

THE BEHAVIOUR OF SINGLE-STOREY INDUSTRIAL STEEL FRAMES IN FIRE

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ABSTRACT: A new dynamic model and a quasi-static solution procedure has been developed and incorporated into the finite element software Vulcan [1-3]. This new numerical model is tested in this paper on benchmark problems which relate to the behaviour of steel portal frames under fire conditions. The capabilities and the accuracy of this numerical model have been validated against other advanced finite element software (ABAQUS [4], ANSYS [5] and SAFIR [6]). A case study on the behaviour of steel portal frames under fire conditions has been carried out using this new tool. The effects of different rotational stiffnesses at the column bases are compared for two different heating profiles. A second failure mechanism, based on large deformation rather than the initial configuration of the steel portal frames, is observed after an initial instability of the roof frame has re-stabilised in an inverted configuration at high temperatures.

Keywords: Dynamic analysis, temporary failure, snap-through, steel portal frames

1. INTRODUCTION

The pitched-roof portal frame is a typical single-storey steel structure widely used for industrial applications in the United Kingdom and elsewhere. For Ultimate Limit State design at ambient temperature, such frames are usually designed assuming that column base connections act as frictionless hinges [7]. In fire conditions it is imperative that building walls adjacent to the boundary of a site stay close to vertical and are not liable to collapse, so that fire is not allowed to spread to adjacent property [8]. A current UK fire design guide [9] allows the rafters, which are normally unprotected against fire, to collapse. To ensure the lateral stability of boundary walls, column base connections and foundations have to be designed to resist the forces and moments generated by this collapse of the rafters. However, the method makes a number of apparently arbitrary assumptions, and does not attempt to model the true behaviour of the frame during fire. Therefore the method can lead to very uneconomical, and even potentially unsafe, design of portal frames [10].

A fundamental aspect of the collapse of portal frame rafters under fire conditions is that they often lose stability in a “snap-through” mechanism. This is capable of re-stabilising at high deflections, when the roof has inverted but the columns remain close to vertical. By most static analysis methods only the initial loss of stability is identifiable. In the modelling of a small-scale test [11] on a single-storey single-bay portal frame, all the static analysis stopped at the point at which the snap-through occurred. The maximum vertical displacement of the apex in these numerical analyses was about 17% of the roof rise, while the deflection of the apex almost reached eaves level in the actual fire test. It was therefore impossible to identify the final Equilibrium state or the intermediate column movements.

Schematically, the true snap-through behaviour should follow the solid curve in Figure 1 when nonlinearity is considered. However, in load-controlled numerical analysis this behaviour can not be traced beyond the initial limit-point because the Equilibrium loses stability and the solution becomes divergent. Instead of tracing the static Equilibrium deformation curve, the dynamic

motion of the frame can be used to allow the analysis to find the second stable state C, if it exists, as shown by the dashed curve in Figure 1. Damping is used to ensure that the analysis can return to a stable Equilibrium curve, by reducing the amplitude of oscillation about this stable path until it is sufficiently small to justify assuming that the structure has regained stability. The main objective of using a dynamic model in this analysis is to allow global structural behaviour to be tracked after a transient loss of stability, or a partial failure, occurs.

