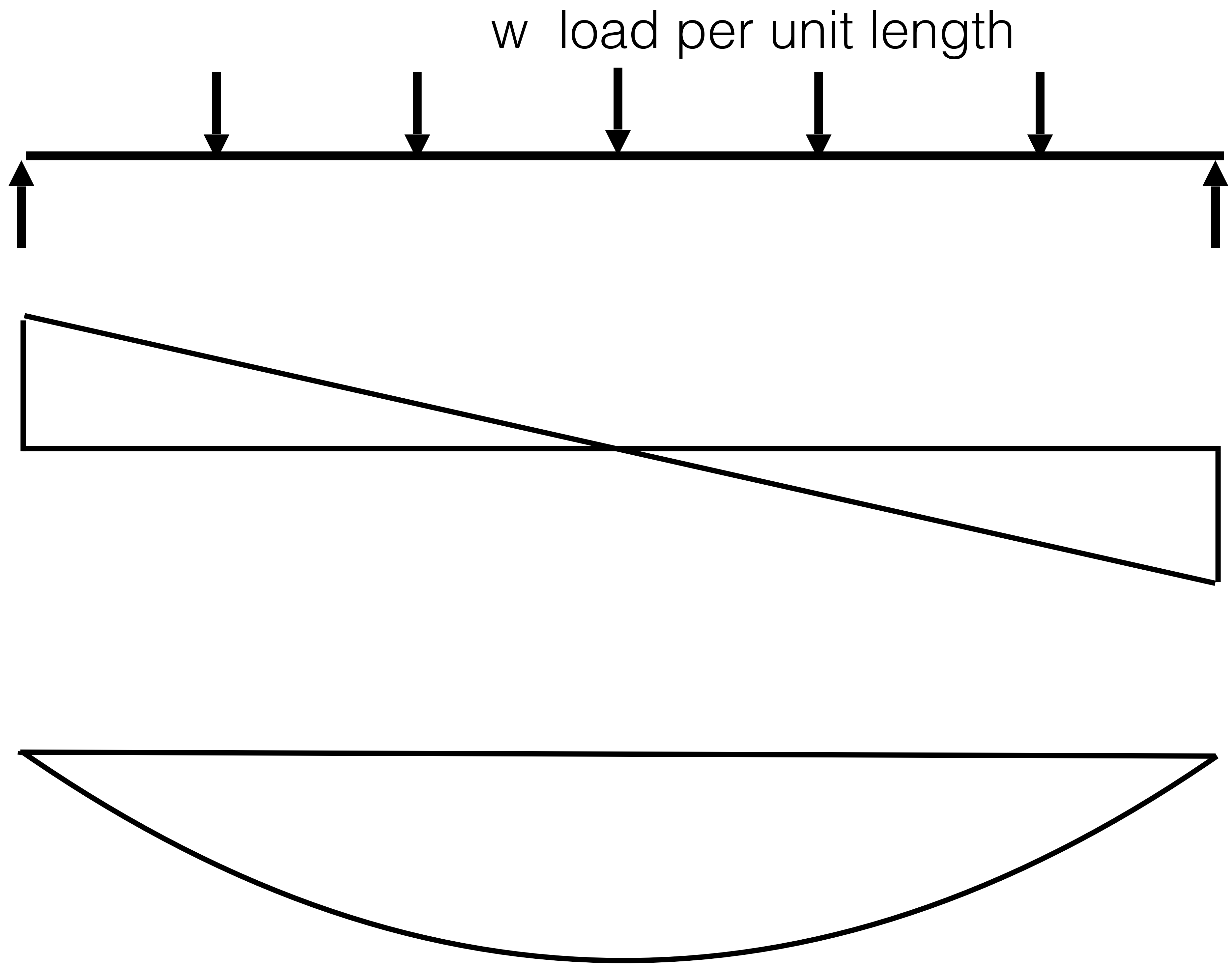
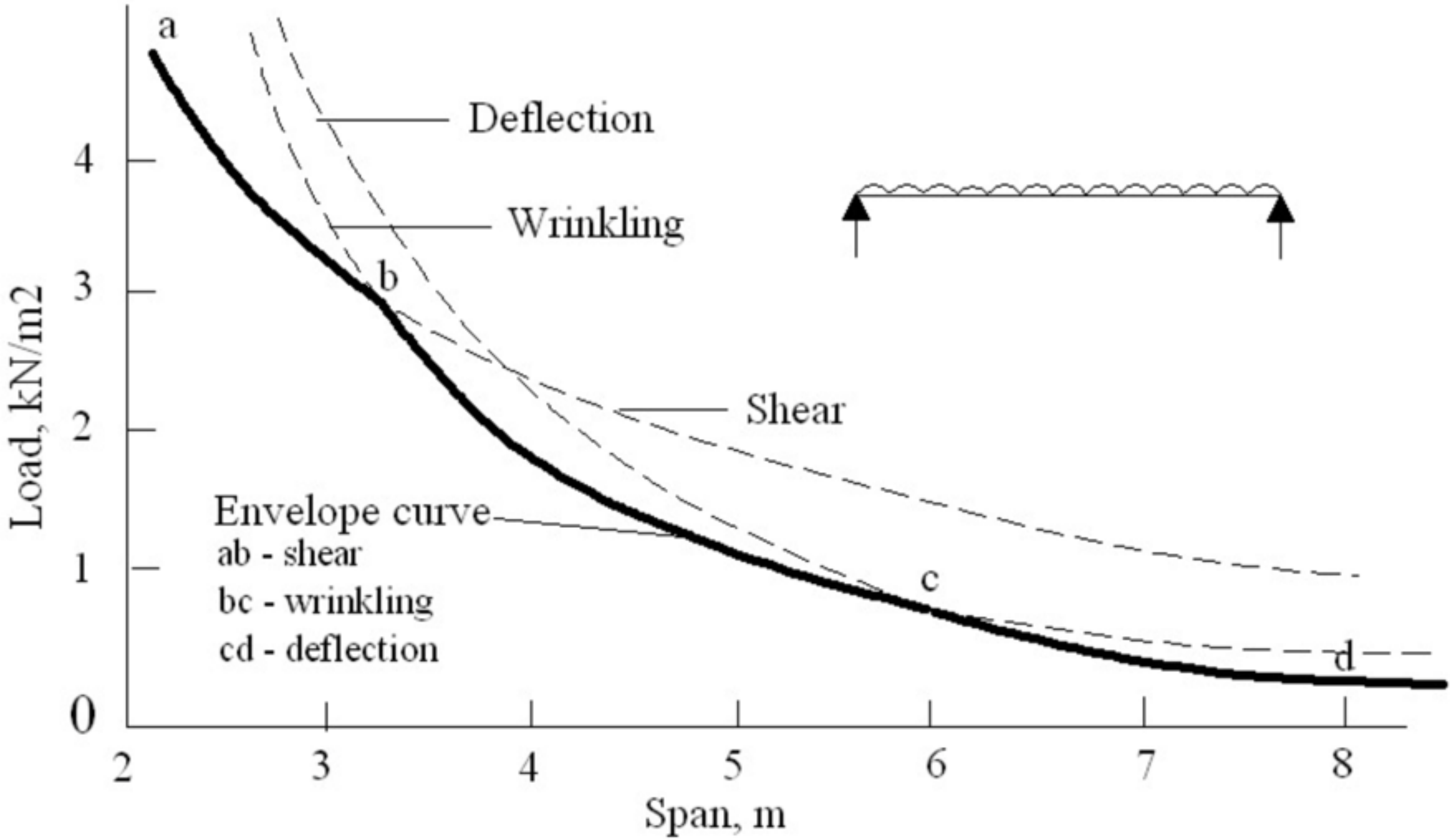


