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NUMERICAL MODELLING OF A SINGLE STOREY INDUSTRIAL BUILDING AT ELEVATED TEMPERATURE - COMPARISON BETWEEN THE 2D AND 3D ANALYSES

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Abstract: *Single storey industrial buildings with steel structure composed by portal frames disposed in parallel constitute a very frequent kind of construction. The computational simulation of steel structures under fire conditions is usually performed by the two-dimensional finite element analysis of the structure. In this way the analysis of the behaviour of single storey industrial buildings is performed either by the simulation in the plane of the steel portal frame or, simply the roof truss.*

However, the two-dimensional analysis takes into account neither the out-of-plane instabilities, nor the out-of-plane redistribution of efforts. These effects can play an important role in the structural performance of a building in a real fire situation.

In this paper a comparative study of two- and three-dimensional models of a portal frame industrial building under elevated temperatures is presented. The importance of the out-of-plane effects and the adequacy of the plane model to represent the real behaviour is discussed.

The computer program SAFIR[5] , developed at University of Liege for analysis of structures submitted to fire, was used in all the simulations. Amongst other features, the structural analysis performed by SAFIR considers: the effects of large displacements; the non-linear variation of material properties with the temperature; the effect of the thermal elongations; the evolution of the structural behaviour given as a function of time (an automatic time step can be used).

1 INTRODUCTION

The structural behaviour under fire conditions aims to minimise the risks to life and reduce the property losses. Due to the high thermal conductivity of the material and the slenderness of the sections, the performance of steel structures under fire demands a special attention.

Single storey industrial buildings used as factories or warehouses constitutes a very common type of steel construction. The risk of life in case of fire in an industrial building is small, because usually the evacuation does not impose much difficulty. The interest of research on this type of structure lies in special characteristics: high fire load, quick fire spread, a relatively elevated risk of fire occurrence and heavy monetary losses.

The structure of this kind of buildings is usually composed by several portal frames, disposed in parallel. The analysis of the structural behaviour under fire conditions of a building by using computational simulation is referred in the Eurocode[4] and the Brazilian standard[2] as *advanced calculation methods*, and is commonly performed by a two-dimensional finite element modelling of the structure. Thus, in the case of single storey portal frame buildings, the analysis is performed either by the simulation in the plane of the steel portal frame or, simply the roof truss.

At elevated temperature considerable out-of-plane effects can take place, as a redistribution of efforts due to the material degradation and due to thermal expansion, and lateral instability of the members. Although these efforts can play an important role in the structural performance they are not taken into account with the two-dimensional simulation. The importance of these effects and the adequacy of the plane model to represent the real behaviour is discussed here by a comparative study of two- and three-dimensional models of a portal frame industrial building under elevated temperatures.

2 ANALYSED BUILDING

The building chosen for the studies is shown in Figure 1. It is a building with 20 m of width and 48 m of length. The roof is covered by corrugated sheet metal, which forms also the lateral walls.

The portal frame, used in the building and shown in Figure 2, has columns with welded I-section, and a roof truss composed by cold-formed sections. The connections between the roof truss and the columns were considered as hinged. The column-base was considered as clamped. In the columns lateral purlins were fixed, spaced 1,5 m; at the top chords of the roof truss the purlins were fixed at a distance of 2.04 m, as indicated in Figure 1 and Figure 2. All the purlins were considered as simply supported over each portal frame. As in the roof truss, for the purlins it was adopted a cold-formed section. A more detailed specification of the parts of the building is given in Table 1.

Due to the large dimensions, fire in an industrial buildings grows in such a way that only one part of the building is taken by fire at each time. For the case studied here it was considered the case of a fire in the central part of the building, as indicated in Figure 1, affecting the whole portal frame in this position, and the purlins linking this portal frame to the adjacent frames. A uniform heating, without thermal gradients was considered.