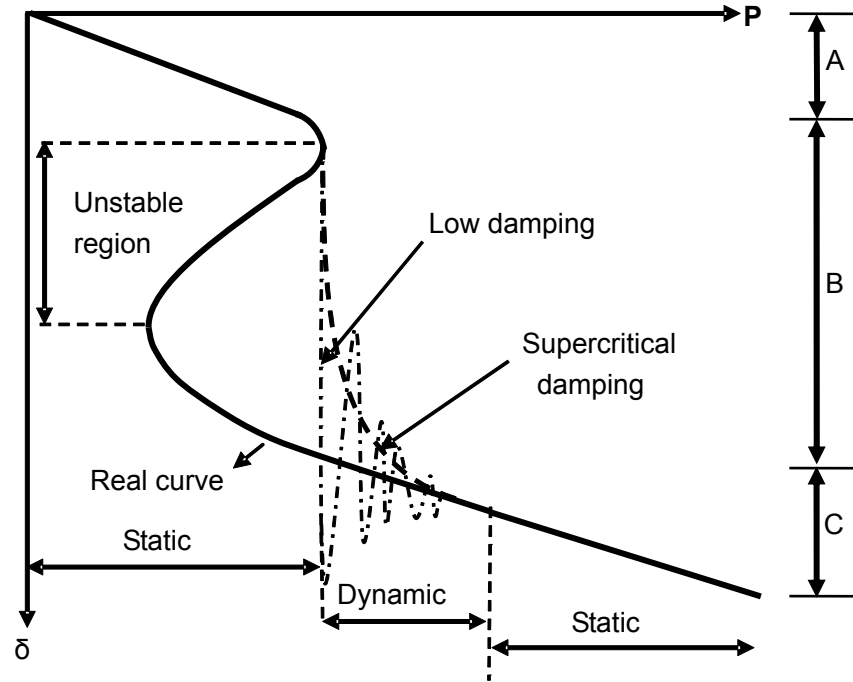


Figure 1. Effect of Damping in Snap-through Process

In this research a simplified dynamic solution procedure [12] has been developed, in which both damping and inertial effects have been added to the static stiffness formulation, and an effective stiffness matrix, which always has positive diagonal values, is generated. The procedure can deal with the instabilities which cause singularity in previous static analyses, and the re-stabilisation after temporary snap-through instability of portal frames at elevated temperatures can be investigated properly. Vibration introduced by the dynamic behaviour can be damped out rapidly by super-critical damping, of which the value can be determined from the lowest natural frequency of the structure (see Figure 1). 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 are the main considerations in this research, as statically stable regions. It is the ability of the dynamic system to mimic the real structure in moving across the region of temporary instability without encountering numerical singularity that is important.

2. DYNAMIC MODEL

In the static model, the internal forces balance the external forces to achieve Equilibrium, so the governing Equilibrium Equation is:

$$\mathbf{F}^{Int}(t) = \mathbf{F}^{Ext}(t) \quad (1)$$

where, $\mathbf{F}^{Int}(t)$ is the internal nodal force vector, and $\mathbf{F}^{Ext}(t)$ is the external nodal force vector. This nonlinear Equation is solved iteratively using the Newton-Raphson solution method. The iterations are based on Eqns. (2) and (3):

$$\bar{\mathbf{g}}_{t+\Delta t} = \mathbf{F}^{Int}(t + \Delta t) - \mathbf{F}^{Ext}(t + \Delta t) \quad (2)$$

$$\bar{\mathbf{g}}_{t+\Delta t, n+1} = \bar{\mathbf{g}}_{t+\Delta t, n} + \frac{\partial \bar{\mathbf{g}}}{\partial \mathbf{U}} \delta \mathbf{U}_{n+1} = 0 \quad (3)$$

where $\bar{\mathbf{g}}_{t+\Delta t, n}$ is the unbalanced force at time-step $t + \Delta t$ and the n th iteration. If a snap-through is encountered in an analysis, some negative or null values will appear on the leading diagonal of the stiffness matrix, which leads to divergence of the analysis. In fact, the snap-through initiates dynamic behaviour which may be bounded by two stable equilibrium states. When the snap-through finishes, the whole structure may return to stable static equilibrium, provided that it is loaded at a slow rate.

In this research the Newmark integration scheme for nonlinear dynamic problems [13, 14] is adopted, due to its significant advantage in numerical computational efficiency. Because the behaviour of portal frames within Stage B (see Figure1) is quasi-static, it is not necessary to carry out a full dynamic solution procedure, which needs more computing time but finds the same results in regions of stable static equilibrium. A quasi-static solution procedure is therefore carried out to find out the steady-state behaviour of the structure. The dynamic model is activated only when singular values are detected on the leading diagonal of the stiffness matrix, and the static model can resume the analysis when a new steady-state is found in the dynamic modelling.

For dynamic analysis, both inertial and damping terms are added to the left-hand side of Eq. 1:

$$\mathbf{F}^I(t) + \mathbf{F}^D(t) + \mathbf{F}^{Int}(t) = \mathbf{F}^{Ext}(t) \quad (4)$$

The dynamic unbalanced force at step $t + \Delta t$ is then given by:

$$\bar{\mathbf{g}}_{t+\Delta t} = \mathbf{F}^{Int}(t + \Delta t) - \mathbf{F}^{Ext}(t + \Delta t) + \mathbf{F}^I(t) + \mathbf{F}^D(t) \quad (5)$$

where $\mathbf{F}^I(t)$ is the inertial force and $\mathbf{F}^D(t)$ is the damping force. When negative values appear on the leading diagonal of the stiffness matrix, the damping and inertial forces are added, to generate an effective stiffness matrix which always has positive diagonal values. This is very helpful to the Newton-Raphson solver which can still solve for the incremental displacements when partial failure of the structure happens.

To restrain the vibration of the structure about equilibrium states in fire a high damping, in excess of the system's critical damping, is required, so numerical damping, which only gives convergence at relatively small values, is not ideal for this model. Because the exact path of the motion is not very important in this case, the mass and damping are assumed to be lumped at the nodes, and the critical damping, relative to the lowest natural frequency and mass, is adopted in the model. The procedure discussed here has been incorporated into Vulcan [1-3] for modelling of steel structures in fire.