Dangerous hazard of fire **above** a panel ceiling.



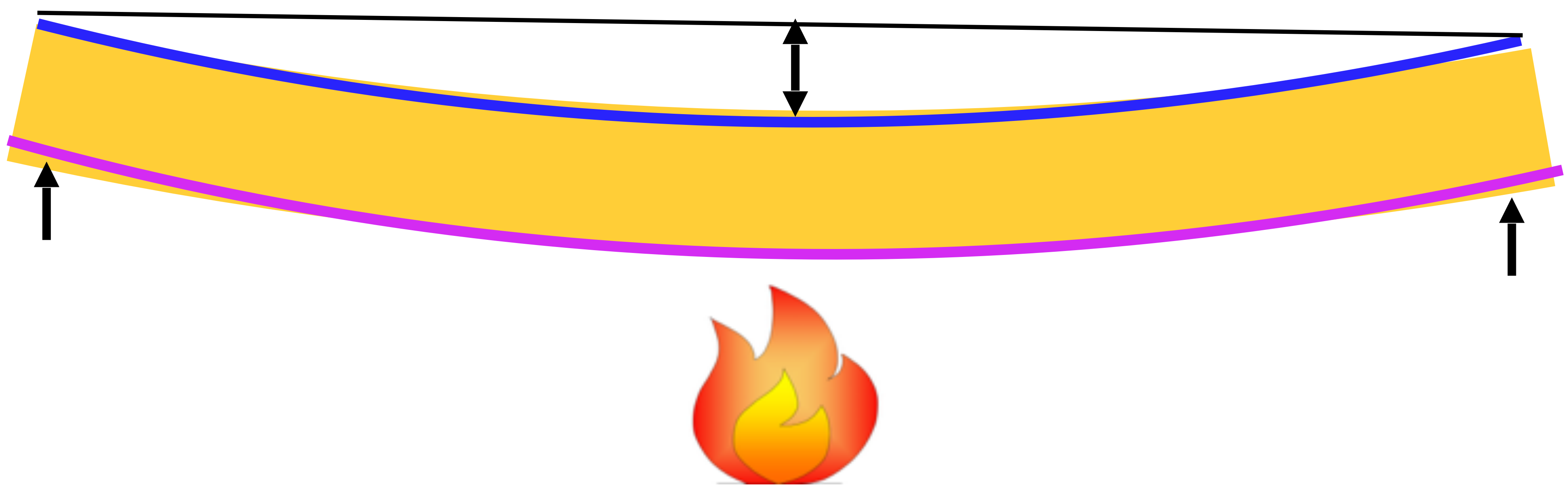
Loading, shear force and bending moment diagrams



