Structural Fire Analysis of One-storey Steel-framed Buildings with Steel Claddings

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Abstract

Traditionally the structural stability of steel building frames is ensured by frame action or bracing elements. Recent European research projects have demonstrated that significant cost savings can be achieved, if cladding panels forming the building envelope are used to provide stability. The purpose of the paper is to investigate the possible stabilization effect of steel structures by steel claddings in fire, without additional wind bracings using Finite Element Method. The Finite Element analyses for full-scale steel-framed buildings with steel claddings under fires are still challenging due to large number of elements in the steel claddings. In this paper, the equivalent orthotropic shell element is used to model the trapezoidal roof sheeting, and composite shell element with soft core is used to model the sandwich panel in the cool compartment. Due to the delamination of inner steel face in fire, the sandwich panels in the fire compartment can be either neglected or modeled with two-layer composite shell ignoring the inner steel sheet. The comparative study is performed, and the results suggest that the difference is small and the sandwich panels in the fire compartment can be neglected. The stabilization effect of steel claddings in fire is studied by the comparison of deformation and force responses between buildings with traditional steel bracings and with steel cladding only. In case of ISO fire, the fire resistance of entire one-storey building with steel claddings only is 22% longer than the one with traditional steel bracings.

Keywords

Steel claddings, steel-framed building, ISO fire, stabilization effect, trapezoidal sheet, sandwich panel

1 Introduction

The fire tests of entire building in case of fire are expensive, and the detailed behavior of structures in fire is difficult to monitor and record with high resolution due to elevated temperatures. The development of computer modeling techniques makes it possible to simulate the structural behavior of entire one-storey building in fires. This paper describes the relevant techniques and model parameters, used in modeling of a one-storey portal framed steel building with building envelope in ISO standard fire. The building envelope consists of sandwich wall panels and trapezoidal roof sheeting.

The goal of this study is to investigate the stabilization effect provided by the building envelope in fire, without additional steel bracing system. Studies have shown that the sandwich panels and trapezoidal building envelopes are able to stabilize the steel structure to resist the wind loads at normal temperature. The research question is whether this stabilization effect exists also in case of fire, especially in resisting the horizontal wind loads and also the horizontal catenary action due to vertical snow loads.

The computer modeling of entire building with steel claddings is still challenging since large number of elements and long fire analysis duration. The computer modeling techniques, including the equivalent orthogonal plate for trapezoidal sheeting, modeling of sandwich panels at normal and elevated temperatures, modeling the fasteners such as shot-nails and self-drilling screws at elevated temperatures, thermal histories of structural elements and boundary conditions, are developed and described. After that, structural responses of a one-storey steel-framed building with steel cladding are analyzed, and the results are compared with the steel structure with traditional bracing system. The stabilization effect by steel cladding in fire is preliminarily investigated.

2 Finite element modelling of steel cladding

2.1 Equivalent orthotropic plate for trapezoidal sheeting

Traditionally, the interest on the equivalent orthotropic plate for a profiled sheeting is mainly on the out-of-plane flexural properties. Recently, there is increasing interest on the in-plane extensional properties in order to model the diaphragm action of the profiled panels. Figure 1 illustrates the notations of a trapezoidal sheet and the equivalent orthotropic flat plate. The constitutive equation of the equivalent orthotropic plate is
Table 1 summarizes the extensional and flexural rigidities of equivalent orthotropic plates from Bian et al [2] and Xia et al [3].

Table 1 Plate rigidities from Bian et al [2] and Xia et al [3]

| Uniform plate | Direct definition |
|---------------|-------------------|
| **Rigidity**  | **Isotropic**  | **Orthotropic** (engineering constants) | **Orthotropic** (Xia et al 2012) |
| \( \overline{A}_{11} \) | \( E_t \frac{h}{1 - \nu^2} \) | \( E_t \frac{h}{1 - \nu^2} \) | \( \frac{1}{2} I_2 \frac{E_t}{1 - \nu^2} + i \frac{1}{12} (1 - \nu^2) \) |
| \( \overline{A}_{22} \) | \( E_t \frac{h}{1 - \nu^2} \) | \( E_t \frac{h}{1 - \nu^2} \) | \( \frac{1}{2} I_2 \frac{E_t}{1 - \nu^2} + i \frac{1}{12} (1 - \nu^2) \) |
| \( \overline{A}_{12} \) | \( E_t \frac{h}{1 - \nu^2} \) | \( E_t \frac{h}{1 - \nu^2} \) | \( \frac{1}{2} I_2 \frac{E_t}{1 - \nu^2} + i \frac{1}{12} (1 - \nu^2) \) |
| \( \overline{A}_{66} \) | \( G_t \) | \( G_t \) | \( \frac{E_t}{G_t} \) |