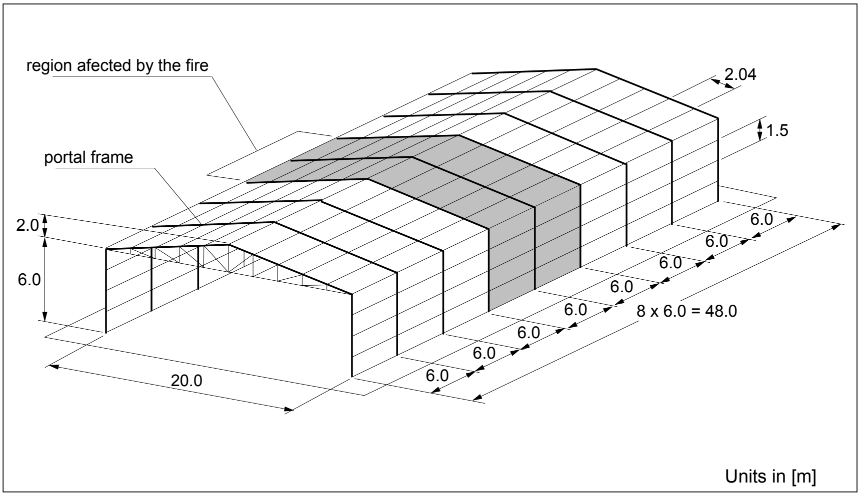

Figure 1 - Steel structure of the studied industrial building.

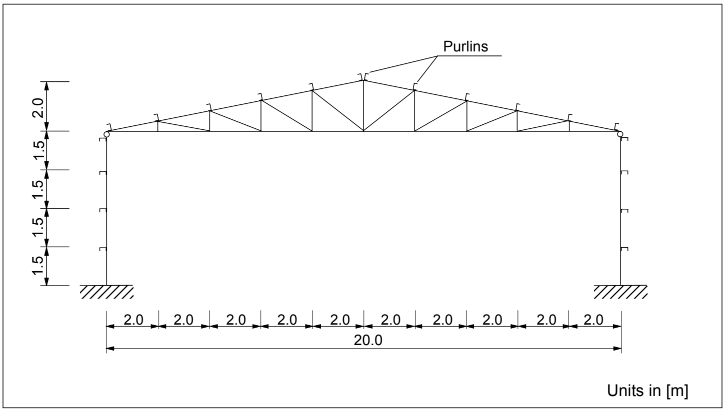

Figure 2 - Scheme of the portal frame used in the building.

Table 1 - Specification of building parts.

Structural member	Load due to self-weight	Specification	Description
Columns	380.6 [N/m]	CS200x39	Welded I-section, with b=200mm, h=200mm, tw=6.3mm, tf=9.5mm
Top and bottom chords of the roof truss	104 [N/m]	HAT 150x100x25#3.00	Hat section, with b=100mm, h=150mm, e=3.0mm, length of the stiffener=25mm. The top chord opening downwards, and the bottom chord opening upwards.
Verticals and diagonals of the roof truss	81.8 [N/m]	2 U75x40#3.75	Two channel sections, with the openings faced to each other, with b=40mm, h=40mm, e=3.75mm
Purlins	99.9 [N/m]	U150x60x20#4.76	Lipped channel section, with b=60mm, h=150mm, e=4.76mm, width of the stiffener=20mm. On the sides of the portal frame the purlins have the openings faced to down, on the left top chord to left and on the right top chord to right.
Bracing bars	9.7 [N/m]	ø1/2"	Bar with diameter of 12.7mm
Tile	50 [N/m2]	CTL 25	Trapezoidal corrugated metal sheet
Used symbology: b = section width, h = section depth, tw = web thickness, tf = flange thickness, e = thickness of the cold formed metal sheet			

3 2D AND 3D MODELLING OF THE PORTAL FRAME ISOLATEDLY

As a starting point of comparison between the two-dimensional and three-dimensional analyses it was considered the portal frame isolatedly, modelled with 2D and 3D beam elements.

For the loads, besides the self-weight of the members, it was imposed a point load of 1.2 *kN*, on the bottom chord, applied in the positions indicated in Figure 3. This is an estimated load corresponding to lighting fixtures, electrical and hydraulic fittings. The wind load was not considered because it induces an uplift of the roof truss, having a beneficial effect.

Steel with a modulus of elasticity $E = 210\,000\,MPa$ and a yield strength $f_y = 355\,MPa$ was adopted. The cross sections adopted for the portal frame receive the classification as Class 3 whereas the purlins sections are classified as Class 1[1,3]. While the Class 1 sections can form plastic hinges in bending, in the Class 3 sections local buckling can occur, preventing the development of its full plastic moment resistance.

The beam element used in SAFIR adopts the Bernoulli-Euler hypothesis, (i.e., the effects of the transverse shear deformations are neglected). This finite element does not take into account the occurrence of local buckling in the member; thus, in principle this element can be used to simulate only members with sections of Class 1 and Class 2 in the Eurocode[1,3], which are not subject to the effects of local buckling.

