

Resisting Collapse of Steel-faced Sandwich Panel Walls and Ceilings Exposed to Fire

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ABSTRACT: This article deals with the overall structural stability of sandwich panels used as ceilings and fire walls when exposed to fire on one side. Only panels with flat steel faces are considered. Simple mechanisms of instability are described, load actions are quantified, and practical design guidance is given to ensure stability i.e., resistance to overall collapse. The analysis for ceilings assumes that, after delamination of the fire exposed steel face due to softening of the adhesive, the panel faces act as catenaries, and calculations are made of mid-span deflection and the catenary force needed to resist collapse. This method is being considered in the work of European committee CEN TC 127 with regard to the development of rules for extended applications for construction products. The analysis for internal fire walls takes account of wind pressure. The importance of achieving adequate panel-end restraint is demonstrated.

KEY WORDS: structural engineering, fire risk assessment, sandwich panel, collapse, fire scenario, fire wall, ceiling, catenary force.

INTRODUCTION

SANDWICH PANELS COMPRISING flat steel faces and a lightweight structural core are often used as walls and ceilings in buildings where their long-span capabilities, high thermal insulation, clean design, rapid installation, and low maintenance make them the preferred choice of designers and building owners.

The fire performance of sandwich panels can be excellent if the correct core material is used and, importantly, if the steel faces are adequately restrained. For example, a fire resistance of 4 hours can be achieved using panels with 0.7 mm thick sheet steel faces and a 150 mm thick noncombustible rock wool core.

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Where sandwich panels are used in cold storage, there is the potential problem of cold-bridging between the faces wherever there is a metallic through-fixing, and this has led to designs which work well in normal conditions but allow panels to collapse very early when exposed to fire because the faces are not tied back to the supporting structure. Such a collapse is a fire hazard to fire-fighters, as shown in the 1993 fire [1,2] in the Sun Valley poultry factory in Hereford, UK in which two firemen lost their lives.

Sandwich panels are often used in single-storey and multi-storey buildings because they are lightweight, energy efficient, aesthetically attractive, and can be handled and erected easily. When constructed with noncombustible structural rock wool cores, panels have reasonable airborne sound insulation and fire resistance which can easily exceed 2 hours. Many panels employ combustible cores of foamed plastic e.g., polyurethane (PUR), polyisocyanurate and expanded polystyrene (EPS), which, in a fire, will delaminate and can produce large amounts of heat, smoke, and toxic gases that can be a hazard to life, property, business continuity, and the environment.

Most factory-produced sandwich panels have flat sheet steel faces bonded to the structural core using a thermosetting adhesive such as PUR. The steel faces, which are nominally 1 m wide, have tongue and groove profiles formed in the longitudinal edges of the steel faces prior to lamination of the core. Steel faced sandwich panels can span up to 12 m and typically have a cross section as in Figure 1.

The adhesive plays an important structural role by bonding the three elements (two steel faces and the core) together so that the panel behaves at

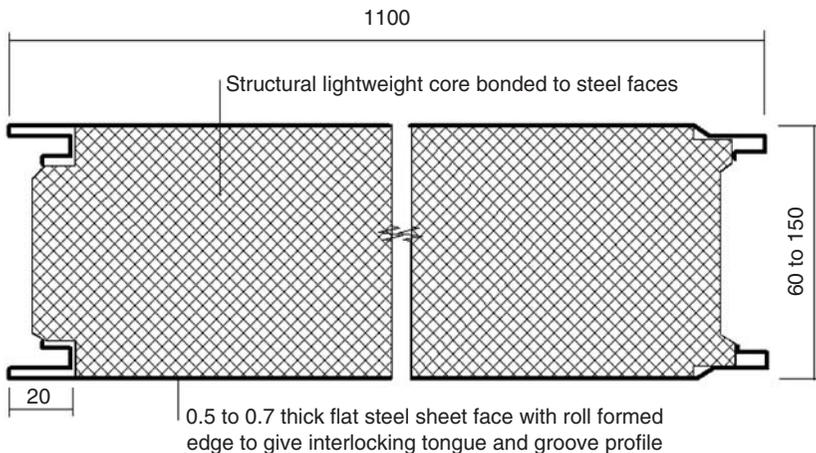


Figure 1. Cross section through typical sandwich panel. [Dimensions are in mm].

room temperature as a flexural member capable of spanning large distances without intermediate support. As the steel face exposed to a fire heats up, the adhesive loses its strength and the face delaminates from the core and all flexural strength is lost. The panel can then only behave as a ceiling or a wall if the ends of the steel faces are restrained to resist the action of the dead load and any other forces acting. For an internal fire wall, it may sometimes be necessary to resist wind loads as shown later.