3. VALIDATION

3.1 Pitched Portal Frame

To test the capability of the dynamic model in dealing with geometrically nonlinear problems, a simplified pitched frame (Figure 2) with an increasing point load was tested by the dynamic model in Vulcan and the results compared with the commercial software ABAQUS [4]. Purely elastic material properties were assumed in this case, in order that the geometrical nonlinearity of the roof frame could be presented and the whole snap-through behaviour of the roof frame could be traced.

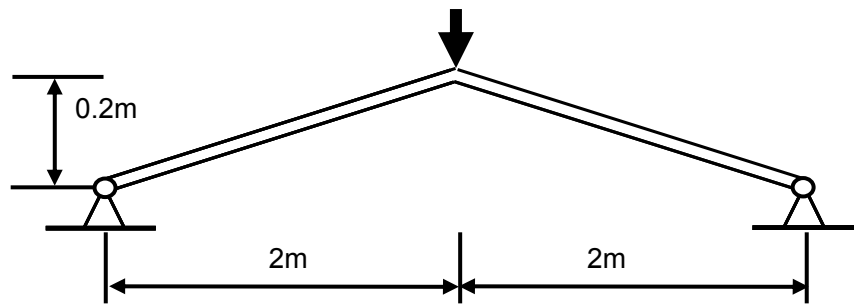


Figure 2. Initial Configuration of the Pitched Roof Frame

For this ideal symmetrical case, a bifurcation is encountered under completely symmetric loading. This causes a problem, even for dynamic analysis, in breaking the inherent symmetry, and the motion must be given the facility to move in an asymmetric fashion by introducing a small imperfection, which in this case is a very small additional load on one rafter. Beyond the limit-point the frame collapses dynamically until it reaches its almost-inverted position.

Because only two Gauss points control integration on each beam element, the accuracy of the result may rely on the element mesh density. According to the parametric study those results are shown in Figure 3 it is found that, after the deflection jump, the load-deflection curves of models with different element mesh density always tend towards the same stable static curve, on which the stiffness of the frame recovers due to the generation of tension in the members. Four elements per member are found to be adequate for convergence of the results, and the position of the limit point corresponds very closely to that given by full dynamic results from ABAQUS [4]. It is evident that this new dynamic model is capable of capturing the new stable position after snap-through of the pitched roof frame in a load-controlled analysis.

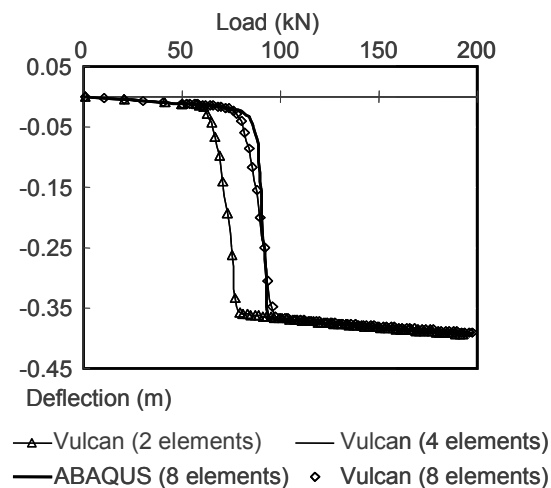


Figure 3. Vertical Displacements of the Apex

3.2 Twin-Bay Single-Storey Steel Portal Frame

A test was carried out on a twin-bay single-storey steel portal frame, as shown in Figure 4, which had been designed by Franssen and Gens [15]. This was modelled using 61 geometrically nonlinear beam-column elements [3]. The two-dimensional analyses of this model were adopted as a benchmark for several finite element programs [16], such as SAFIR [6] Dynamic, ABAQUS [4] Dynamic and ANSYS [5] Dynamic. Both dynamic and quasi-static analyses were performed to simulate this case; Figures 5 to 11 show the comparisons. It is evident that very good agreement was achieved between the current model (Vulcan) and the other software. The right-hand half-frame kept almost its original shape when the left-hand locally heated part had collapsed to column-base level. The collapsed shape of the frame is shown in Figure 12. This also attests that the current dynamic model can handle partial or local failure conditions in steel structures.

The current dynamic and Quasi-static analyses (Vulcan) give very close results for this case, but it is worth noting that the difference between the curves from these two methods reveals that, although very high damping is used, there is still some dynamic effect which remains in a transient dynamic analysis, and this may influence the results. To eliminate the dynamic effect in the analysis, quasi-static analysis is carried out in the following case studies.