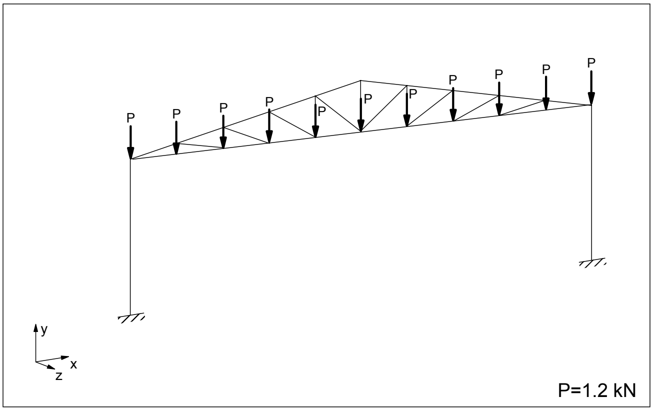

Figure 3 – Nodal load applied at the bottom chord.

For advanced computer analysis the Eurocode[2,4] recommends a linear-elliptic-plastic model for the stress-strain curves (Figure 4). As at elevated temperatures the yielding point is not well defined this stress-strain model is based in a strain limit, which generally corresponds to a value of 2 %. An alternative approach used to model members with Class 3 sections, is to consider a lower strain limit. This approach was adopted here, to model the members of the portal frame, with stress-strain curves based in a strain limit of 0.2 %. Numerically speaking, this restriction was obtained by using the reduction of yield strength as recommended for deformation criteria in Eurocode 3. In the purlins modelled in the next section a strain limit of 2 % was used, since they have Class 1 section.

For both the 2D and 3D analysis, the frame shown in Figure 2 was modelled using beam finite elements; 8 elements in each column, 10 elements in each top chord, 20 elements in the bottom chord. For the verticals and diagonals of the roof truss it was used 2 elements for each bar.

In all the cases load was applied firstly and then the structure was uniformly heated, without thermal gradients. The temperature was raised, until the stiffness matrix becomes non-positive and SAFIR stops the calculations. The last temperature for which the equilibrium was reached was taken as the critical temperature of the structure.

For the 2D model the critical temperature was 675°C. The failure was caused by the plastification at the lateral extremities of the roof truss, where occurs a concentration of stresses. The deformed shape of the structure at the critical temperature, plotted over its initial shape, is shown in Figure 5.

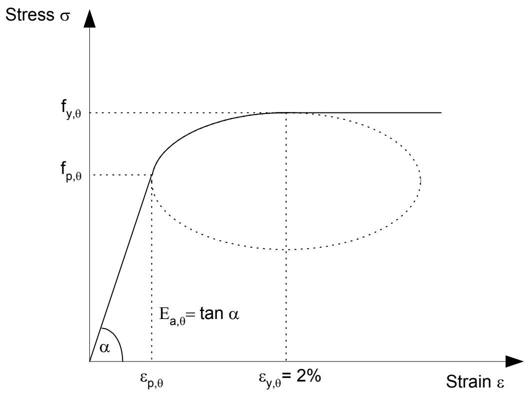

Figure 4 - Stress-strain relationship for steel at elevated temperature.

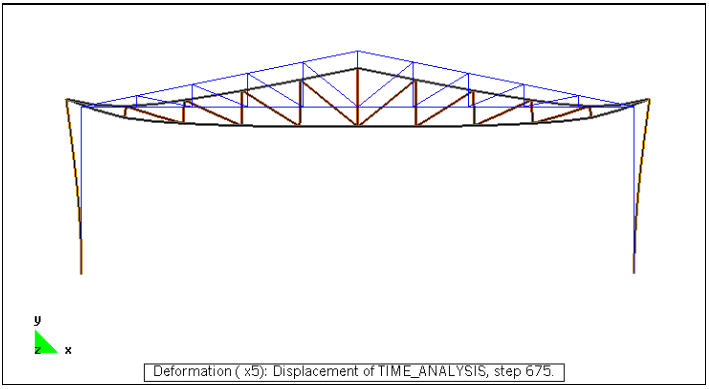

Figure 5 - Deflected shape of the 2D model in the moment of failure, with amplification factor of 5.