Small-scale tests by the UK's Fire Research Station, Building Research Establishment, have shown that delamination occurs when the temperature of the fire-exposed steel face is in the range 130–350°C. This range is the result of the different softening points and shear strengths of core material and adhesive used. This means that panels can delaminate and collapse well before flashover (usually assumed to occur when combustion gas temperatures reach 600–650°C) unless the panel faces are adequately restrained.

Sandwich panels used as external wall and roof cladding are attached to a supporting structure which prevents both panel faces from falling down in a fire. However, when used as ceilings and free-standing internal walls, sandwich panels can collapse if the faces are not adequately restrained and this has frequently happened in buildings used for cold storage which typically employ panels with an EPS core.

If a fire resistance test on a representative specimen has been successfully conducted, it is unlikely that collapse will occur in the building context for an equivalent fire severity, similar size of panel and adequate panel restraint. However, not all sandwich panel constructions are tested for fire resistance and an assessment then needs to be made for panel stability, especially if the panel is much longer than its fire-tested counterpart. This can pose a problem as the span of ceiling panels and internal fire wall panels in a building can be up to 12 m but the fire test will probably have been conducted in a furnace which can only accommodate a span of up to 3 m for a wall specimen and 4.5 m for a ceiling specimen.

This paper deals solely with theoretical aspects of structural stability in the fire condition, and it should be noted that the use of the term 'stability' here means the ability of the whole panel to remain stable, i.e., resist collapse; it is not intended to mean or include buckling-related behavior. Information on other aspects such as the fire load represented by combustible panel cores and appropriate fire scenarios and fire testing for such panels is available in references [3–5] while some preferred panel attachment methods are given in reference [6]. Following a number of damaging fires associated with plastic foam cored sandwich panels in the food industry, two codes of practice [7,8] have been published in the UK. A book on sandwich panels [9] has been published which comprehensively deals with all aspects of design and testing. The author has made a study [10]

of the difficulties of making a fire risk assessment that takes into account the hazard of using external panels with combustible cores. Inadequacies in *ad-hoc* fire tests for combustible-cored sandwich panels have also been reported [11]. Some other aspects of fire resistance, e.g., insulation and integrity of long span sandwich panels, are covered in reference [12].

THEORY

Here, equations are derived, which are used later in the article to determine the deflections and forces acting on sandwich panels after delamination has occurred and the faces act as catenaries (cable-like members). The equations are based on simple statics and geometry and ignore stress-related strains.

The Catenary Force Equation

The horizontal restraint force H needed to support a catenary of span L carrying a uniformly distributed load w per unit length having a mid-span deflection D , as shown in Figure 2, is given in classical engineering text books [13] in the form of Equation (1). In the case of the sandwich panels considered here, w is solely the dead load since imposed (live) loads are not considered and often not present in the ceiling context.

$$H = \frac{wL^2}{8D} \quad (1)$$

Note from Equation 1 that H becomes infinite as D becomes small. This has important implications in practice as shown later.

Equation Relating Mid-Span Bow to Axial End Displacement

The following theory relates the axial shortening F of a flexible member to the mid-span deflection D when the member bows into a circular arc

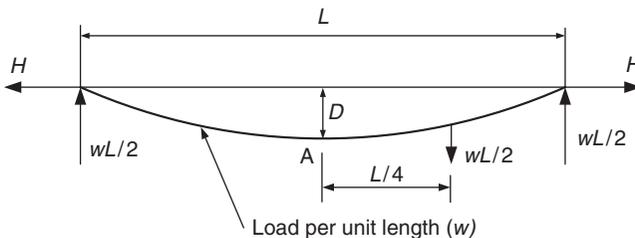


Figure 2. Catenary force diagram.

of radius R . Consider an initially straight member AB of length L , as shown in Figure 3, in which end A is fixed in position but allowed to rotate. End B is then pushed in by an amount F and this causes a mid-span deflection D . It is assumed that the member is slender so that the application of the axial compressive force at end B causes negligible elastic compressive strain in the material but simply causes it to bow into a circular arc ACE i.e., length of arc ACE equals length AB.