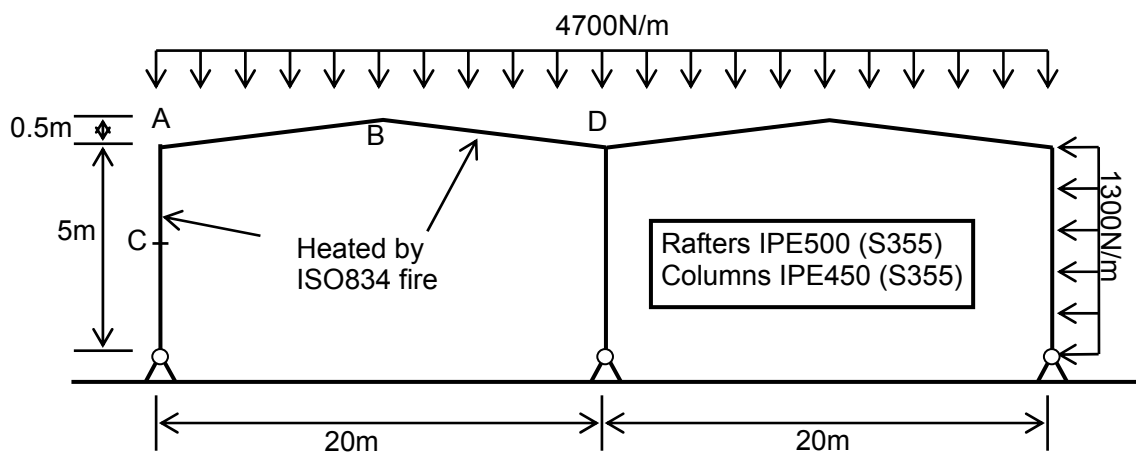


Figure 4. Initial Configuration and Loading Arrangement of Franssen's Model

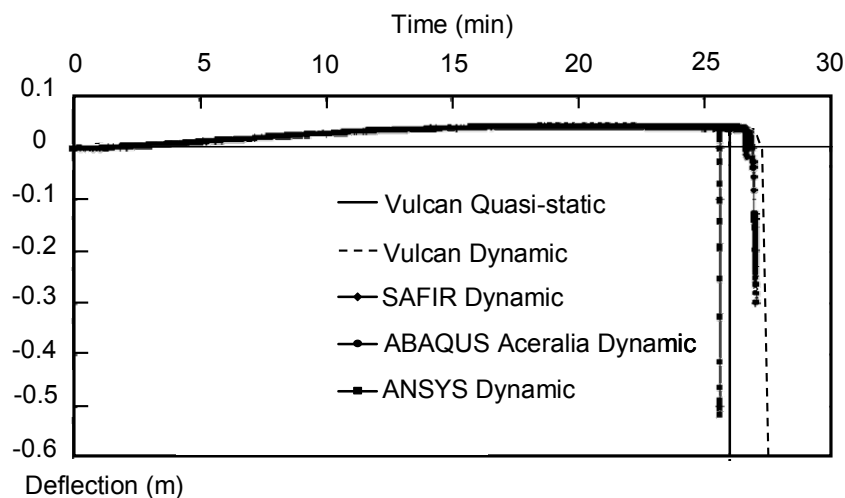


Figure 5. Vertical Displacement at Node A

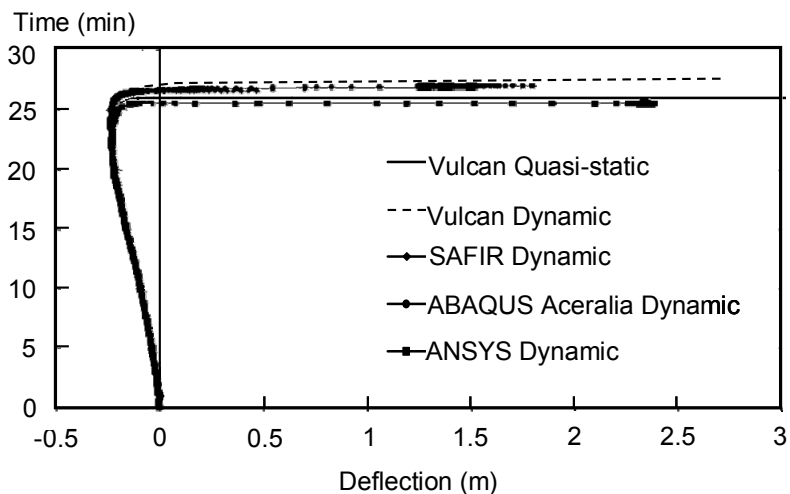


Figure 6. Horizontal Displacement at Node A

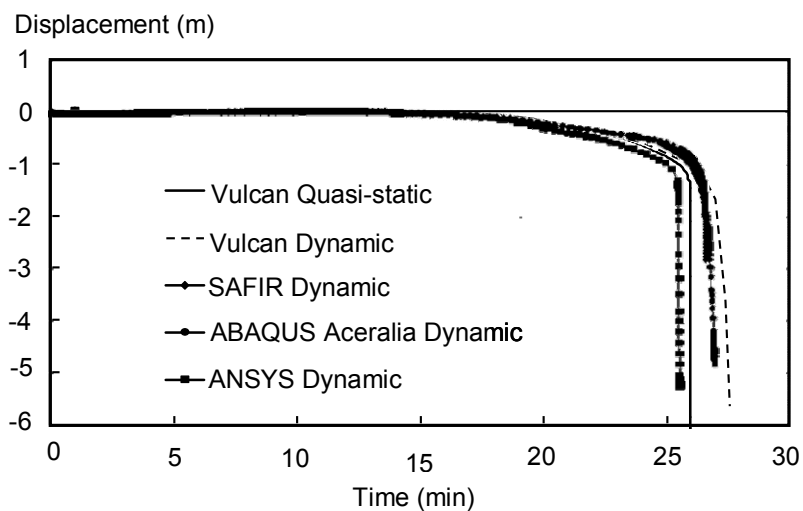


Figure 7. Vertical Displacement at Node B

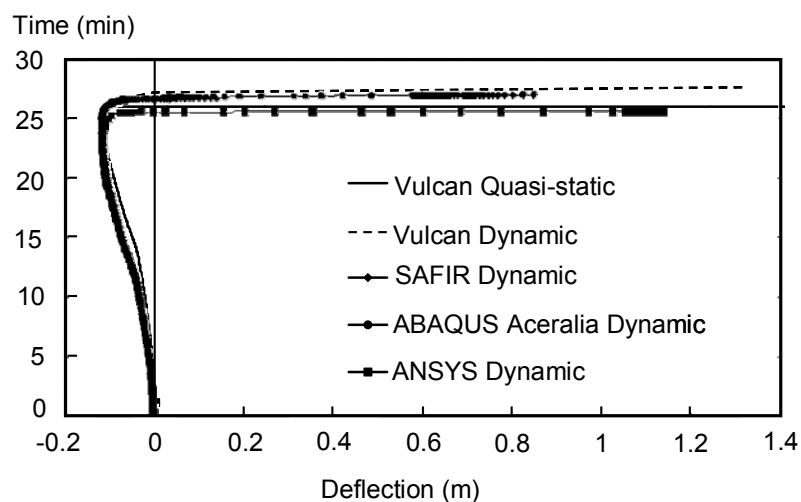


Figure 8. Horizontal Displacement at Node B

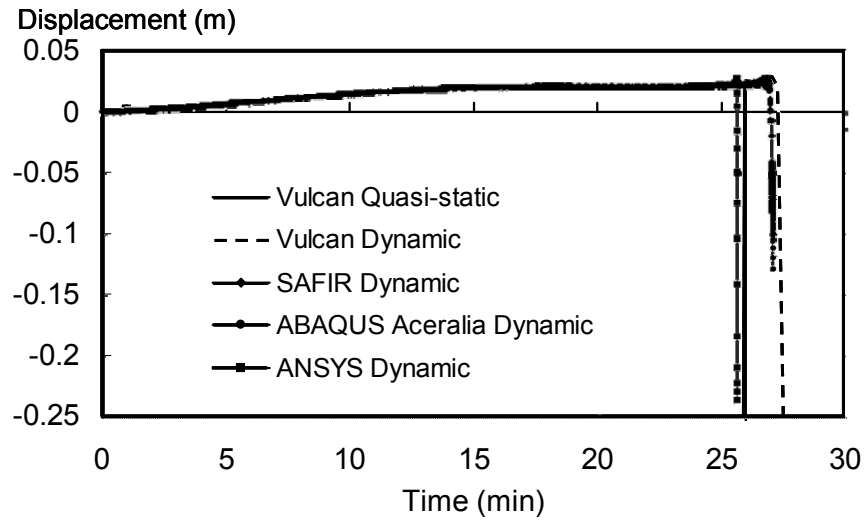


Figure 9. Vertical Displacement at Node C

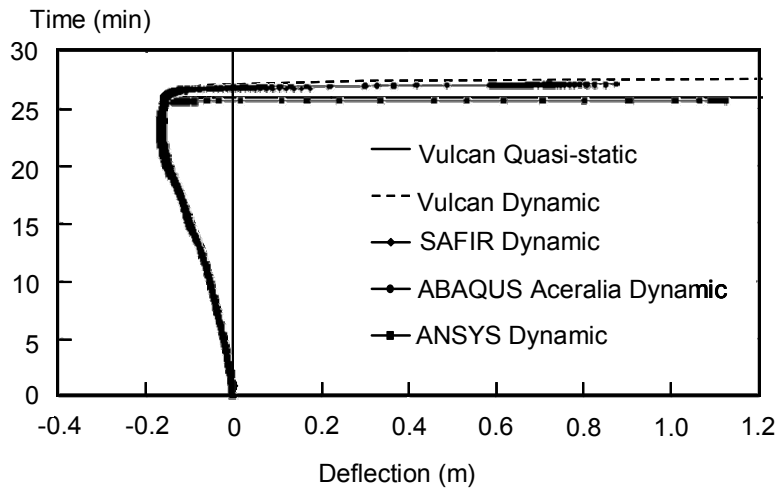


Figure 10. Horizontal Displacement at Node C

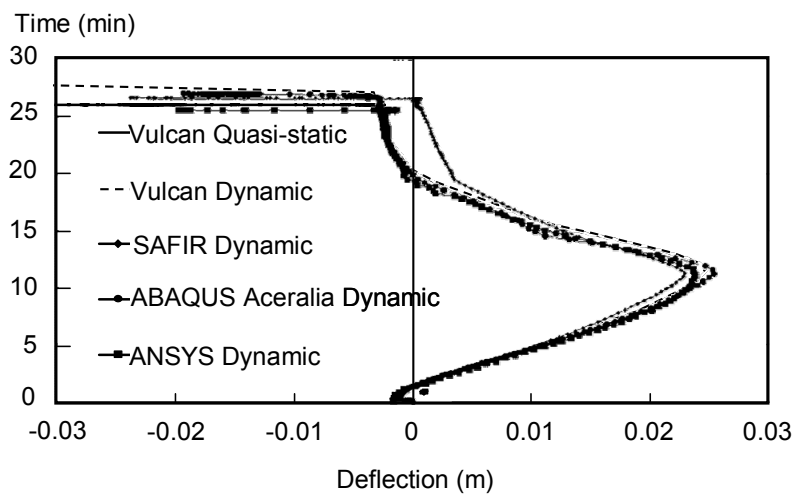