In order to have an indication of the importance of the out-of-plane restriction in the critical temperature attained by the portal frame, for the 3D analysis three different levels of out-of-plane restrictions were considered. According to the first case the out-of-plane translation was prevented in all the positions (from P1 to P11) of the purlins in the roof truss, as shown in Figure 6. In second case, the out-of- plane translations of the roof truss were blocked in the positions indicated as P1, P3, P6, P9 and P11. As a last case, it was considered the restrictions in the ridging and in the extremities of the roof truss, corresponding to the positions P1, P6 and P11.

In the first case it was obtained a critical temperature of 632°C, slightly lower than the temperature obtained in the 2D analysis. The failure was caused by an out-of-plane buckling in the top chords, at the spans of the extremities, as shown in Figure 7. In the second case an out-of-plane buckling started also in these spans, but out-of-plane displacements are presented also in another parts of the roof truss (Figure 8) and the critical temperature was decreased to 626°C. In the third case, where the translation was blocked in three parts of the roof truss, out-of-plane displacements are presented in lower temperatures and the critical temperature was reduced to 514°C (Figure 9).

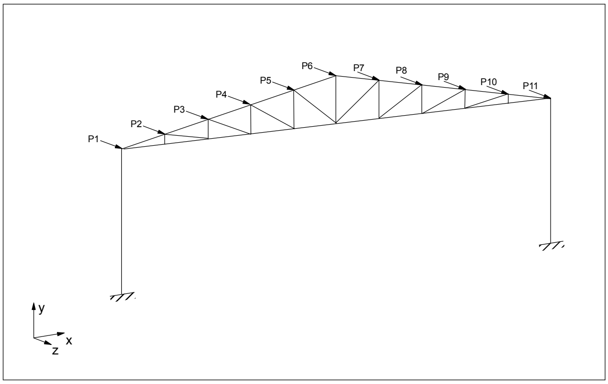

Figure 6 - Restrictions imposed in the first case of the 3D model.
In the positions P1 to P11 the out-of-plane translation was restrained.

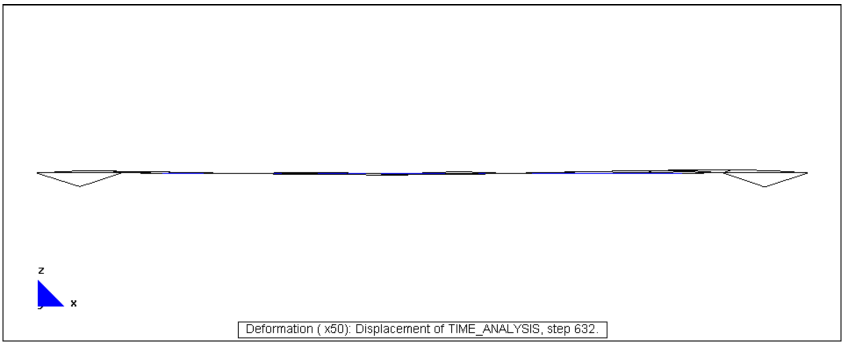

Figure 7 - Deflected shape for the first case modelled with 3D elements
in the moment of failure, with amplification factor of 20.

Concerning the out-of-plane buckling of members, an approach commonly used when performing a 2D analysis is to verify the possibility of its occurrence by simple calculation methods and, if needed, prevent it. However this procedure is rather unrealistic, as in the simple calculation methods the member are considered individually, without taking into account the interaction between them, and the efforts on the member are considered as constant, without taking into account the evolution of these efforts with the increase of temperatures.

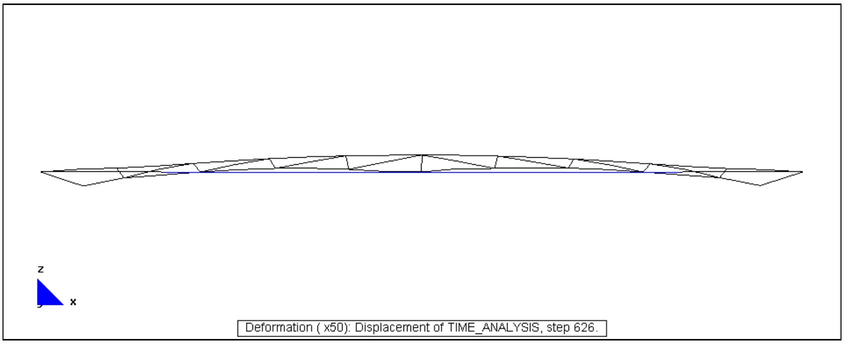

Figure 8 - Deflected shape for the second case modelled with 3D elements
in the moment of failure, with amplification factor of 20

The analysis shows anyway that providing lateral supports and thus reducing buckling length for instability is a very efficient way to prevent out of plane displacements and can increase the critical temperature of the frame.