From geometry it can be shown [14] that:

$$D = \sqrt{0.375LF} \quad (2)$$

If member AB were heated along its length to give a temperature rise, ΔT , the unrestrained longitudinal expansion would be $\alpha L\Delta T$ where α is the coefficient of linear thermal expansion which may be taken as $14 \times 10^{-6}/^{\circ}\text{C}$ for steel at elevated temperature according to the structural Eurocode for steel. If, now, both ends of the member AB are fixed in position but allowed to rotate, and the member is then heated to give temperature rise, ΔT the expansion caused by the temperature rise would cause the member to bow into a circular arc. In this case $F = \alpha L\Delta T$ and substituting in Equation (2) gives:

$$D = L\sqrt{0.375\alpha\Delta T} \quad (3)$$

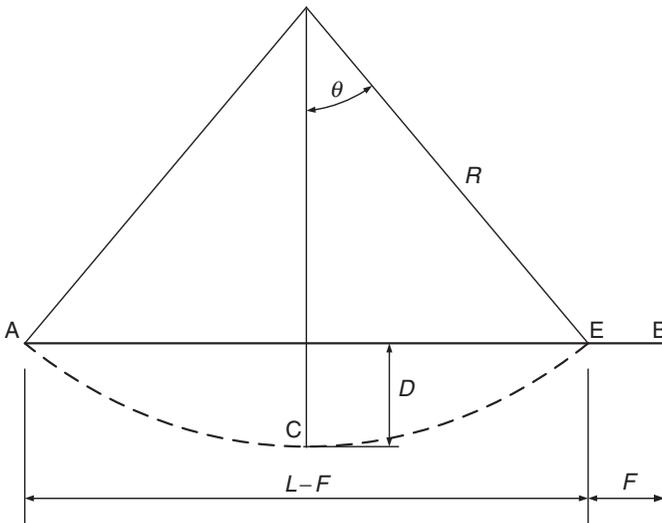


Figure 3. Geometry of bowed member.

LOADING AND MATERIALS DATA FOR ELEVATED-TEMPERATURE CALCULATIONS

The dead load of the face and core can easily be calculated from information on the volume and density of the construction materials. The density of steel sheet can be assumed to be 7850 kg/m^3 . The density of the core material at elevated temperature should be assumed to be the density at room temperature unless (a) there are appropriate data available on the time-dependant change in density due to the effects of fire exposure e.g., due to charring or significant reduction in moisture content, or (b) the core material is consumed in the heating process, as with EPS foam and PUR foam.

Because the steel used for the faces of sandwich panels undergoes rolling in the steel mill at room temperature, the grain structure is elongated and this leads to enhanced mechanical properties at room temperature compared with the family of low carbon steels used in hot rolled structural sections [14]. Upon heating, this strength enhancement is irreversibly lost and this means that the loss of strength of cold-formed steel at temperatures in the range $400\text{--}600^\circ\text{C}$ is 10–20% greater than that in hot rolled steel.

The reduction in strength properties of steel at elevated temperature may be assumed to vary according to the relevant national standard. In the UK, BS 5950: Part 8: 1980 gives strength reduction factors for hot rolled steel and cold-formed steel at different temperatures (strength reduction factor is the ratio of the elevated temperature strength to the room temperature strength). Alternatively, information in the structural Eurocodes could be used, for instance, Eurocode 3: Design of steel structures, Part 1.2, General rules.

The variation of linear coefficient of thermal expansion with temperature is not reviewed here as it proves to have very little effect, typically around 8%, on the value of catenary force calculated for large span panels.

CEILINGS

Structural sandwich panels rely on an adhesive layer between the flat steel faces and the core material for their flexural strength. Most adhesives used in proprietary panels delaminate at quite low temperatures – in the range 130 to 350°C according to tests carried out by the Fire Research Station of the Building Research Establishment, UK. These temperatures are reached in <5 minutes in the ISO 834 and ASTM E119 standard fire resistance test exposure and well before flashover in a real fire. If the panels simply rest on the end supports with no horizontal restraint, the panels will, on delamination, sag, and slip off the supports. Since panels can be 1 m wide and 12 m long, a collapsing panel is a substantial missile threat to occupants making their escape or more likely professional fire-fighters performing their

search, rescue and fire fighting duties. A particularly hazardous scenario occurs when fire breaks through the ceiling on one side of a fire wall and then travels unseen above the ceiling causing delamination and collapse while fire fighters are unaware of the fire. This scenario, shown in Figure 4, was the one in which two fire-fighters lost their lives [1,2].