Behaviour of panel at room temperature

Sophisticated analysis possible

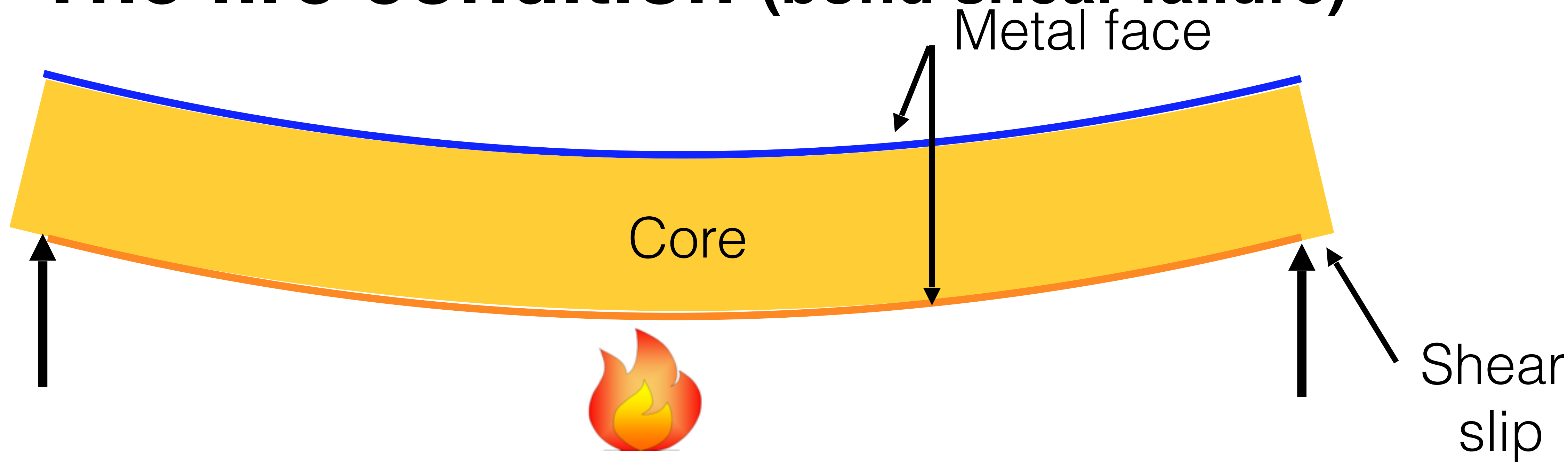
Thermal bowing



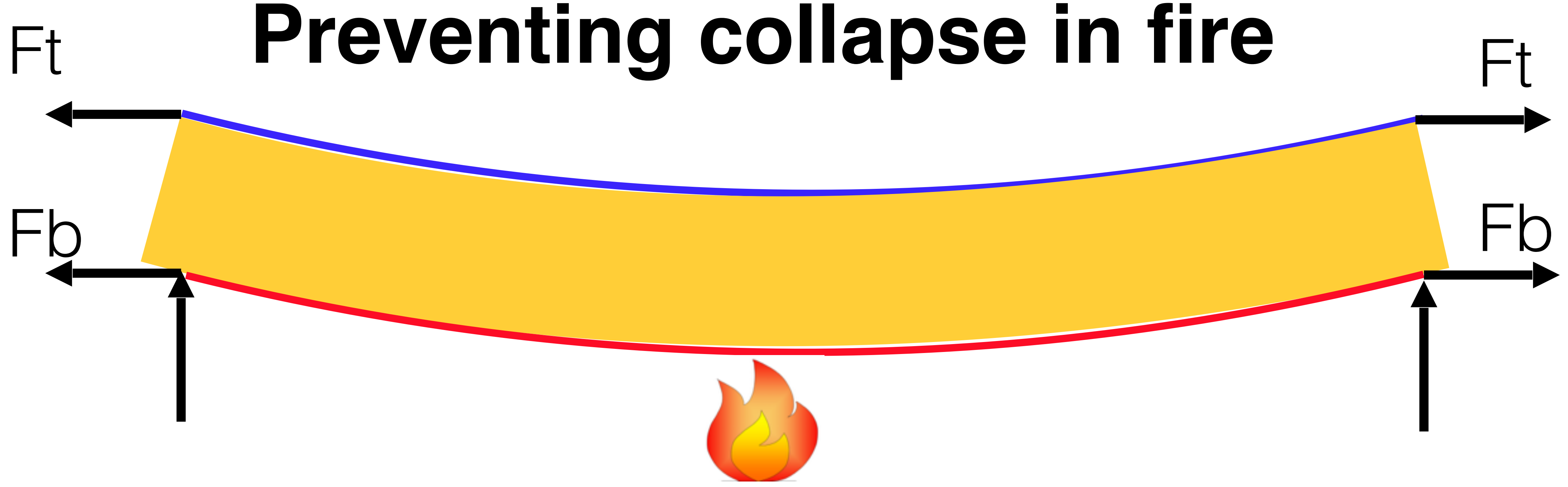
Plane sections remain plane.
No core deformation or bond slip.

In early stage of fire the panel bows towards the fire. When adhesive fails, thermal bowing stops and faces act as catenaries if restrained; if not, panel collapses.

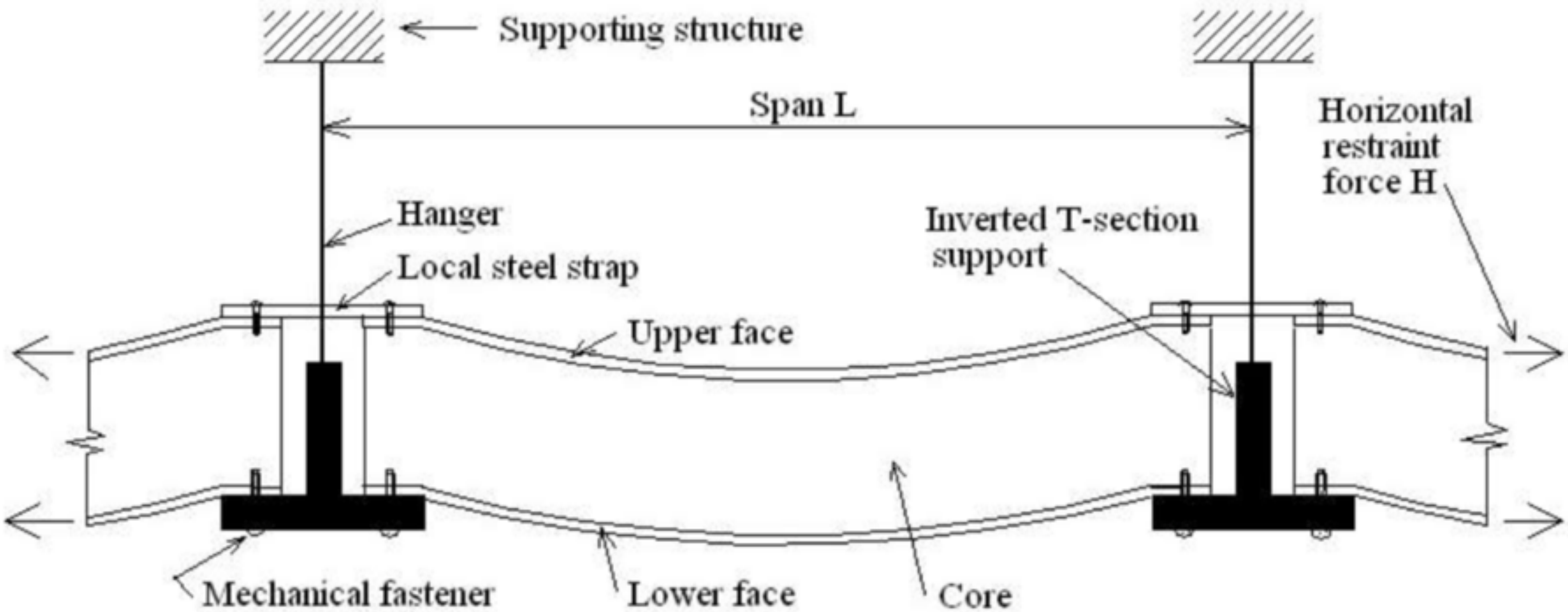
The fire condition (bond shear failure)



Preventing collapse in fire



Collapse prevented by applying catenary force F



A recommended way of installing sandwich panels.

Note horizontal restraint to top and bottom faces

Adhesive data

Small scale tests by BRE showed that delamination occurs when temperature of steel facing is in the range 130 - 350 degC.

Failure temperature is important as this affects magnitude of initial catenary force. The higher the failure temperature the better.