Figure 11. Horizontal Displacement at Node D

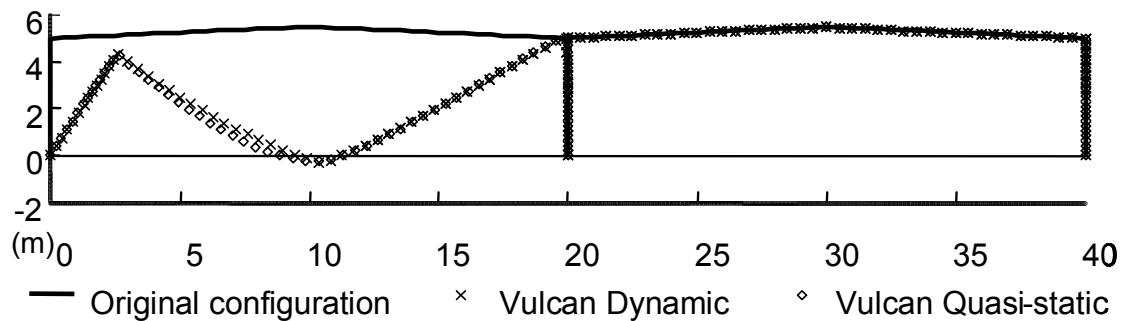


Figure 12. Deflection Shape of the Frame

4. CASE STUDY

According to previous research conducted at the University of Sheffield [11], the static analysis in Vulcan [1-3] has shown very good accuracy in comparison with experimental fire test results up to the large deflection jump which happens at the limit-load. This study is designed to trace the post-failure behaviour of the portal frame and to explore the failure mechanism under fire conditions. A two-dimensional portal frame model was designed as shown in Figure 13 according to plastic theory [17]. For convenience in modelling, the whole frame was assumed to have the same member cross-section size, and was designed using plastic theory for ultimate limit state conditions at ambient temperature. The design loadings and the fire limit state partial safety factors are listed in Table 1, and these loads were applied to the frame before heating. It was assumed that the whole frame was exposed in fire, and a uniform temperature distribution across the member sections was used. The distance between centres of adjacent portal frames was 6m, and a horizontal imperfection force of 1.7kN was applied to the left eaves connection. The temperature of the steel used in this test was calculated by the simple Eurocode 3 Part 1.2 method [18].

