# The Design of Pitched-Roof Steel Portal Frames Against Fire

Y. Song; Z. Huang; I. W. Burgess
*Department of Civil and Structural Engineering*
*University of Sheffield, S3 7XL, UK*
*y.y.song@sheffield.ac.uk; z.huang@sheffield.ac.uk; ian.burgess@sheffield.ac.uk*

R. J. Plank
*School of Architectural Studies*
*University of Sheffield, Sheffield, S10 2TN, UK*
*r.j.plank@sheffield.ac.uk*

**Abstract**

In this paper the failure mechanisms in fire of single-storey haunched portal frames, subject to different support conditions at their column bases, are investigated using a simplified dynamic solution procedure which has been developed. These studies are used to evaluate a current design method.

## 1. Introduction

For steel portal frames in fire, it is imperative that boundary walls stay close to vertical, so that fire is not allowed to spread to adjacent properties. The current UK fire design guide [1] requires that the whole frame has to be protected as one element in fire, or else leaves the rafter unprotected. The column bases and foundations then have to be designed to resist the forces and moments generated by the collapse of rafter to ensure the lateral stability of boundary walls. A simple mathematical model based on the force equilibrium and shape of the collapsing rafter has been developed to estimate the overturning moment applied to the foundation. It is assumed in this guide that the pitched-roof portal frame always deforms symmetrically in fire under the vertical distributed load, and that a fire hinge is generated at each end of the rafters, as shown in Fig. 1. To simplify the design method, the equations are based on the assumption that the rafters collapse due to these fire hinges and that the inclination of the columns is limited to 1°, with a rafter elongation of 2%. The fire hinge moments are all equal to 0.065 of the plastic moment resistance of the rafter and the haunch length is 10% of the span. These arbitrary assumptions regarding behaviour of the frame in fire can lead to very uneconomical foundation design and baseplate detailing.

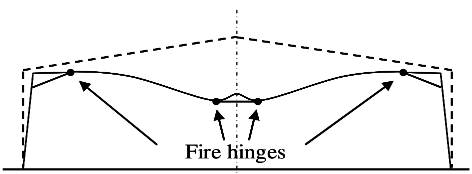


**Figure 1. Positions of fire hinges in a pitched portal frame**

A fundamental aspect of the collapse of portal frame rafters is that they often lose stability in a “snap-through” mechanism, which is capable of re-stabilizing at high deflection, when the roof has inverted but the columns remain close to vertical. By most static analysis methods only the initial loss of stability is identified. In a model-scale test [2] on a single-storey single-bay portal frame, the static modelling could not continue beyond the point at which snapthough occurred. The maximum vertical apex displacement from the numerical analysis was about 17% of the roof rise, whereas it almost reached eaves-level in the actual fire test, so it was impossible to identify either the final equilibrium state or the intermediate column movements.

In 2001, Wong [2] developed a simplified method to estimate the critical temperatures of portal frames in fire. This method calculates a strength reduction factor based on the common plastic hinge mechanism of portal frames under fire conditions. From this reduction factor the corresponding critical temperature can be deduced from *BS5950: Part 8* [3].

Recently, a simplified dynamic solution procedure has been developed in which both damping and inertial effects have been added to the applied forces and an effective stiffness matrix, which always has positive diagonal values, is generated [4]. The procedure has been incorporated into the finite element software *Vulcan*. Hence the instabilities encountered in previous static analyses, and the snap-through behaviour of portal frames at elevated temperatures, can be properly investigated.

## 2. Dynamic Model

The main objective of this dynamic analysis is to trace the global structural behaviour after a transient loss of stability, or partial failure, occurs. The vibration introduced by the dynamic behaviour can be damped out in a very short time period by super-critical damping, which can be determined from the natural frequency of the structure (see Fig. 2). The exact nature of the dynamic motion during period B is not very important for this study. The limit point at the end of period A and the behaviour beyond period B, which are two statically stable regions, are the main considerations in this research. It is important that the dynamic system can cross the region of transient motion during the analysis.