To prevent collapse, the ends of the panel faces must be fastened to the supporting structure and horizontally restrained so that they act as catenaries (cable-like) structures. This can be done without forming a thermal bridge between the upper and lower faces, which is important in cold storage. A suitable construction detail is shown in Figure 5. Structural support to the panels is given by inverted T-members which are suspended from the underside of the roof or floor structure. The upper steel face is restrained by local steel straps. For the ceiling to resist collapse from fire attack in the ceiling void, the steel hangers need fire protecting or should be over-sized so that at elevated temperature they remain sufficiently strong to support the dead load of the ceiling. The supporting structure (e.g., roof beam) should also be fire resisting if the ceiling is to serve as a fire resisting membrane.

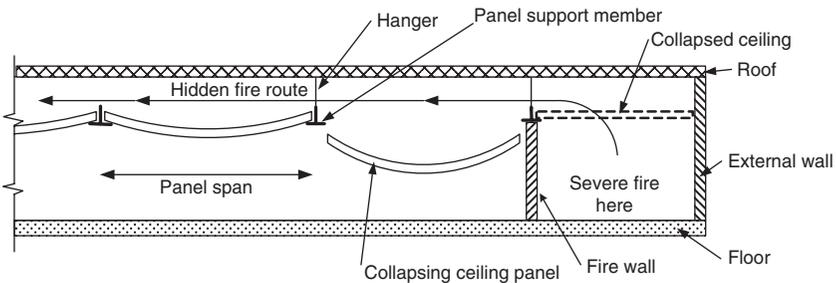


Figure 4. The hazard of unseen fire above the ceiling.

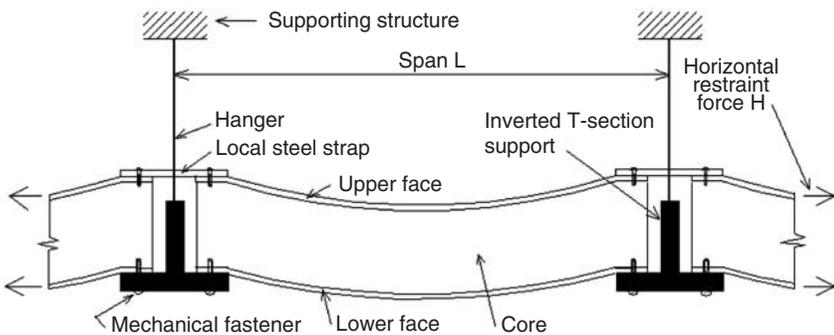


Figure 5. A practical design of support for ceiling panels (in fire condition).

The catenary forces in the upper and lower faces can be large and some attempt should be made to calculate them to ensure that the mechanical fasteners at the panel-end do not fail. In Equation (1) it was shown that for a simply-supported panel of span L , the horizontal force H needed to support the catenary is $wL^2/8D$.

Deflection D may be caused by thermal expansion of the face and by inward displacement of the panel ends due to in-plane flexibility of the panel assembly. Both effects are beneficial because they increase D and thereby reduce H .

Before Equation (1) can be used, D must be calculated or estimated. This can be done if the temperature of the face and/or inward panel end movement can be estimated. It is assumed in Equation (3) that the face hangs in the shape of a circular arc which is a reasonable assumption. The term, inward panel end movement, means the inward movement of one end of a face relative to the other end of the face caused by:

- movement of the support members, e.g., dragging together the inverted T-sections in Figure 5 caused by the tensile catenary force in the panel
- slippage in the mechanical fastenings relative to the face and
- elongation of the fastening holes in the panel faces.