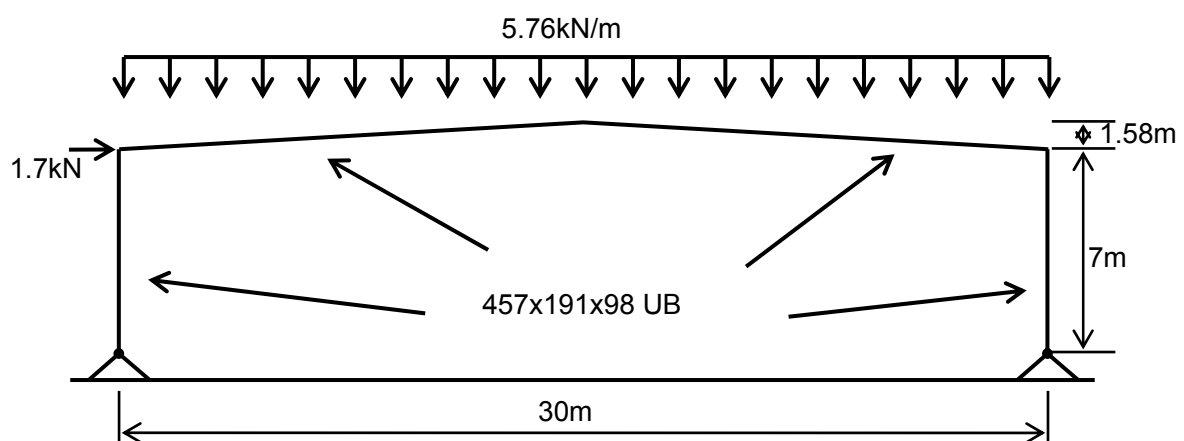


Figure 13. Numerical Model of the Pitched Portal Frame

Semi-rigid column bases were assumed, and modelled by springs with a rotational stiffness which was calculated using the rule recommended in the SCI design guidance document [19]. For a nominally rigid base the rotational stiffness is defined as $4EI_{column}/L_{column}$, and for a nominally pinned base the rotational stiffness is one tenth of this, $0.4EI_{column}/L_{column}$, where E is the Young's

modulus, I_{column} is the second moment of area of the column section and L_{column} is the length of the column. A mean value of rotational stiffness $2EI_{column}/L_{column}$, between the values for the nominally pinned and rigid bases, was defined for a nominally semi-rigid base.

Table 1. Load Combinations

Load Type	Unfactored load kN/m ²	Ambient Load factors	Fire load factors
Dead load	0.66	1.4	1.0
Imposed load	0.60	1.6	0.5

4.1 Frames Without Fire Protection

In this group of numerical tests, the frames are assumed to be uniformly heated. The vertical deflection of the apex and the horizontal deflections of the eaves are presented in Figures 14 to 16. It is clear that the failure modes are very similar, and that the difference in failure temperatures between the defined cases was not more than 2°C. The base rotational stiffness clearly does not show a significant influence on the behaviour of the portal frame for this heating profile.

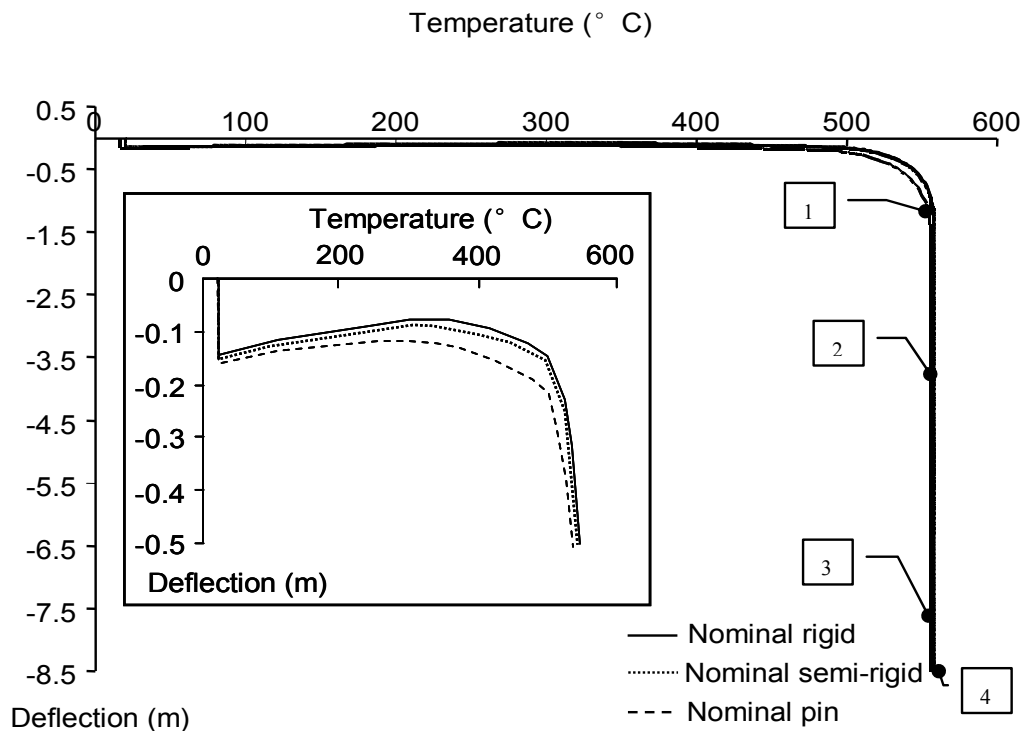


Figure 14. Vertical Displacement of Apex (Numbers are Referred to Figure 17)