An equivalent orthotropic flat plate with uniform thickness, cannot match all of the eight directly defined rigidities from Xia et al [3]. Most commercial structural engineering softwares at best allows the orthotropic engineering constants: \( E_1, E_2, G_{12}, G_{66}, t \) and equivalent thickness \( t_e \) to be defined. For diagrams the in-plane quantities are of greatest prominence. The equivalent extensional properties can be defined as follows:

1. \( E_2 = \frac{I_2}{t} E \)
2. \( E_1 = \frac{I_1}{I_{11}} E_2 \)
3. \( G_{12} = \frac{A_{12}}{A_{11}} E_2 \)
4. \( \nu_{21} = \frac{I_2 E_2}{I_1 E_1} \)
5. \( t_e = \frac{A_{11}^{-1} I_{12} + A_{12}^{-1} I_{12}}{A_{11}^{-1} I_{12}} \)
6. \( G_{12} = \frac{A_{66}}{t_e} \)

For trapezoidal steel sheet in Figure 1a

\[
I_1 = 2c - 2h \frac{\cos(1 - \cos a)}{\sin a} \]

\[
I_2 = \frac{2(h - x_0)^2 + x_0^2}{3(m_a^2 - 3m_0^2)} + r_1 (h - x_0)^2 + 2r_2 x_0^2 \]

where \( x_0 = \frac{r_1 h}{m_0} + \frac{b^2}{2(m_a - m_0)} \), \( c, h, r_1, r_2 \) and \( I \) are defined in Figure 1a.

From above we can see that there are two types of important properties for the equivalent orthotropic plate: bending rigidities in both \( x \) - and \( y \) - directions and extensional rigidities. For trapezoidal sheet subjects to transverse loading, bending rigidities dominate the structural response; for diaphragm action of building envelope, the extensional properties dominate the structural response. The ideal case is that the software is capable of taking both properties into mechanical formulation such as in Table 1. However, in the current version of LS Dyna (R7.71) [6], the orthotropic plate model with temperature-dependent properties is defined in MAT23 (Temperature_Dependent_Orthotropic). Elastic moduli and shear moduli can
be based on bending properties or extensional properties, not both. For trapezoidal roof panel in this study, the diaphragm action has the dominating role. The extensional properties $\lambda$ in Table 1 can be readily calculated. It is assumed that the elastic modulus $E$ is temperature-dependent, which can be taken according to EN1993-1-2 [1]. For Ruukki T153 roof panel, the extensional properties $E_1, E_2, Y_{12}, Y_{22}$ and $G_{12}$ are shown in Table 2. The equivalent thickness $t$ is 0.208 mm. These data are used in the FE analyses by Ls-Dyna in this paper.

Table 2 Temperature-dependent extensional properties for orthotropic plates of MAT23 in Ls-Dyna.

| Temperature (°C) | $E_1$ (N/mm²) | $E_2$ (N/mm²) | $Y_{12}$ | $Y_{22}$ | $G_{12}$ (N/mm²) |
|------------------|---------------|---------------|----------|----------|------------------|
| 20               | 200000        | 12.069        | 3.48E+5  | 1.04E-5  | 0.3              |
| 100              | 200000        | 12.069        | 3.48E+5  | 1.04E-5  | 0.3              |
| 200              | 180000        | 10.862        | 3.13E+5  | 1.04E-5  | 0.3              |
| 300              | 160000        | 9.655         | 2.78E+5  | 1.04E-5  | 0.3              |
| 400              | 140000        | 8.449         | 2.44E+5  | 1.04E-5  | 0.3              |
| 500              | 120000        | 7.242         | 2.09E+5  | 1.04E-5  | 0.3              |
| 600              | 62000         | 3.741         | 1.08E+5  | 1.04E-5  | 0.3              |
| 700              | 26000         | 1.569         | 4.52E+4  | 1.04E-5  | 0.3              |
| 800              | 18000         | 1.086         | 3.13E+4  | 1.04E-5  | 0.3              |
| 900              | 13500         | 0.815         | 2.35E+4  | 1.04E-5  | 0.3              |
| 1000             | 9000          | 0.543         | 1.57E+4  | 1.04E-5  | 0.3              |
| 1100             | 4500          | 0.272         | 7.83E+3  | 1.04E-5  | 0.3              |