Panel behaviour in fire (fire below)

Panel experiences sag due to thermal expansion of one face when adhesive fails

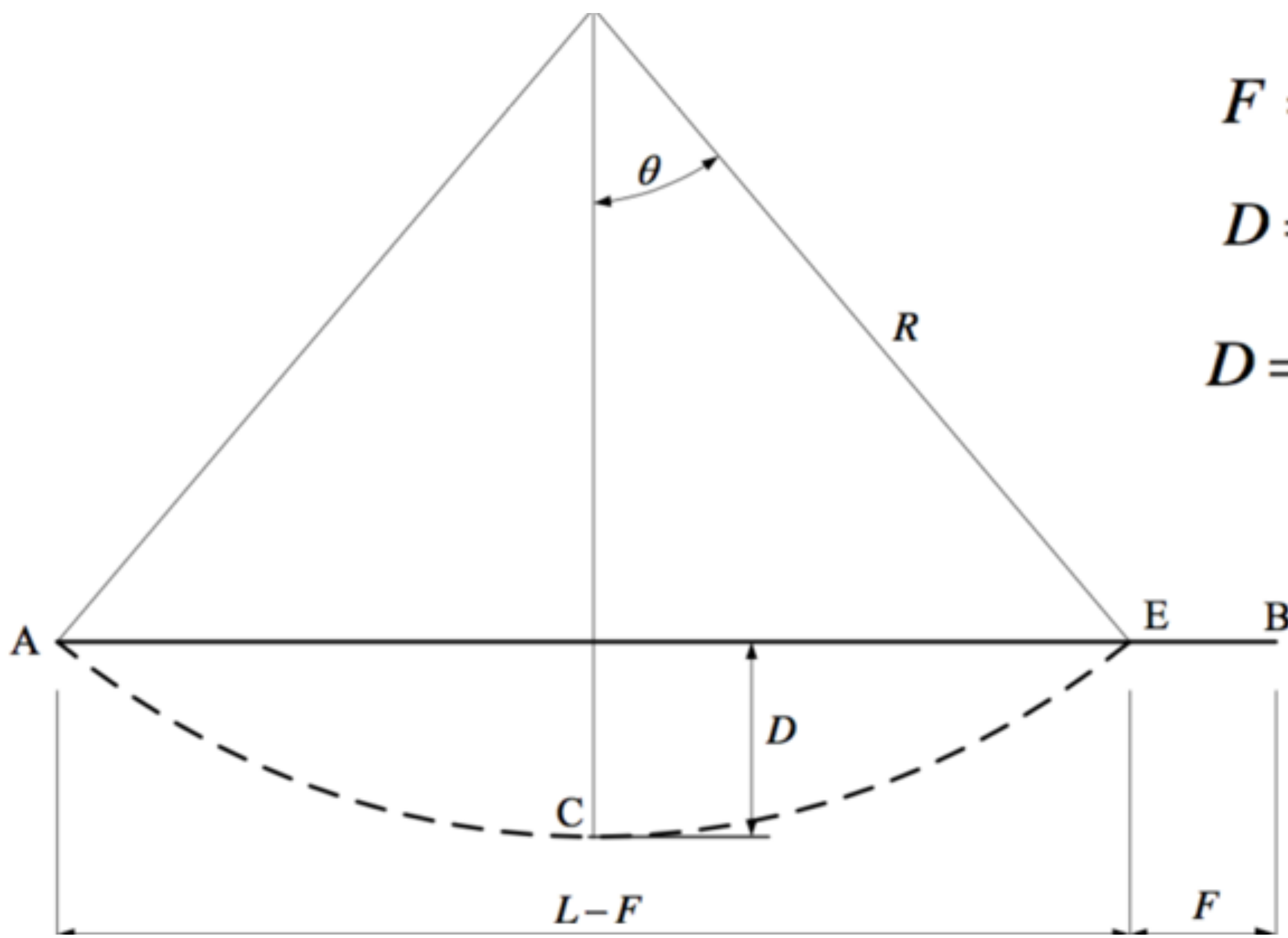
plus

beneficial sag due to inward panel-end movement under action of catenary force

The bigger the sag the smaller the catenary force and vice versa.

Panel Stability Calculation Procedure

Decide adhesive failure temperature (test data)
Calculate mid-span deflection at temperature when adhesive fails
Calculate panel inward-end movement and resulting beneficial increase in mid-span deflection
Calculate catenary force
Calculate number of panel fastenings needed to resist fastening/panel face failure under imposed catenary force



$$F = \alpha L \Delta T$$

$$D = \sqrt{0.375 L F}$$

$$D = L\sqrt{0.375 \alpha \Delta T}$$

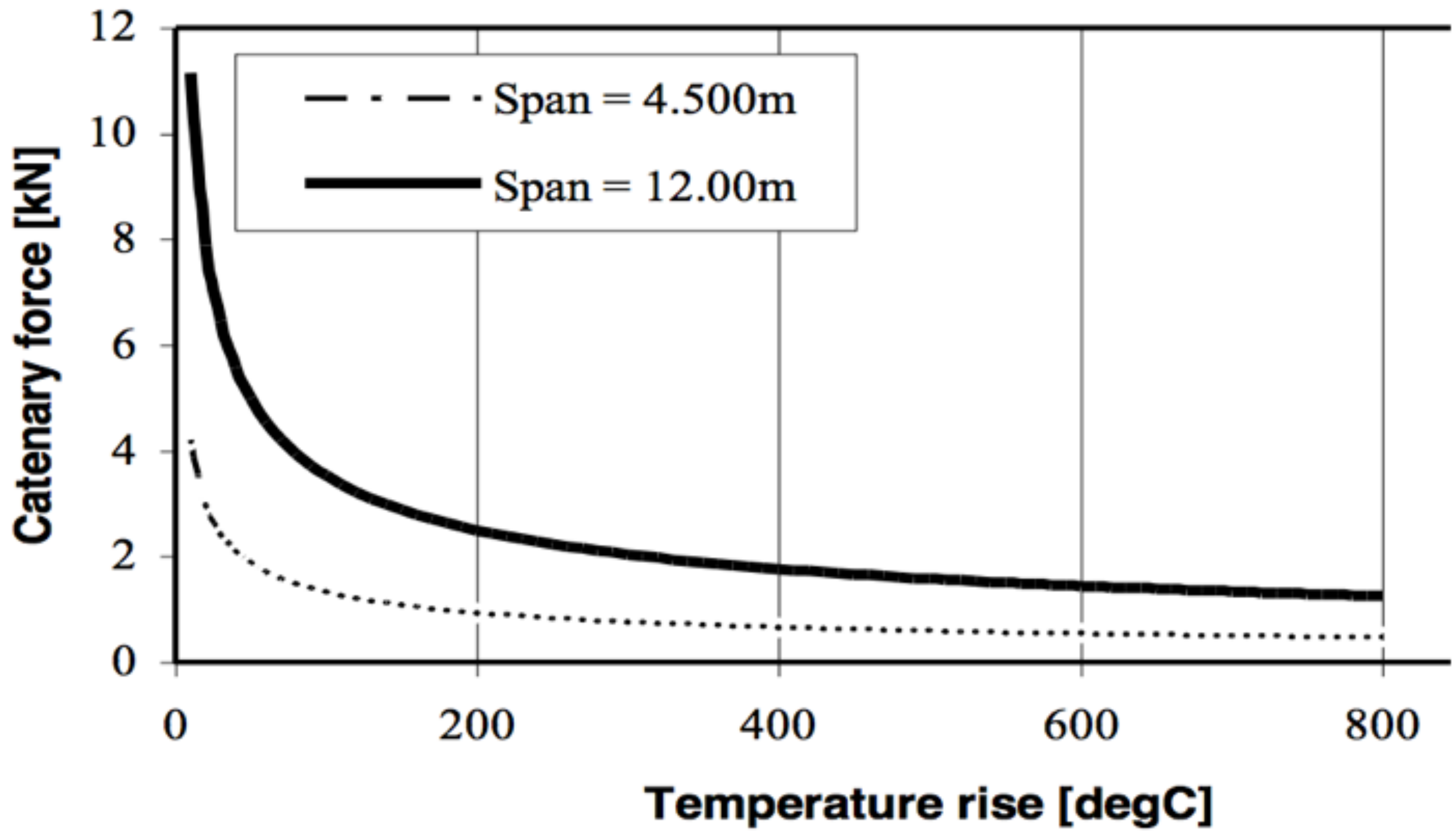
AE = unheated length. EB= expansion if unrestrained.
ACE = bowed shape.