Using Equations (1) and (3) calculations have been made for the catenary force and mid-span deflection as a function of temperature rise for one steel face which is 1 m wide by 0.7 mm thick. Two panel spans were chosen – 4.5 m and 12 m – to illustrate the effect of panel span and also the large increase in catenary force associated with the 12 m span. From Figure 6, assuming the adhesive fails when a temperature rise of 200°C has occurred, a mid-span deflection of ~400 mm will be produced for the 12 m span and, from Figure 7, this causes a catenary force of ~2.5 kN. For the same

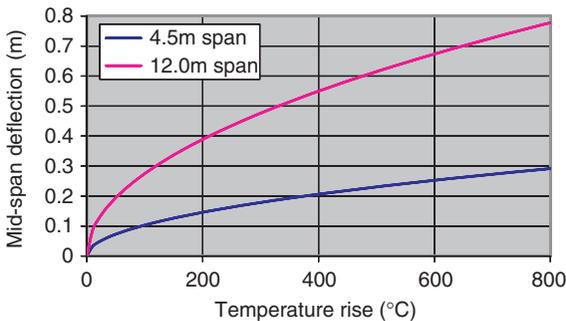


Figure 6. Variation of mid-span deflection with temperature (The color version of this figure is available online).

temperature rise in a 4.5 m fire test span, the catenary force is ~ 1 kN. The catenary force in the 12 m span is 2.5 times that of the fire test span and the number of fastenings would have to be increased accordingly. Similarly, from Figures 6 and 7, if an adhesive is used which fails at 100°C , the catenary force for the 12 m span will be ~ 3.5 kN which is a large increase in the catenary force for failure at 200°C . This demonstrates that the higher the failure temperature of the adhesive, the smaller the catenary force, and the smaller the force to be transmitted by the panel end fastenings. Similar calculations can be made for the effect of inward panel end movement using Equation (2). It can also be shown that adopting extreme values of coefficient of linear thermal expansion makes little difference to the calculated catenary force: using $\alpha = 0.000012$ (value at 20°C) instead of the elevated temperature value of 0.000014 for steel increases the catenary force by only $\sim 8\%$ for a 12 m span. It is suggested that a value of $14 \times 10^{-6}/^\circ\text{C}$ should be adopted.

Fire Exposure from below the Ceiling

When the lower face is exposed to fire each panel initially bows downwards due to thermal bowing as in the bimetallic strip effect. Delamination of the fire exposed face occurs when the strength of the adhesive layer is lost. The flexural strength of the panel assembly then approaches zero and collapse will occur unless one or both faces are restrained horizontally at the panel ends so that they become catenaries.

If only the lower face is horizontally restrained the catenary force in that face is a maximum because the dead load of the whole panel (upper face, core and lower face) has to be carried by the lower face and its fastenings. The catenary force can be beneficially shared between both faces if both faces are horizontally restrained as shown in Figure 5.

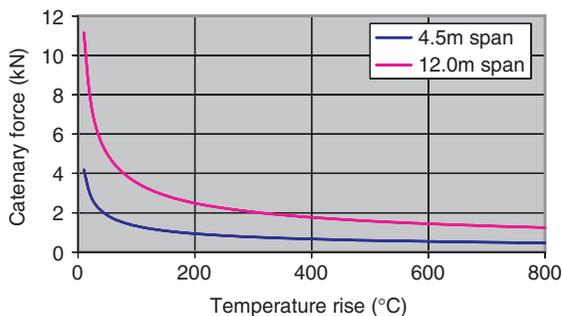


Figure 7. Variation of catenary force with temperature (The color version of this figure is available online).

Fire Exposure from above the Ceiling

When the upper face is exposed to fire each panel initially bows upwards. Delamination of the fire exposed face again occurs when the strength of the adhesive layer is lost. The flexural strength of the assembly then approaches zero and collapse will occur unless at least the lower face is restrained horizontally at its ends so that it becomes a catenary. Again the catenary force can be beneficially shared between both faces if both faces are horizontally restrained. It should be noted that the catenary force in the lower face will be large because there is no beneficial sag in the lower face in the absence of a temperature rise in the lower face. However the transfer of the dead load of the upper face and core onto the lower face will cause some beneficial sag due to in-plane flexibility and consequent beneficial panel inward end movement of the whole panel assembly. However some components of the panel inward end movement are not easy to quantify in this context and expert professional judgment is necessary.

FREE-STANDING INTERNAL FIRE WALLS

Practical Design

A practical panel head detail for vertically oriented panels is shown in Figure 8. The steel angles are securely fixed to the roof (or the underside of floor slab). The vertical legs of the angles have slotted holes to allow the roof or floor structure to move up and down relative to the top of the panels in normal use: such movement can occur due to changes in live load and thermal expansion effects in large roof spans. In a fire the panel face slides down until the fastening is limited by the bottom of the slot and the delaminated face then hangs like a curtain. This movement is beneficial if the panel is subjected to wind pressure as it provides some slack after delamination and reduces the catenary force.