2.2 Composite shells for sandwich panels

Ma et al (2017) showed that both composite shell model and shell faces + solid core model can achieve sufficient accuracy at ambient temperature. In case of fire, shell faces + solid core model is convenient in taking the face delamination into account at elevated temperature. However, the computational effort is huge for analysis of entire building. Therefore, three-layers composite shell model for sandwich wall panels is used for the cool compartment and the sandwich wall panels are removed for the fire compartment based on observations from fire tests. According to the studies at ambient temperature, the major contribution to the in-plane stiffness and strength of panel from the inner face. In case of fire, the inner steel face will delaminate from the panel cores and lose the functional contribution to the in-plane stiffness and strength. Two-layers composite shell model that neglects the delaminated inner face is also used for the fire compartment for comparison purpose in Section 5.1.

When the composite shell model is defined for sandwich panel in Ls-Dyna, the option LAMSHT=1 is set in *Control_Shell keyword. When this flag is activated, laminated shell theory is used. Lamination theory is applied to correct for the assumption of a uniform constant shear strain through the thickness of the shell. Unless this correction is applied, the stiffness of the shell can be grossly incorrect when there are drastic differences in the elastic constants from ply to ply, especially for sandwich type shells.

Element size for sandwich panels is 300 mm in width direction and 500 mm in length direction. Table 3 shows the definition of composite shell model for sandwich panel SPA230 in Ls-Dyna.

Table 3 Composite shell model definition in Ls-Dyna

| Ls-Dyna | Material | Thickness (mm) | Material | Thickness (mm) |
|---------|----------|----------------|----------|----------------|
| *Part_Composite | Steel | 0.5 | - | - |
| | MW | 76.6 | MW | 76.6 |
| | MW | 76.6 | MW | 76.6 |
| | Steel | 0.6 | Steel | 0.6 |

3 Geometry and loads of building

The one-storey steel-framed buildings (Figures 2 and 3) with three mid-frames (D, E, and F) are engulfed of ISO fire are analyzed. Two following cases are studied:

1) In fire, as designed with stabilization for portal frames by roof and wall bracing systems, no steel claddings considered;
2) In fire, including the sandwich panel wall and trapezoidal roof sheeting, without roof and wall steel bracing systems

The purpose is to study the fire resistance behavior of the reference portal frame building under fire with and without stabilization action of building envelops. The research question is: do the steel claddings provide similar stabilization effect in fire to the traditional steel bracing system?

3.1 Geometry

The portal frame steel building with traditional steel bracing system is shown in Figure 2, and the one with steel claddings is shown in Figure 3.
The basic data of the designed portal frame solution for the middle frames are:

Wall column: HEA600, grade S355; Rafters: HEA500, grade S355; Struts: RHS180x180x10, grade S355; Braces: RHS 120x120x5, grade S355

The width of building is 30.75 m (5 x 6.15 m) and frame spacing is 6 m; the height of wall column is 10m and maximum height of pitched roof is 11.5 m; the slope of the roof is 1:10 (5.7°).

### 3.2 Loads

Self-weight of roof is 0.5 kN/m², self-weight of wall panels is 0.3 kN/m². Ground snow load is 2.5 kN/m² and shape coefficient is 0.8. The wind peak velocity pressure is 0.65 kN/m². The utilization ratio at normal condition is 0.95 for column and 0.71 for rafter.

The load combinations for permanent load, snow load and wind load can be expressed as [1]

\[
\text{Load Case 1: } G + 0.4S \\
\text{Load Case 2: } G + 0.25 + 0.2W
\]

Where G is the permanent weight, S is the variable snow load, W is the variable wind load. The variable ground snow load is 2.5 kN/m² based on location in Hämeenlinna, Finland. S=2.0 kN/m². The wind load W is shown in Figure 4. The transverse roof loads are directly applied to the rafters.