Geometry of heated panel face and equations used in calculation of catenary force.

Panel inward-end movement.

Can be beneficially caused by:

- **movement of support members together as result of catenary force in panel face**
- **slippage in the mechanical fastenings relative to the face and**
- **elongation of the fastening holes in the panel face**



This graph is for one steel face 1m wide by 0.7mm thick with position-fixed ends (no imposed load and no panel inward end movement).

Note that catenary force can be large at low temperatures. Therefore optimum for minimum catenary force is to use an adhesive that fails at highest temperature commercially viable.

BS EN 1524-7: 2012 Extended Application of results from fire resistance tests to BS EN 1364-2

$$F_{E_d} = gL^2 / 8w$$

where

$L = \text{span length}$

$T = \text{temperature}$

$p = \text{relative end movement of fastener}$

$g = \text{panel weight} / m^2$

$w = \text{deflection of ceiling, where}$

$$w = L(0.375\alpha T)^{0.5} + (0.375Lp)^{0.5}$$

This uses deflection equations published in Cooke paper (Journal of Fire Protection Engineering, November 2008; vol. 18, 4: pp. 275-290).

Worked example calculation in the BS EN

$w = \left(L \cdot \sqrt{0{,}375\,\alpha \cdot T}\right) + \left(\sqrt{0{,}375\,L \cdot p}\right)$ $\alpha = 1{,}2 \cdot 10^{-5}$

Panel = 21 kg/m2 $g_{Panel} = \text{Panel} \cdot 9{,}81$ $g_{Panel} = 206{,}01$ N/m^2

EI90: $w_{tot90\ min} = 440$ mm $L_{90\ min} = 5\,000$ mm

$T_{90\ min} = 160$ °C

$w_{dT90\ min} = L_{90\ min} \cdot \sqrt{(0{,}375\,\alpha \cdot T_{90\ min})}$ $w_{dT90\ min} = 134{,}2$ mm

$w_{inc90\ min} = w_{tot90\ min} - w_{dT90\ min}$ $w_{inc90\ min} = 305{,}8$ mm

$p_{90\ min} = \dfrac{w_{inc90\ min}^{2}}{0{,}375 L_{90\ min}}$ $p_{90\ min} = 49{,}9$

$L_{90min} \cdot \sqrt{(0{,}375\,\alpha \cdot T_{90min})} + \sqrt{0{,}375 \cdot L_{90min} \cdot p_{90min}} = 440$ mm

$F_{Ed\_5,0m} = \dfrac{g_{Panel} \cdot \left(L_{90\ min} \cdot 10^{-3}\right)^2}{8 \cdot w_{tot90\ min} \cdot 10^{-3}}$ $F_{Ed\_5,0\ m} = 1\,463$ N/m

EI60: $w_{tot60\ min} = 250$ mm $L_{60\ min} = ?$

$T_{60\ min} = 90$ °C

$w_{dT60\ min} = L_{90\ min} \cdot \sqrt{(0{,}375\,\alpha \cdot T_{60\ min})}$ $w_{dT60\ min} = 100{,}6$ mm

$w_{inc60\ min} = w_{tot60\ min} - w_{dT60\ min}$ $w_{inc60\ min} = 149{,}4$ mm

$p_{60\ min} = \dfrac{w_{inc60\ min}^{2}}{0{,}375 L_{90\ min}}$ $p_{60\ min} = 11{,}9$

When the span is $L_{60\ min} = 10\,900$ mm, then:

$w_{L60\ min} = L_{60\ min} \cdot \sqrt{(0{,}375\,\alpha \cdot T_{60\ min})} + \sqrt{0{,}375 \cdot L_{60\ min} \cdot p_{60\ min}}$

$L_{60min} \cdot \sqrt{(0{,}375\,\alpha \cdot T_{60min})} + \sqrt{0{,}375 \cdot L_{60min} \cdot p_{60min}} = 439{,}9$ mm

$F_{Ed\_10,9m} = \dfrac{g_{Panel} \cdot \left(L_{60\ min} \cdot 10^{-3}\right)^2}{8 \cdot w_{L60\ min} \cdot 10^{-3}}$ $F_{Ed\_10,9\ m} = 6\,955$ N/m

Some comments on the BS EN Extended application document

- great step forward as we now have a fire engineered approach to panel stability for longer-than-tested spans (previous tabular approach was rubbish).
- gives no derivation of deflection calculation.
- does not indicate how panel inward end movement may be calculated.
- confusion over what fire scenarios should be used (ie fire above or fire below the ceiling or both)
- some clauses are confusing.
- worked example very welcome, but more transparency needed (too pithy).