The rock wool fire stopping, therefore, has to be sufficiently resilient and flexible to accommodate repeated compression in normal use and also, importantly, fill the gap when the faces drop in the fire condition without leaving a void. If the panels are subjected to wind pressure as in Figure 9, the steel angle also has to withstand the horizontal shear force. Unless fire protected, the fire-exposed steel angle will be at elevated temperature and allowance should be made for its reduced strength if left bare. The nonfire-exposed steel angle will, however, retain its room temperature strength as its temperature rise cannot be permitted to exceed 150°C if the panel is to

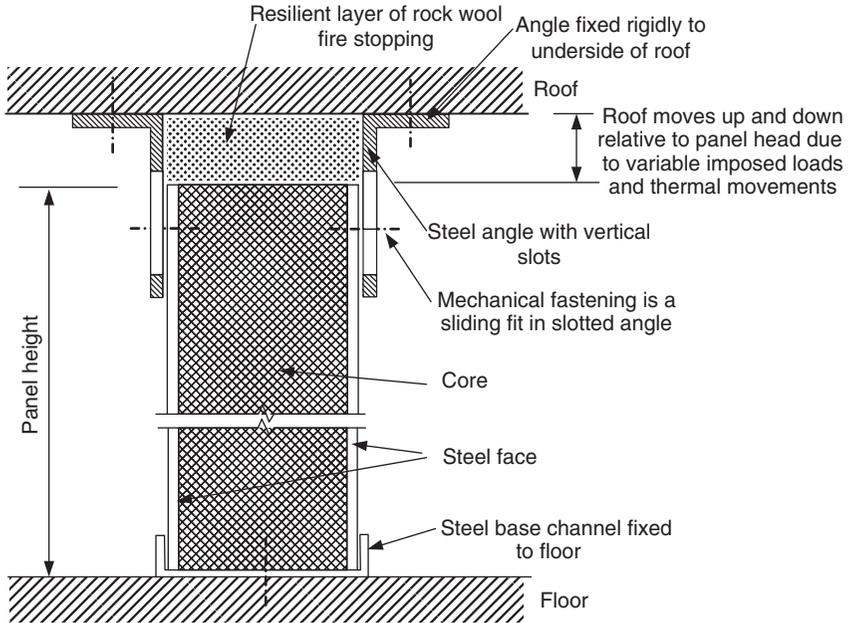


Figure 8. A suitable panel head support detail.

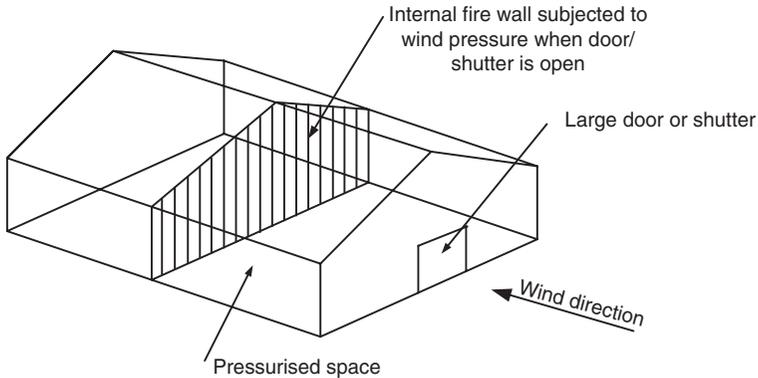


Figure 9. Wind acting on a fire wall through an open door or shutter.

satisfy the criterion for unexposed face temperature in the standard ISO 834 fire resistance test. Because the direction of fire attack in the building is usually not known the strength of fastenings for each face should be checked for the relevant dead load and relevant temperature and the more onerous condition should be catered for.

Dead Loading

The loads acting on an internal fire wall will be the dead load of the wall and also, in some cases, wind load. These load actions are now discussed. It is again assumed that the panels are vertically oriented.