![Figure 4: Wind load in one middle frame (6.0 m frame spacing).](image)

### 4 Finite element models

#### 4.1 Frame components

Hughes-Liu beam element with user-defined cross section integration is used in the modeling of portal frame components, including columns, rafters and struts. Truss element is used for bracings in comparison cases. In total there are 39 integration points in the I-

#### 4.2 Modeling of fasteners

There are two types of fasteners used in this building structure: shot nails with 5.5 mm diameter for trapezoidal roof sheeting and self-drilling screws with 5.5 mm diameter for SPA230 sandwich wall panels. An equivalent Hughes-Liu beam element with a length of 10 mm is used to simulate the temperature-dependent spring-like in-plane shear properties of these fasteners. The major spring properties come from hole-elongation of trapezoidal sheet or inner sheet of sandwich panel. Lu et al [4] performed shear properties tests at elevated temperatures for shot nails and Wald et al [5] conducted shear tests for self-drilling screws with sandwich panels. These test data are used to calibrate the equivalent properties of the Hughes-Liu beam element. Ls-Dyna analyses are carried out to find out the equivalent material properties. MAT24 with bi-linear properties has three variables: elastic modulus, yield strength and secant modulus. For self-drilling screw connecting sandwich panel with steel section, another beam element is used to model the screw shank between inner steel face and external face of sandwich panel. A pinned connection is defined between these two beam elements for self-drilling screw.

Figures 6 and 7 show the spring-like force-displacement responses of the equivalent beam model for 5.5 mm diameter shot nails and self-drilling screws.
Temperature histories and boundary conditions

Assume that the frames D, E and F are exposed to ISO fire. In these exposed frames, columns, rafters and lower side of trapezoidal roof sheeting are exposed directly to fire. Inner sheets of sandwich wall panels are exposed to fire between those frames.

Two dimensional thermal analyses are carried out to obtain the temperature histories in the cross-section of columns, rafters and struts. These analyses are similar to the thermal analysis cases in [11]. The temperature histories for columns and rafters under ISO fire are illustrated in Figure 8. Temperature histories of struts and roof sheeting under ISO fire are illustrated in Figure 9. The wall bracing member in fire compartment utilizes the uniform temperature history in the cross-section, same as lower flange of strut in Figure 9a.

Fixed boundaries are defined for all column bases in case of fire.
5 Structural responses

5.1 Effect of sandwich wall panels in fire compartment

In sandwich wall panel with mineral wool (MW) core, the steel faces are adhered to the MW core using adhesive or glue. Usually these glues will melt at 200-300 °C, and therefore delamination of steel inner face will occur at elevated temperatures. The inner face will tangle at screw locations without load bearing capability. The external face will remain adhered with the MW core. However, the MW core is not connected with self-drilling screws. The external face is connected with screws with a distance of panel thickness from the column flange. This distance varies from 100 to 230 mm and it is very flexible in resisting the in-plane force.

It was observed from many fire tests that the sandwich panels will lose both the in-plane and out-of-plane load-bearing capacity. However, there is still a question on the possible positive role of the external face on the fire resistance of steel frames.

Two cases are analyzed to investigate the contribution of sandwich panel walls in the fire compartment. The purpose is to understand the difference between the case without panels (‘np’ in the Figures 11-15) in the fire compartment, and with simplified method to include the panel (‘wp’ in the figures). It should be noted that the simplified method uses two layers of composite shell for wall panels in fire (external steel face and MW core). It also assumes in the FE model that the both layers are connected with screw at inner face position.

5.1.1 Deformation response

Figure 10 illustrate the overall deformation of the building immediately after first runaway.

Figure 11 shows that the vertical displacements at apex of rafter E are close to each other for both cases until final runaway. The first runaway occurs at 26 minutes fire exposure and the corresponding temperature at lower flange of rafter is about 580 °C. The elastic modulus E of steel at this temperature is about 30% of that at normal temperature, and yield strength 24%. The final runaway of roof is 49 minutes for the case without sandwich panels in fire compartment and 53 minutes for the case with simplified panel. There is a secondary runaway at 46 minutes for the case with simplified panels (wp). This is due to the buckling of struts of bay EF at eaves. Similar behavior is observed for the horizontal displacement along x-direction for frames A and H at eaves and apex (Figures 12 and 13).

Figure 12 Horizontal displacement (x-direction) of rafter A at node A1 (Frame A-1), apex (Frame A-2) and A6 (Frame A-3).

Figure 13 Horizontal displacement (x-direction) of rafter H at node H1 (Frame H-1), apex (Frame H-2) and H6 (Frame H-3).