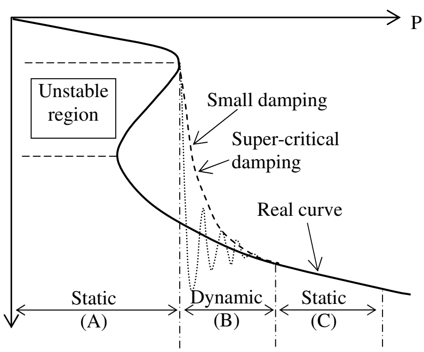


**Figure 2. Effect of damping in snap-through process**

The true static snap-through behaviour should follow the solid line in Fig.2 when nonlinearity is considered. However, in load-controlled numerical analysis this behaviour cannot be traced beyond the initial limit-point because the equilibrium becomes unstable. Instead of tracing the static deformation curve, the dynamic motion can be used to allow the analysis to find the second stable state, as shown in Fig. 2. Damping is used to ensure that the analysis can go back to a static one in period C when the motion is sufficiently slow to assume that the structure has reached a new stable position.

The implicit integration scheme for nonlinear dynamic problems [5] is adopted in this research due to its significant advantage in numerical computing efficiency. Because the behaviour in Stages A and C (see Fig.2) is static, it is not necessary to carry out a full dynamic solution procedure, which needs more computing time but shows the same static results. In this model, if singular values on the diagonal of the stiffness matrix are detected the dynamic analysis is activated. A lumped-mass matrix is assumed and the damping is both proportional to the mass and related to the lowest natural frequency of the structure. To ensure the accuracy of the dynamic analysis, the time-step size is limited to one twentieth of the maximum period.

To damp out the unnecessary inertial effect introduced by the damping energy, a step-loading schedule is used in this model.

## 3. Numerical Modelling of the Pitched Portal Frame

To investigate the failure mechanism of the single-span portal frame, a two-dimensional simplified pitched-portal frame designed by plastic theory at ambient temperature according to a well known design manual [6], which is shown in Fig. 3, was used in the numerical tests. Three-noded beam elements in *Vulcan* were used to set up this model. It was assumed that on heating the whole frame using the ISO834 Fire the temperatures of the roof are calculated by the simple *Eurocode 3 Part 1.2* method [7], and the temperatures of columns remain at 20°C because portal frame fires usually vent through removed roof cladding. An imperfection, which was assumed as a very small horizontal force on the left eave, was built into the model.

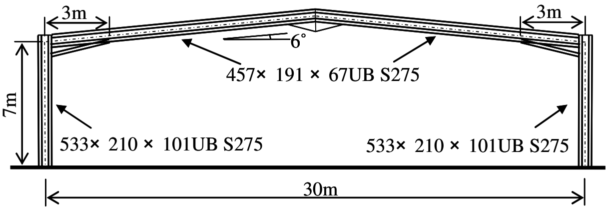


**Figure 3. Layout of the pitched portal frame**

Haunches of prescribed properties are assumed in the design guide [1], so the effect of the haunches in fire is considered in this model. The haunch was modelled by beam elements in three ways. In model H1, the haunch was modelled by three beam elements with the same cross-section, equivalent to the mid-haunch section. To represent the taper of the haunch section, the element near the eave was replaced by a bigger section and a relatively slender element was adopted for the inner end of the haunch in model H2. To emphasize the effect of the haunch, in model H3 the haunch was represented by constant beam sections equal to the maximum section of the haunch.

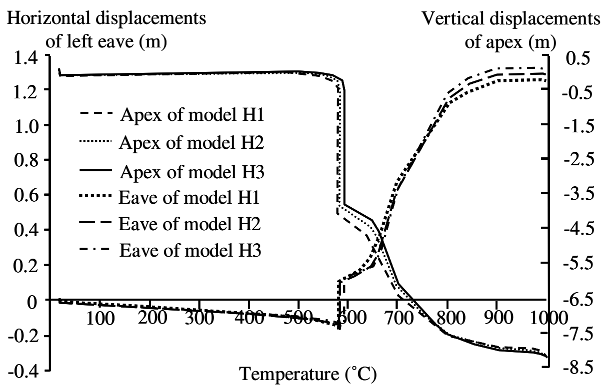


**Figure 4. Comparison of the behaviour of the frame with different haunch models.**

As shown in Fig. 4, the failure modes of three models are very similar. The temperature at the deflection jump in model H2 approaches that of H1, because the snap-through is initiated by the hinges generated at the end of the haunch, and the sections at the end the haunch affect the moment distribution on the structure. Due to the similar moment capacities at the eaves in models H2 and H3, these behave very similarly after the snap-through. All of the three models achieved the same failure mode when the apex descends to the foundation level. In the following numerical tests the model H2, which has a variable haunch section, is adopted.