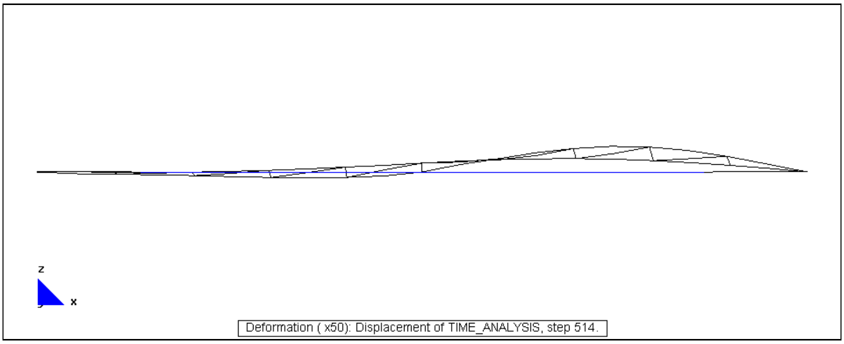

Figure 9 - Deflected shape for the third case modelled with 3D elements
in the moment of failure, with amplification factor of 20
(the restriction was imposed in the ridging and in the roof truss corners).

Concerning this aspect, it can be said that the 2D analysis of the portal frame presents a non conservative response, as the frame is modelled as totally restrained in the plane. In a building the out-of-plane stability is provided by the bracing system, which includes the action of the purlins. In the next section, the purlins were included in the model, with the aim of simulating in a more realistic way the out-of-plane restriction imposed to the portal frame.

4 MODELLING OF THE PORTAL FRAME WITH THE PURLINS

In order to see the interaction between the portal frame and the purlins during the fire evolution, it was considered the portal frame in the middle of the building, and the section of purlins linking it to the adjacent portal frames.

For the portal frame the same discretization used before was adopted. The purlins considered were modelled with 12 beam finite elements each. As it can be seen in the scheme presented in the Figure 10, in the purlins on the roof it was considered the existence of bracing bars that divide their buckling length in the weakest axis by two. It was considered for these bracing bars a circular section, with 1.27 cm of diameter. As the surrounding structure can impose a restriction to thermal expansion of the bracing bars which can cause their buckling, they were modelled using two beam finite elements for each span, instead of a single truss element. The ends of the bracing bars were considered as tied to each other in the ridging, in a way that their translations were consolidated.

As it can be noted in Figure 2, there are two purlins in the ridging and also in the eaves of the roof that are quite close from each other. In the finite element model, these purlins were modelled with different finite elements, but located initially at the same coordinates.

The connections between the purlins and the portal frame were considered in a way that the 3 translations and the rotation about z were consolidated between the coincident nodes of the purlins and the portal frame.

The part of the structure not modelled for this analysis will still impose some restriction on the longitudinal translations in the extremities of the modelled part of the purlins. This restriction will be larger if the action of the bracing system on the structure is considered. This effect could be modelled properly by using spring elements at the purlins ends, but the lack of information on the stiffness imposed to the structure to these translations make its use unsuitable.

Thus, the longitudinal translation of the purlins was considered as totally restrained as the remaining option closer to reality. This way at the purlins ends the translations and the rotations about the z axis were restrained, as shown in Figure 10.

As before, the structure was loaded at ambient temperature and then subjected to thermal loading increments. At 48°C the instability caused by the buckling of the purlins due to thermal expansion, resulted in a negative stiffness of the stiffness matrix of the structure. This negative stiffness made SAFIR to stop the calculations. The deflected shape of the structure is shown in Figure 11.