5.1.2 Axial forces

Similar response of axial forces in the rafter E is observed for both cases (Figure 14). Figure 15 shows the axial forces in the struts at bay EF at eaves and apex. It can be seen that the axial forces are close to each other until respective final collapse for the strut at apex, and larger difference of struts at eaves. The case with simplified sandwich panels in fire compartment stiffens the wall system and therefore a larger compressive force is developed in the eave.
struts due to the restrained thermal expansion. First runaway occurs at 26 minutes due to material softening at elevated temperatures and rapid increase in compressive forces at eave struts due to the catenary force in apex strut. Then it keeps almost constant for 235 kN until the second drop at 46 minutes due to buckling of eave struts. The maximum compressive force at eave struts for the case without panels is around 185 kN.

Further investigation shows that the eave struts at EF bay start to buckle at 26 minutes for the case with bracings, while 49 minutes for the case with steel claddings. The runaway mechanisms of roof are different for these two cases. The roof runaway of bracing case is due to steel softening at elevated temperature and the buckling of eave struts, and consequently frames lose out-of-plane stability. The first runaway of steel cladding case is due to the steel softening and second runaway is due to buckling of eave struts and weak roof diaphragm (especially frame H).

5.2 Stabilization of steel claddings

In order to compare the fire-resistant behavior of steel frames with steel claddings with the case of traditional roof and wall bracing system, the structural fire analyses are performed for two cases using traditional bracings or steel claddings only. These cases possess fixed column bases and unprotected frame members and are carried out for both snow load case (Load case 1) and wind + snow load case (Load case 2).

5.2.1 Load case 1: snow load

Figure 16 shows the comparison of vertical displacement of frame E at apex. It can be seen that up to 28 minutes iso fire exposure, the vertical displacement at apex of rafter start to deviate. For the case with bracings, the vertical displacement increases faster than the case with steel claddings. A complete runaway occurs at 43 minutes for bracing case and 49 minutes for steel cladding case.

Figures 17 and 18 illustrates the horizontal displacement along longitudinal direction of building (x-direction) of frame A and H. Displacements of three nodes for each frame are illustrated, at eaves and apex. It can be seen that the horizontal displacement at eaves of frame A and H are much smaller and stable until collapse at 49 minutes for the case with steel claddings (SC), and already become unstable at 26 minutes for the case with bracings. This indicates a stiffer presence of sandwich walls in unaffected compartments. The larger horizontal displacement at apexes for the case with steel claddings indicate a less stiff roof diaphragm in x-direction than wall diaphragm. However, the overall horizontal displacement at apexes for the case with steel claddings are smaller than the case with bracing system.

It can be concluded that the difference between the studied cases are relatively small. Taking the overestimation of panel stiffness for the simplified panel case, the actual structural response is very close to the case without inclusion of panels in fire compartment. Meanwhile, the case without panels is slightly in the safer side. Therefore, in the following analyses, the sandwich wall panels in the fire compartment is not included in the models.
Figure 18 Horizontal displacement (x-direction) of rafter H at node H1 (Frame H-1), apex (Frame H-2) and H6 (Frame H-3) – with bracing system (BR) or steel claddings (SC).

Figure 18 shows the axial forces of rafter E for the cases with steel claddings or bracings. It can be seen that the axial forces developed in rafter are close to each other before 34 minutes. The rafter is under compression due to thermal expansion and restraint from columns, in addition to bending moment. After 35 minutes fire exposure, the axial force of rafter E becomes tensile, due to large displacement and the corresponding catenary action.

Figure 19 illustrates the axial forces of struts at EF bay. It can be seen that at 25 minutes fire exposure, the strut at apex becomes tensile, and the case with bracings has higher tensile force than the case with steel claddings. For the struts at eave position, the development of axial forces for these two cases is very different. For the case with bracings, the compressive forces are developed almost linearly up to 9 minutes, mainly due to the thermal expansion and restraint from wall bracing system. It becomes constant, meaning a balance between thermal expansion, softened restraint and softening of strut itself. At 26 minutes, buckling occurs for both struts at EF bay, and the compressive force declines drastically.

Figure 20 Axial forces of struts between frame E and F: at axis 1 (Strut EF - 1), at apex (Strut EF - 2), and at axis 6 (Strut EF - 3) – with bracing system (BR) or steel claddings (SC).