The bases of the portal frame are assumed as perfectly-pinned connections in design at ambient temperature. However, in reality the column bases are neither pinned nor rigid connections. They are actually semi-rigid connections with different moment capacities which are largely dependent on the capacity of the bolts in tension. Hence, in this paper dummy members an with equivalent moment capacity were introduced into the model to provide partial fixity to the bases. These were modelled by a beam with one end rigidly connected to the column base perpendicular to the column, with restraint to movement but free to rotate, and the other end pin-supported by the foundation. The size of the dummy members was determined by the length of the column and the second moment of the column area as recommended in Section 5.5.1 of [6].

Because the load ratio was very low, only the self weights of the frame and the purlins were considered under fire conditions, the analyses of both cases finished beyond 1100°C and only the collapse of the roof frame was observed. The deformation of the roof was much more significant than the movements of the column tops. The collapse of the roof frames, as shown in Figs. 5 and 6, for frames with nominally pinned and nominally fixed bases, at high temperatures, were quite different.

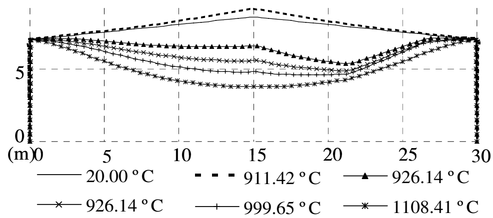


**Figure 5. Progress of collapse of the portal frame with nominally rigid bases**

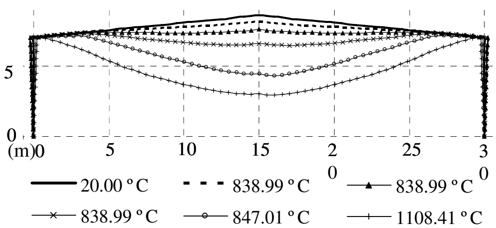


**Figure 6. Progress of collapse of the portal frame with nominally pinned bases**

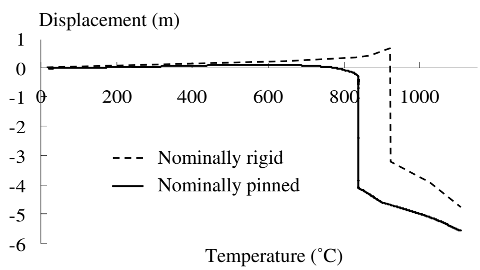


**Figure 7. Comparison of vertical deflections of the apex**

For the portal frame with nominally rigid bases, the rafters expanded symmetrically as the roof was uniformly heated, until the apex plastically yielded at around 880°C. The hogging moments then continued to increase on the rafters until the slight asymmetry caused by the imperfection led peak moments to develop at the middle section of the right-hand rafter and in the region near the end of the right haunch. When the temperature of the roof frame reached 926°C, the right rafter began to lose stability and collapse in a three-hinge mechanism with fire hinges at the apex, the end of the haunch and the central section of the rafter. As a result of the symmetrically distributed load on the roof, the moment on the left half of the frame then increased, which allowed the haunch sections near the eaves to plasticize gradually. Plasticizing of the sections near the eaves provided more flexibility to the rafters, and therefore the displacement of the apex (see Fig. 7) continued to drop until the roof transformed to an almost symmetrical shape below the eaves level, and pulled the column inward. Although the elongation of the heated steel and the vertical loading tended to make the apex deform downward, the restraint from the cold columns made the descending of the apex follow the slope of the dashed curve in Fig. 7.

Unlike the nominally rigid bases, the nominally pinned bases allowed more horizontal movement of the tops of columns. This avoided failure of the rafters during the raising of the roof. The rafter sections near the ends of the haunches formed the first plastic hinges which initiated the snap-through of the steel roof at around 840°C. The plastic region around the apex developed when its deflection was over 2m. The frame re-stabilized at the position where the apex was about 2.5m lower than the eaves and the column tops had moved inward by about 0.15m. Due to the degradation of the steel and the vertical loading, another two plastic hinges formed at the eaves. This transferred the restraint of the rafters from the haunches to the columns, which changed the global stiffness of the frame, so a kink can be seen on the displacement-temperature curve.

## 4. Comparison with the current Design Method

The key concern of the current design regulations for portal frames in fire is that the columns should keep upright to provide enough stability to the masonry wall to stop the fire spreading to nearby buildings. It is assumed that the masonry wall will collapse when the slant of the column is more than 1°. This means that, for a column with length 7m, if the columns almost keep their original shape, the failure criterion of this frame is that the horizontal movement of the column top should not be more than 122mm, as marked in Figs. 8 and 9.