The stability of free-standing sandwich panels forming a fire wall is achieved by attaching both steel faces of the panel at the top to a roof beam which has the required fire resistance. In a fire the panel loses its flexural strength when a face delaminates from the core and the panel faces then becomes suspended from the top like curtains. In this condition, and in the absence of wind pressure, the fire-exposed face carries only the dead load associated with that face but the nonfire-exposed face to which the core remains attached has a dead load of that face and the core.

For a 12 m high by 1 m wide by 0.7 mm thick steel face, the dead load is obtained so:

$$\text{Volume} = 12000 \times 1000 \times 0.7 = 8.4 \times 10^6 \text{ mm}^3$$

$$\text{Density of steel} = 7850 \times 1000^{-3} \text{ kg/mm}^3$$

$$\text{Mass} = \text{volume} \times \text{density}$$

$$\text{Therefore mass} = 65.94 \text{ kg}$$

$$\text{Dead load for 12 m span} = 65.94 \times 9.81 \text{ N/m} = 646 \text{ N per m width of face.}$$

In a similar way, the dead load for a 12 m by 1 m by 150 mm thick core of rock wool of density 135 kg/m^3 is 2383 N per m width of face (this thickness and density of rock wool can satisfy the insulation criterion in the standard fire resistance test for up to 4 hours and represents the largest core dead load likely to be met in practice). In this example, the fastenings at the top of the unexposed steel face need to withstand a total dead load of 3029 N per m width (i.e., $646 \text{ N/m} + 2383 \text{ N/m}$) in the ambient condition. The fastenings for the fire exposed face need to withstand only 646 N per width but this may become onerous as the fire progresses and the fastenings reach elevated temperatures and lose strength. Because the direction of fire attack is not known in most projects, the worst dead load condition must be catered for in both faces.

Wind Loading

The effect of wind blowing on a large (dominant) opening in the external wall of a compartmented building subdivided by an internal fire wall, Figure 9, sometimes needs to be considered as the internal wall will be subjected to wind pressure, although of a lesser magnitude to that acting on the external wall.

Although, the room-temperature design of the sandwich panel fire wall may allow for the effect of wind pressure, the forces imposed by wind when acting simultaneous with fire needs will need to be considered. Implicit in this

consideration is the probability of high wind occurring at the same time as fire, the probability of wind being in an onerous direction and the probability of the large door or shutter being open at the time of the fire. In a fire engineered approach one strategy might be to have the door or shutter closed automatically on detection of fire so that the difficulty can be avoided.

In the ambient-temperature design, the panel will act as a composite flexural member, but in fire the two panel faces will act as curtains suspended from their upper edges. Wind pressure will cause the delaminated panel faces to act as vertical catenaries and this will add to the effect of the dead load at the top of the panel resulting in larger forces to be withstood by the mechanical fastenings at the panel top.

The calculation below assumes that the principle of superposition applies – that is that the catenary force generated by wind pressure can be simply added to the dead load force. To calculate the wind force, the basic wind speed is first selected, the design wind speed is then determined and hence the corresponding dynamic pressure. This is dealt with in national standards. In the UK, guidance on wind loads is given in BS 6399 Part 2.

Referring to Figure 10, force P1 is the force needed to support only the dead load assuming the delaminated face hangs like a curtain. It is clear that force P2 restraining the top of the panel face under the action of dead load and the wind-induced catenary force will be greater than P1. Force P3 at the bottom of the panel face will be solely the catenary force. In a practical design, the shear force arising from the reaction force R

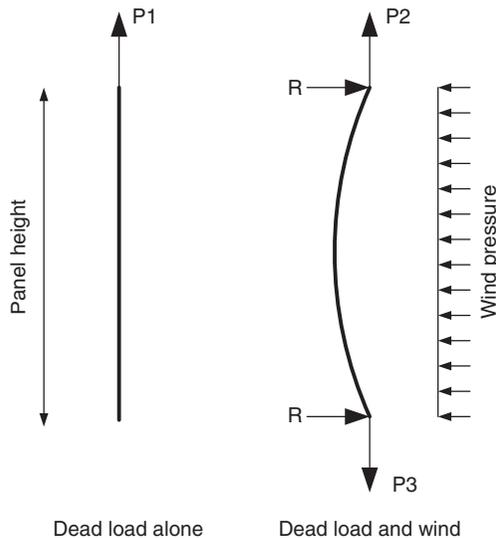


Figure 10. Effect of wind pressure on panel support forces.

will also need to be considered. The main objective is to calculate the maximum force, P_3 , which is transmitted into the supporting floor or roof structure above.