Confusing clauses in BS EN 15254-7: 2012

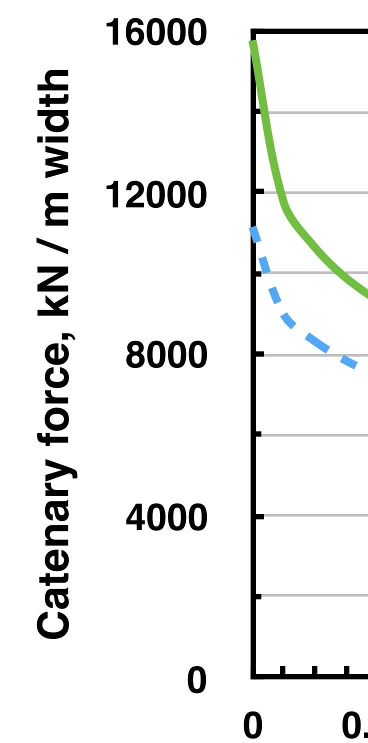
5.6 The rules given in 5.1 to 5.5 are valid for both cases *(fire above or fire below the ceiling exposed to EN 1363-1 fire resistance test conditions)*. Test results from a test with fire exposure from above the ceiling cannot be used for a situation with exposure from below the ceiling. OK ?? but why not vice versa

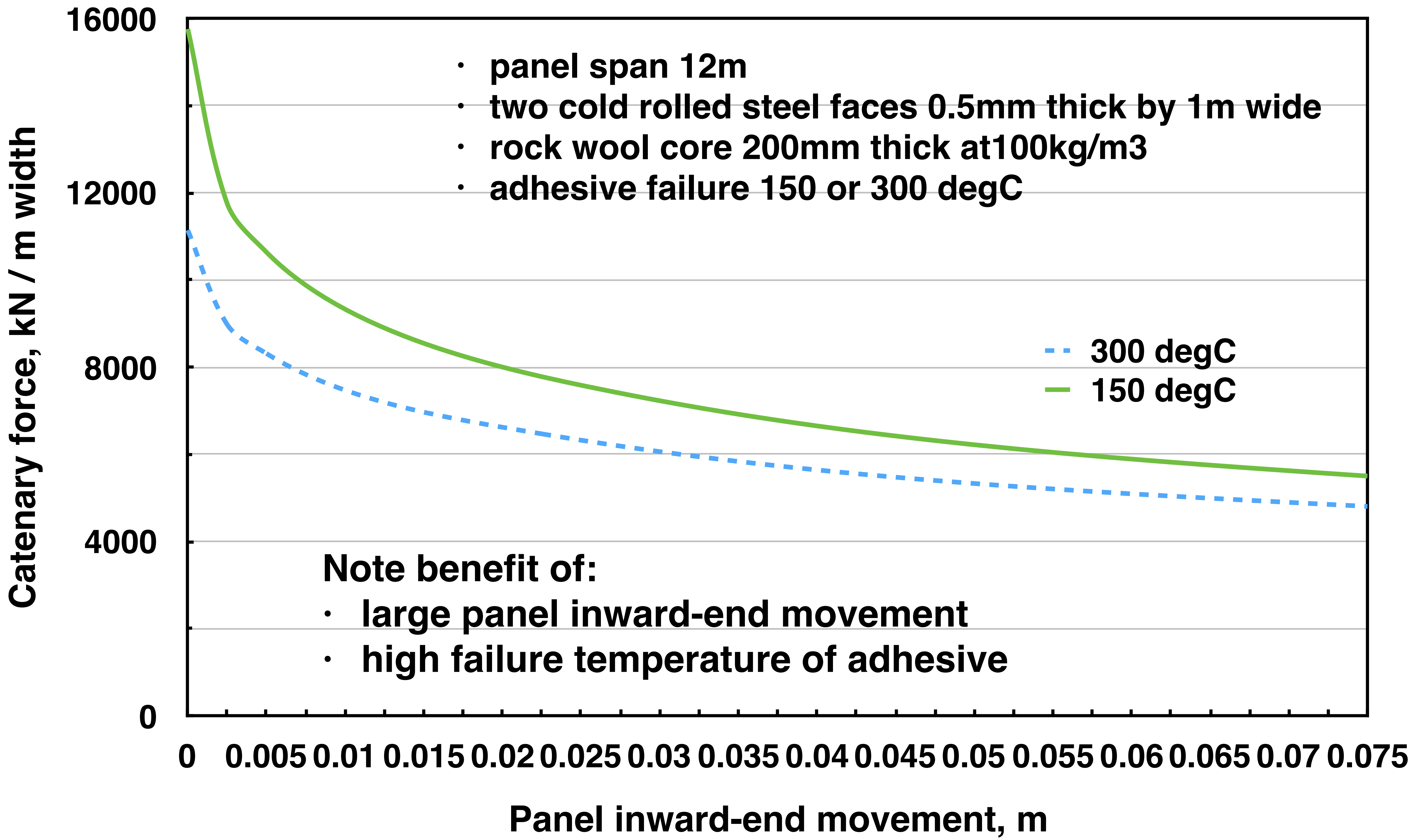
6.2.2.Calculations of panel-fastening capacity shall be made for both metal sheets assuming that both can carry the full dead load of the panel in non-exposed condition and the load of the steel sheet only in exposed condition. ??

Poor nomenclature, lack of clarity, rationale and transparency!

Table 1

	Sensitivity calculation of catenary force for ceiling sandwich panel with variable panel inward end movement, 6 april 2016, gordon cooke								
	Input values								
	panel width, m	**1**							
	panel span, m	**12**	near the upper limit of panel span						
	panel core mass, kg/m2	20	eg 200mm thick rockwool at 100kg/m3						
	mass of 2 panel faces, kg/m length.	7.85	panel face = 0.5m thick. volume of one face/m = 0.0005*1*1 = 0.0005 m3 : mass of one face = 0.0005*1*7850 = 3.925kg/m length, therefore 2 faces have mass of 7.85kg/m length						
	mass of core and faces, kg/m length	**27.85**	ie 20 + 7.85						
	panel inward end movement, m (Variable)	**0**	**0.02**	**0.04**	**0.06**	**0.08**	**0.1**		
	hot face temperature rise, degC (variable)	**150**	**300**	assumed debonding dictated by temperature for substantial weakening of adhesive due to fire. Values chosen at upper and lower end of commercially used adhesives (BRE)					
	Parameter values								
	density of steel, kg/m3	7850							
	acceleration due to gravity, m2/s	9.81							
	steel expansion coefficient/degC (alpha)	**0.000012**							
	Formulas								
	dead load, kN/m length	9.81 x panel mass/length = 9.81x 27.85 = **273** kN/m							
	mid-span deflection - thermal, m	**L(0.375 alphaT)^0.5**	Cooke derived equation (also used in EXAP BS EN 1524-7: 2012)						
	mid-span deflection - end movement, m	**(0.375Lp)^0.5**	Cooke derived equation (also used in EXAP BS EN 1524-7: 2012)						
	w = total mid-span deflection, m	equals 'thermal' and 'end-movement' components of deflection =12*(0.375*0.000012*300)^0.5 + (0.375*12*$B23)^0.5 (for T=300							
	Catenary force, kN	**(gL^2)/8w**	Ignoring any live load on panel = 273*12*12/(8*$C23) for T= 300 or $E23 for T =150						
	Results								
p		w, 300C	F, 300C	w, 150C	F, 150C				
0		0.440908153700972	11145.1783 296635	0.311769 1453623 98	15761.66 23488768				
0.0025		0.546974170878954	8983.97083	0.417835	11760.61				