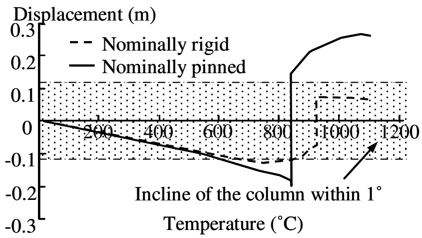


**Figure 8. Horizontal displacement of the left eaves**

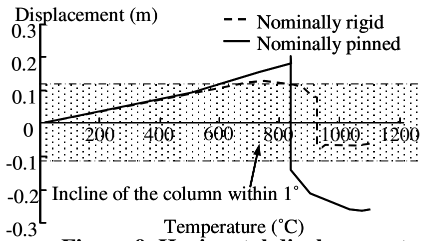


**Figure 9. Horizontal displacement of the left eaves**

The calculation in the current design guide [1] is based on the failure mode shown Fig. 1 and a simultaneous column inclination of 1°. However, according to the numerical analysis, no matter whether the frame has the nominally rigid bases or the nominally pinned bases, the frame can not achieve the failure mechanism shown in Fig.1 before the movement of the eave is over 122mm. Although the nominally rigid bases provide more restraint to the frame and almost keep the movement of the column within the safe region, the peak movement actually exceeds the limit marginally, but the failure mechanism is still far from that assumed in Fig.1.

The peak overturning moment (OTM) found at the nominally pinned base is only about one third of the maximum OTM developed on the nominally rigid base, which is about 480kNm. It is evident that the OTM at the column base has a significant effect on the stability of the columns. However, the design value of OTM to prevent excessive lean of the columns, which is deduced from the simple design method, is only 128kNm. It seems that, if the column bases of this frame are designed using the OTM calculated from the simple design method, the slant of the column should be more than 1°.

The strength reduction factor calculated by Wong's method is 0.0899, and the corresponding critical temperature interpolated from *BS5950: Part 8 Table 1* is 826°C for a strain level of

1.5% and 835°C for a strain level of 2%. This is very close to the critical temperature for the snap-through from the *Vulcan* analysis with nominally pinned bases.

## 5. Conclusion and Future Work

In this study, the whole collapse progress of the single-span portal frame under fire conditions has been modelled successfully using the new dynamic solver. The failure mechanism of a typical portal frame has been investigated. The results indicate that the current design method may not always provide conservative results for design. It has also been shown that the failure mechanism of the portal frame is a rotational failure of part of the roof section.

It is worth noting that the semi-rigid base connections were modelled using dummy elements whose length was three quarters of the height of the columns in these models [5]. These offer a relatively wide rotational range to the column base before plastic yield of the dummy element. In fact real base connections hardly work like this. The lean of the column always makes the base plate rotate first, and the moment resistance of the base is fairly small until yield of the holding-down bolts occurs, which gives the connection a certain amount of flexibility and moment resistance. Another problem is that several factors, such as the loading ratios and the column stiffnesses, may influence the behaviour of the portal frames in fire. This can not be judged from a single particular case.

A series of parametric studies will be carried out to investigate the failure mechanisms of portal frames in fire. The model of the column bases will be modified to make sure that it behaves similarly to the real base connections. Relying on the further understanding generated on the behaviour of portal frames in fire, a new simplified method for practical design will be developed.

## 6. References


[1] Newman, G. M., (1990), *Fire and Steel Construction: The Behaviour of Steel Portal Frames in Boundary Conditions (2nd Edition)*, The Steel Construction Institute, Berkshire, UK

[2] Wong, S. Y., (2001), *The structural response of industrial portal frame structures in fire*, PhD Thesis, University of Sheffield

[3] British Standards Institution, (1990), *BS5950: Structural Use of Steelwork in Building, Part8: Code of Practice for Fire Resistant Design*, British Standards Institution

[4] Song, Y., Huang, Z., Burgess, I.W. and Plank, R.J., (2007), *The behaviour of single-storey industrial steel frames in fire*, Acc. ICSCS 2007, Manchester

[5] Bathe, K. J., (1996), *Finite Element Procedures*, New Jersey: Prentice-Hall

[6] Salter, P. R., Malik, A. S. & King, C. M., (2004) *Design of single-span steel portal frames to BS 5950-1:2000, SCI Publication P252*, Ascot: The Steel Construction Institute, Berkshire, UK

[7] CEN, (2005), Eurocode 3: Design of Steel Structures – Part 1.2: General Rules – Structural Fire Design, European Committee for Standardization.