Example Calculation of Panel Support Force Taking Account of Wind Pressure

Assumptions:

- The calculation is confined to the forces acting on only one steel face
- Panel height = 12 m
- Panel width = 1 m
- Basic wind speed = 50 m/s
- Design wind speed = 44 m/s (which gives a dynamic pressure of 1.20 kN/m² acting on the external windward face)
- Internal pressure coefficient in fire compartment = 0.2
- There is a large opening in the external wall in the windward face of the building when fire occurs.
- Mid-span panel deflection $D = 0.30$ m. This value is determined from Equation (3) and corresponds, in this example, to a temperature rise in the 12 m long steel face of 100°C.

Note: In a practical design, it is necessary to make the calculation of D for the smallest temperature rise that the unexposed steel face might attain (since the smaller the temperature rise, the smaller the deflection, and the larger the catenary force) and it is suggested that at least two temperature rises are considered. The first should be the temperature rise in the steel face at the time of delamination, obtained from fire test data on a specimen of the panel construction. If this temperature rise is, say, 100°C then, from Equation (3), D becomes 0.30 m, as in the following calculation example. The second should be the temperature rise which is necessary not to exceed the insulation criterion in the ISO 834 standard fire resistance test exposure, i.e., that the average temperature rise of the unexposed face should not exceed 150°C during the test, and in this case D would be 335 mm.

Calculation

$$\begin{aligned} \text{Total wind force on panel} &= \text{pressure coefficient} \times \text{panel area} \\ &\quad \times \text{dynamic pressure} \\ &= 0.2 \times 12 \times 1 \times 1.2 = 2.88 \text{ kN} \end{aligned}$$

$$\text{Wind force per unit area} = \frac{2.88}{12} = 0.24 \text{ kN/m}^2$$

From Equation (1), the end restraint force needed to support the catenary for this example, when $D = 0.30$ m, is:

$$H = \frac{0.24 \times 12^2}{8 \times 0.3} = 14.40 \text{ kN}$$

From the earlier calculation of dead load, the dead load for one steel face $12 \text{ m} \times 1 \text{ m} \times 0.7 \text{ mm}$ is 0.646 kN . This load is small when compared to the wind-induced catenary force of 14.40 kN . The total force to be transmitted through the mechanical fastenings at the top of the panel is, therefore, 15.046 kN (i.e., $0.646 \text{ kN} + 14.40 \text{ kN}$) for one steel face alone i.e., ignoring dead load of the core. Since it is the unexposed steel face which will transmit all the wind force and this face will have the core bonded to it, a more realistic assessment of the total force to be transmitted at the top should include the dead weight of the core. If the core was of 150 mm thick rock wool (the maximum foreseeable, the associated dead load would be 2.383 kN (as shown earlier under Dead Loading) and in the example here the total force to be transmitted at the top would become 17.43 kN . This force is very large when compared to the force resulting from the dead load alone.

Snow Loading

Fire is an accidental limit state and because the simultaneous occurrence of fire and snow is unlikely in many countries, the reserve of strength in the roof structure needed to carry the full snow load can sometimes be utilized to carry the panel dead load in the fire condition so that the strength of the roof structure does not have to be increased to carry the dead load of the delaminated panels.

CONCLUSIONS

Sandwich panels have many advantages. Care is needed, however, to ensure that premature instability and collapse does not occur in a fire. Practical methods of retaining the stability of panels have been described and a theory developed for calculating the catenary forces which occur in ceiling panels when a steel face delaminates from the core due to fire. Fire exposure from above a ceiling causes large catenary forces and this is unfortunate as a fire in the ceiling void may go unseen by people, e.g., fire fighters, below the ceiling, and unexpected collapse could present a life hazard. Catenary forces may also need to be considered in internal fire walls when subjected to wind pressure occurring at the same time as the fire. Example calculations have been presented for deriving dead loads and catenary forces acting under fire conditions.

NOMENCLATURE

- D = mid-span deflection (m)
 L = span of panel or steel face (m)
 w = uniformly distributed load per unit length (kN/m)
 ΔT = temperature rise of steel face ($^{\circ}\text{C}$)
 H = catenary force (kN)
 α = coefficient of linear thermal expansion ($^{\circ}\text{C}^{-1}$)

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