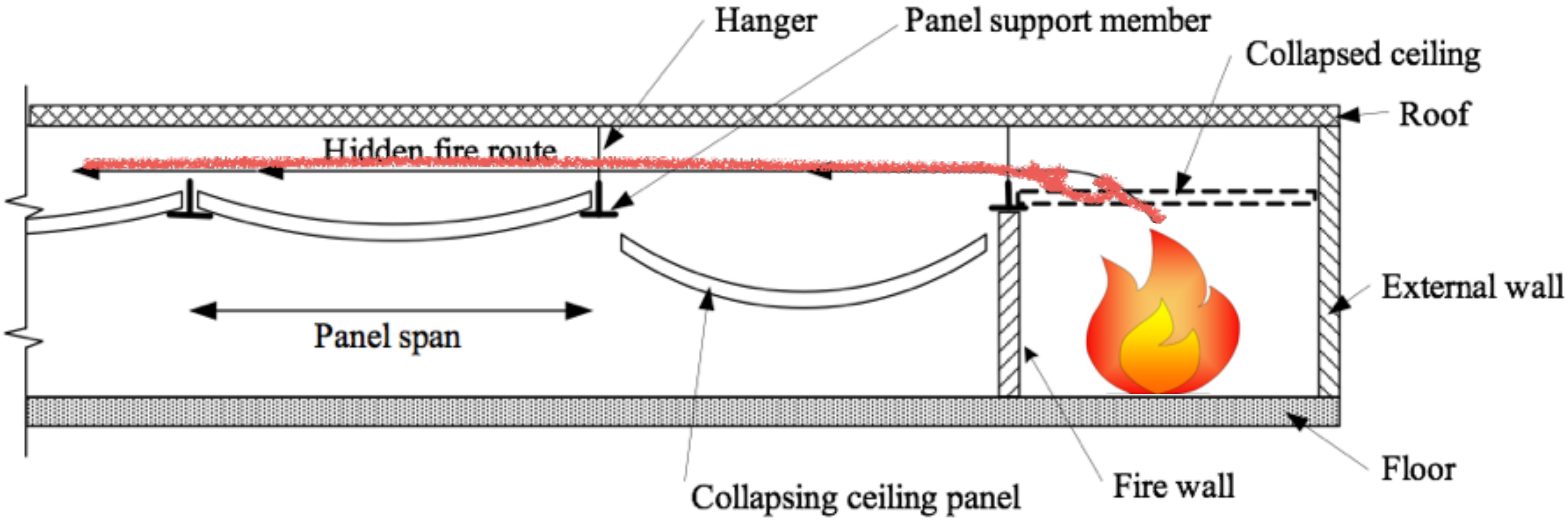
Variation of catenary force for 12m long panel with variable panel-end movement

Why no collapse in fire?

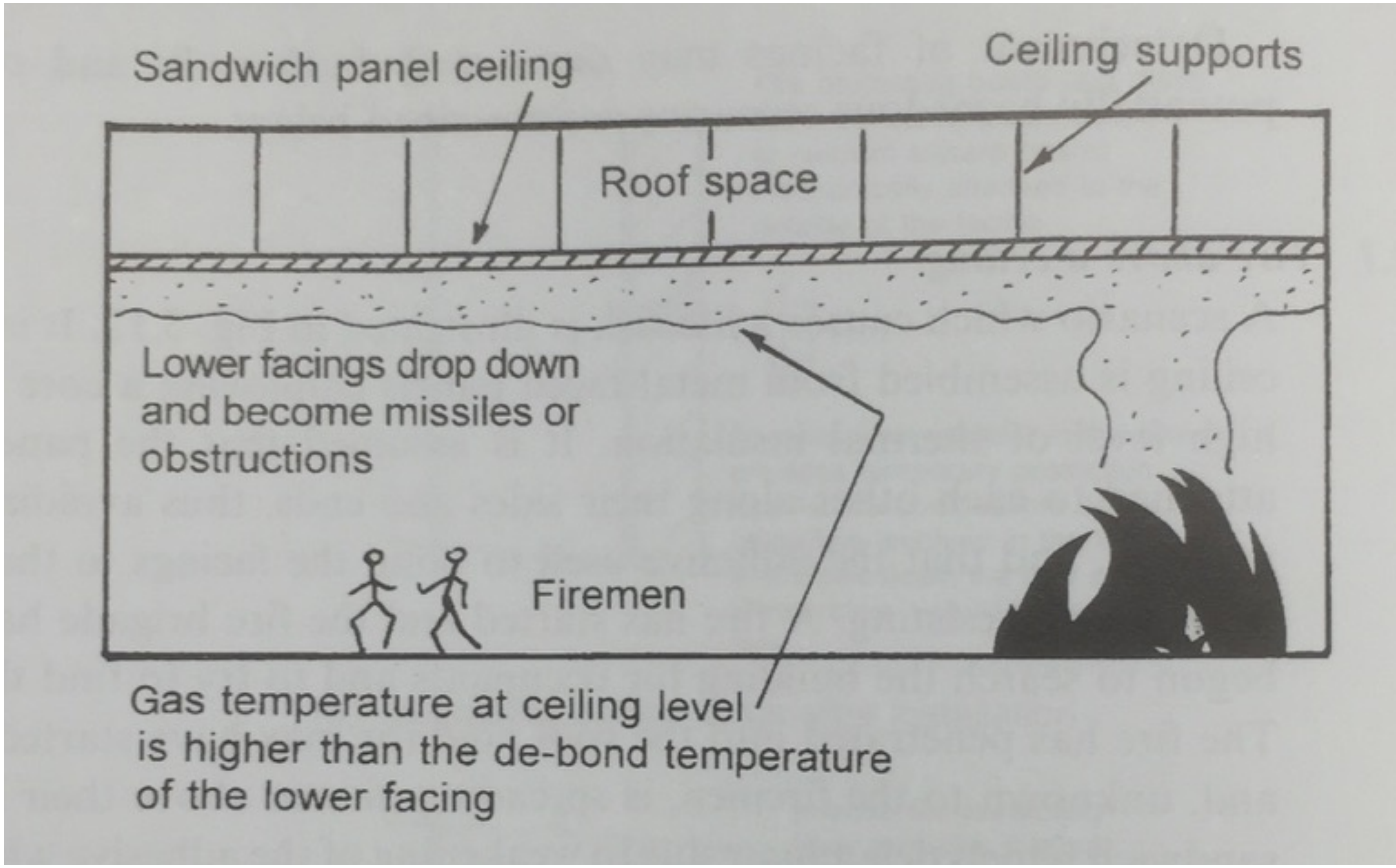
- Panel specifier recognised need to resist catenary forces in one or both faces. In the past unlikely! Fire risk assessment may have identified the hazard and perhaps sprinklers have been retrofitted, or
- fire severity trivial such that bond failure temperature not reached, or
- panel end fastenings were adequate, or
- catenary force reduced to sustainable level by large panel inward end movement, or
- post-fire investigation cannot reliably establish at what time in the fire the ceiling collapsed. Often one cannot tell from random fire debris when ceiling collapse occurred.