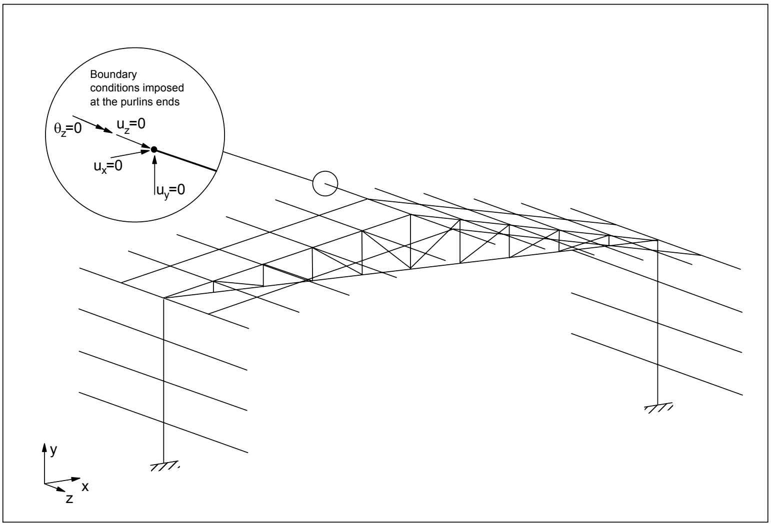

Figure 10 - Model with the portal frame and
the part of the purlins between the adjacent portal frames

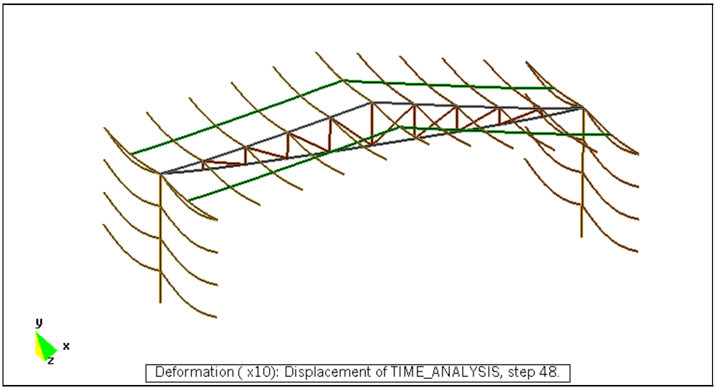

Figure 11 - Deflected shape for the model with purlins
in the moment of failure, with amplification factor of 10.

The observation of real fires shows that the purlins are very susceptible to the fire effects[6]. As the purlins sections are usually relatively slender, in real fires they tend to heat quickly; as the portal frame restrain their thermal expansion, they fail in the early stages of the fire.

However, in spite that the buckling of the purlins or even the bracing bars endangers momentarily the stability of the portal frame, this does not determine the failure of the whole structure. After its failure, the purlins tend to deform in catenary shape[7]. It is hoped that this catenary effect can contribute in a later stage to the lateral stability of the portal frame, as shown in Figure 9.

The simulation by SAFIR of the structural behaviour after the purlins buckling would require the implementation of special numerical strategies, not present in the current version. The temperature and the load imposed to the structure must be kept constants and a new equilibrium position must be found. Afterwards a new increment of temperature can be imposed, continuing the analysis. A new research project is currently starting, sponsored by the ECSC (European Coal and Steel Community), to implement such a procedure, the necessity of which has been demonstrated by this study.

An alternative procedure to overcome the matrix ill-conditioning due to the buckling of parts of the structure is to simulate the structural behaviour until the buckling occurs. Afterwards it could be performed the analysis of a new case, with the structure without the buckled members. The total displacement path of the initial structure is considered as the displacement path obtained in the first analysis, until the buckling. After this position, the continuation of the displacement path is the one obtained with the analysis of the structure without the buckled members. This strategy is not interesting in this example because it would lead to the situation studied before (see section 3).

The modelling of the whole building structure would probably not solve the problem, as the structure adjacent to the purlins with thermal elevation imposes a restriction that would lead also to their buckling.

5 CONCLUSIONS

The simulation of the structural behaviour of single storey industrial buildings by the two-dimensional modelling is unrealistic, as it does not take into account the lateral instability of the members of the portal frame because it is modelled as totally restrained in the plane. At ambient temperature the out-of-plane instability of the portal frame can be easily prevented by the purlins. However, in fire situations, the failure of the purlins caused by thermal expansion can compromise this stability.

The modelling of the role played by the purlins in fire conditions and their post-failure interaction with the portal frames requires the implementation of special numerical strategies to overcome the ill-conditioning of the stiffness matrix caused by the buckling of purlins due to thermal effects. This modification could allow simulating the behaviour of a single storey industrial building under fire in a more realistic way.

6 ACKNOWLEDGEMENTS

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