### 5.2.2 Load case 2: wind + snow loads

Figure 21 illustrates the vertical displacement of rafter E at apex for the Load Case 2 (wind+snow load). It can be seen that the vertical displacements for the case with steel claddings and that with bracings are close to each other before 58 minutes. After that the case with steel claddings starts the runaway, and the case with bracings continues to develop relatively slowly until 78 minutes. First runaway occurs at 33 minutes.

Figure 21 Vertical displacement of frame E at apex – with bracing system (BR) or steel claddings (SC) for load case 2.

Figure 22 Horizontal displacement (x-direction) of rafter A at node A1 (Frame A-1), apex (Frame A-2) and A6 (Frame A-3) – with bracing system (BR) or steel claddings (SC) for load case 2.
5.3 Overall fire resistance

The critical deflection criteria is widely used in assessing the fire resistance tests of structural elements [7]. According to Law et al [8] and Lamont et al [9], the deflection criterion of L/20 originates from the standard fire test furnace and was chosen so that the furnace will not be damaged during testing. The failure criterion of L/30 was defined in European Recommendations for fire safety of steel structures in 1983 (ECCS 1983). European standards for testing (EN1363-1) and for classification (EN13501-2) specify the critical deflection of L/400d (where d is the height of cross-section) and limiting rate of L/9000d over one minute. The mid-span deflection criteria of L/20, L/400d and deflection rate L/9000d are used in this study. The corresponding times are summarized in Table 4. The time corresponding to actual collapse of entire building is also included.

From Table 4, it can be seen that deflection criterion of L/20 and deflection rate L/9000d give close fire resistance predictions – both corresponding to the first runaway due to material softening. This corresponds also to the exposed rafter temperature of 550-600°C. The fire resistances given by the deflection criterion of L/400d are closer to fire resistance by the start of final collapsing. For the case with steel cladding and wind+snow load, the 2nd runaway starts slowly at 59.7 minutes and the deflection rate is around 300 mm/minutes.

From this table, the deflection criterion of L/400d gives better fire resistance prediction of actual one-storey steel-framed building with restraints from cool compartments.
6 Conclusions

3D Finite Element analyses are carried out to investigate the fire resistance behavior of a one-storey industrial steel-framed building with steel claddings in fire. The equivalent orthotropic plates are used to simulate the trapezoidal sheet at elevated temperature. Composite shell element with soft core are used to model the sandwich panels in cool compartment. Equivalent beam elements are used to model the shot-nails and self-drilling screws, and the equivalent properties are derived based on the testing results from literature. Based on the comparative study, the presence of sandwich wall panels in the fire compartment can be neglected in the structural fire analysis of entire building.

The fire resistances corresponding to the deflection criterion of L²/400d is used to derive the conclusions in this section:

1) In most cases above, there are two or three runaways of vertical deflection of roof and horizontal deflection of frames. The first runaway is corresponding to the material softening of steel at temperatures around 550-600°C. The second runaway is corresponding to the buckling of eave struts due to the developed compression force, which is increased by catenary action of roof sheeting and apex strut.

2) The sandwich wall panels in the cool compartments provide horizontal stabilization in resisting the catenary actions in roof system at fire compartment.

3) Snow load cases (Load case 1) have shorter fire resistance than wind + snow case (Load case 2). Therefore, Snow load case is perhaps the most unfavorable case in fire for one-storey industrial steel building in Finland. More cases are needed in order to reach the conclusion.

4) In the case of fixed column bases and unprotected columns, the fire resistance of steel frames with steel claddings (no bracings) is 22% longer than with traditional bracings only. This indicates that the steel claddings systems provide better stabilization in fire.

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| Load case, fire protection and boundary of column bases | Apex deflection L/20 at rafter E (1540 mm) | Apex deflection L²/400d at rafter E (4730mm) | Apex deflection rate L/9000d at rafter E (210 mm/min) | Collapse of entire building (start of final runway) |
|------------------------------------------------------|------------------------------------------|---------------------------------------------|-------------------------------------------------|--------------------------------------------------|
| Bracing, snow load | 26.3 | 40.7 | 25.7 | 42.7 |
| Bracing, wind+snow | 34.3 | 80.3 | 33.3 | 78.0 |
| Cladding, snow load | 26.3 | 49.7 | 25.7 | 49.3 |
| Cladding, wind+snow | 34.3 | 65.3 | 33.3 | 59.7 |