Does it matter if ceiling collapses?

- **Unlikely, if fire is below the panel because life there would probably be untenable except for a fire fighter suitably clothed.**
- **Yes, if fire is above the panel and people below, e.g. fire fighters, don't know about fire above and possibility of imminent collapse. Sun Valley scenario.**
- **Yes, if concerned about property protection and life safety of people in storeys above the fire - ceiling may contribute to fire resistance of structure/services and compartmentation.**



Dangerous hazard of fire **above** a panel ceiling.



Hazard to fire fighters, fire **below** the ceiling

Further guidance needed

BS EN needs accompanying document to explain a) how the 2-component deflection equation was derived and b) how to undertake the analysis of panel-end movement calculation

clarity on why the BS EN EXAP method is being used - if strictly to confirm end fastenings are strong enough for the extended span under the ISO 834 standard fire test exposure conditions, all well and good . If used because there is no alternative code guidance one needs to understand the fire scenario being simulated.

fire scenarios need to be clarified- (fire above or below?)

need statement in design on how panel relative end-movement is calculated (not easy as boundary conditions are often ill defined)

need info on end fastening/face 'failure modes' and pull-out strength/ deformation data (this needs room temperature tensile test results)

References

BS EN 15254-7: 2012 Extended application of results from fire resistance tests — Non-loadbearing ceilings Part 7. Metal sandwich panel sandwich panel construction. Annex C (normative) Rules and calculations methods for extending the span length of sandwich panel ceilings (22p)

Cooke GME, Resisting collapse of steel-faced sandwich panel walls and ceilings exposed to fire, Journal of Fire Protection Engineering November 2008; vol. 18, 4: pp 275-290.

Davies J M (ed), Lightweight sandwich construction, (joint CIB-ECCS commission), pub Blackwell Science, 2001, pp 370.

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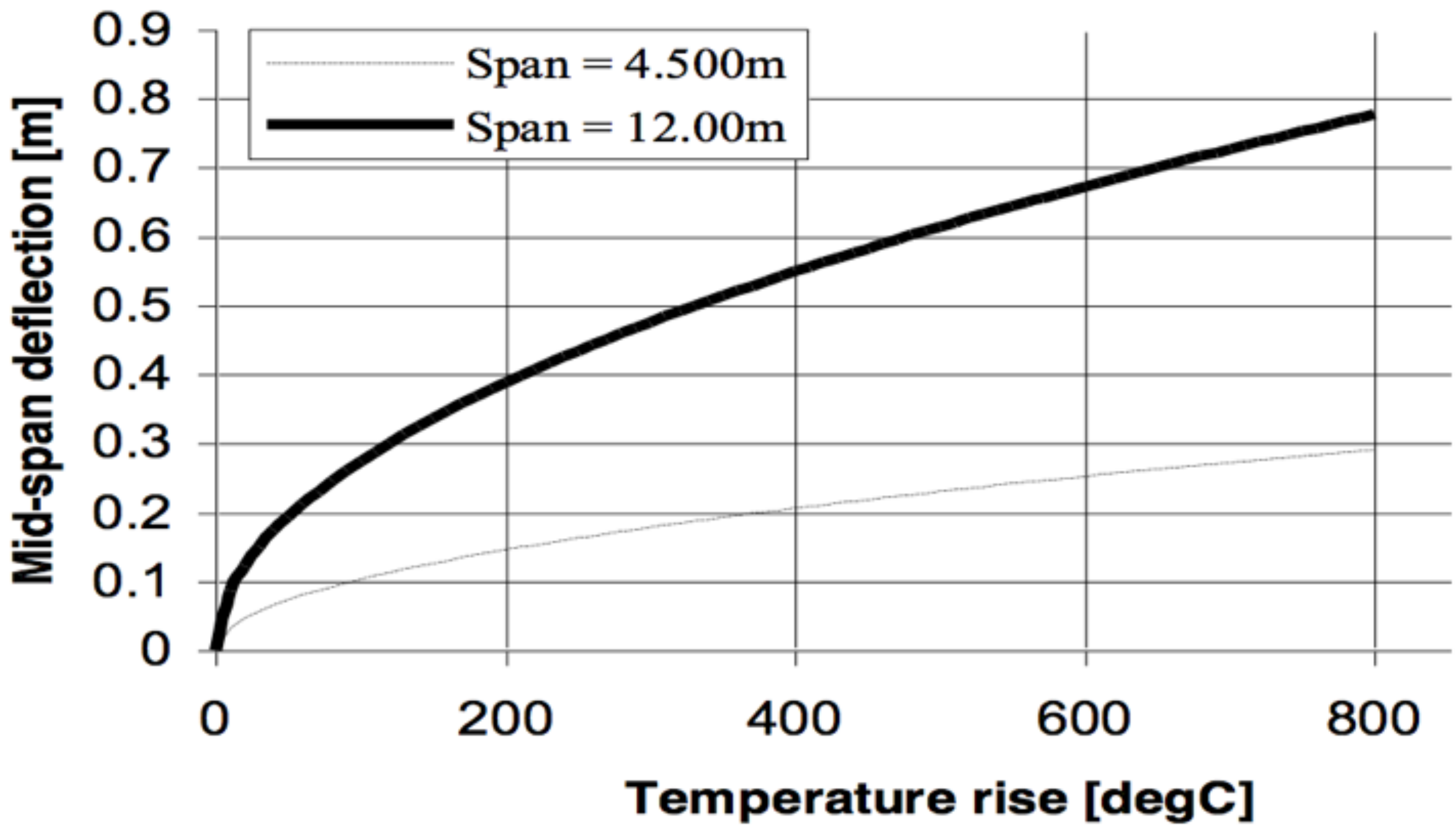
Span = 4.500m
Span = 12.00m
Mid-span deflection [m]
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
200
400
600
800
Temperature rise [degC]