Due to the thermal expansion of the steel, the apex initially deflected upward until the steel had reached about 330°C. Under the combined effect of elevated temperature and the external loading, the roof frame then began to lose stiffness and pushed the columns outward. The maximum outward deflection of the eaves was reached when the roof frame had fallen to eaves level at around 560°C (Shape 1 in Figure 17). At this moment, only the two sections adjacent to the eaves had a fully-plastic flange. Beyond this point the apex deflected below the eaves level and the

column tops were pulled inward. After a further temperature increment of about 2°C, one fire hinge had developed at each eaves, which provided considerably more flexibility. Near to the apex, the strains in the lower part of the rafter cross-section exceeded the plastic limit when the displacement of the apex was greater than half the original height of the frame (Shape 2 in Figure 17). The plastic region had expanded to about one fifth of the length of the rafters when the apex had deflected to base level, and because of the imperfection force at the left eaves the plastic regions had developed mainly in the left rafter. It is worth noting that, at this stage, more than half of the cross-section at the Gauss point closest to the left-hand column base had reached its plastic strength (Shape 3 in Figure 17). Figure 17 presents the sequence of collapse of the frame up to the point at which the apex had descended to column base level.

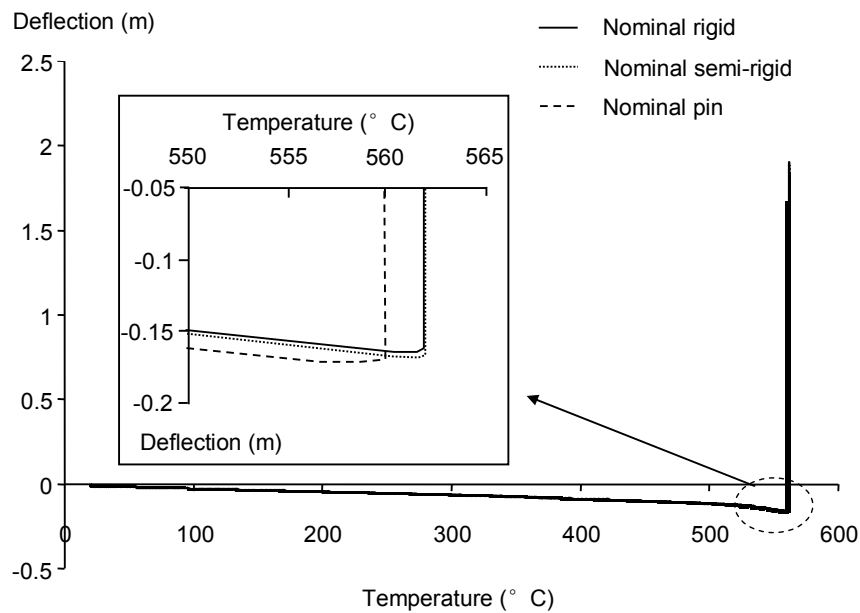


Figure 15. Horizontal Displacement of Left Eave

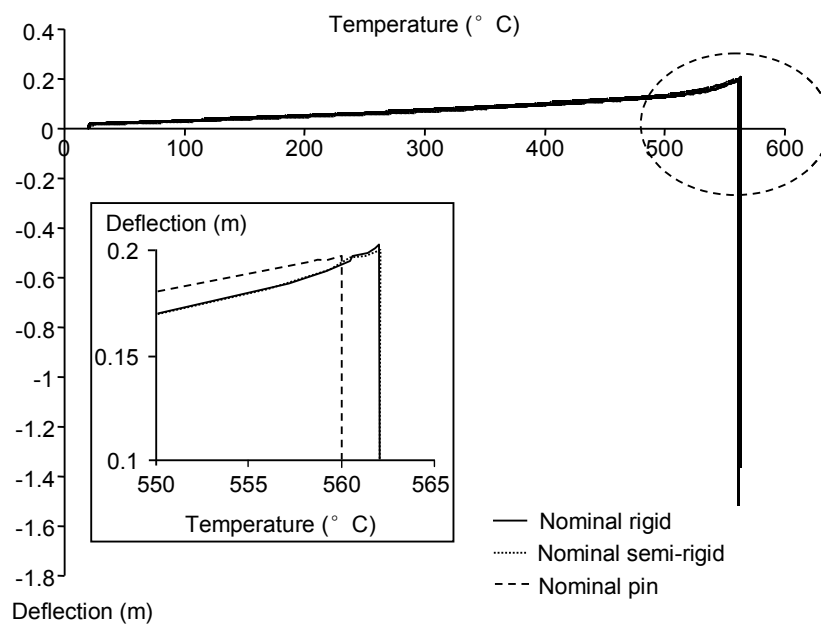


Figure 16. Horizontal Displacement of the Right Eave

The phenomenon of re-stabilisation after snap-through was not detected during these analyses because, beyond the stage at which plastic hinges were generated at the eaves, the columns were too flexible to provide enough restraint to produce any re-stabilisation.

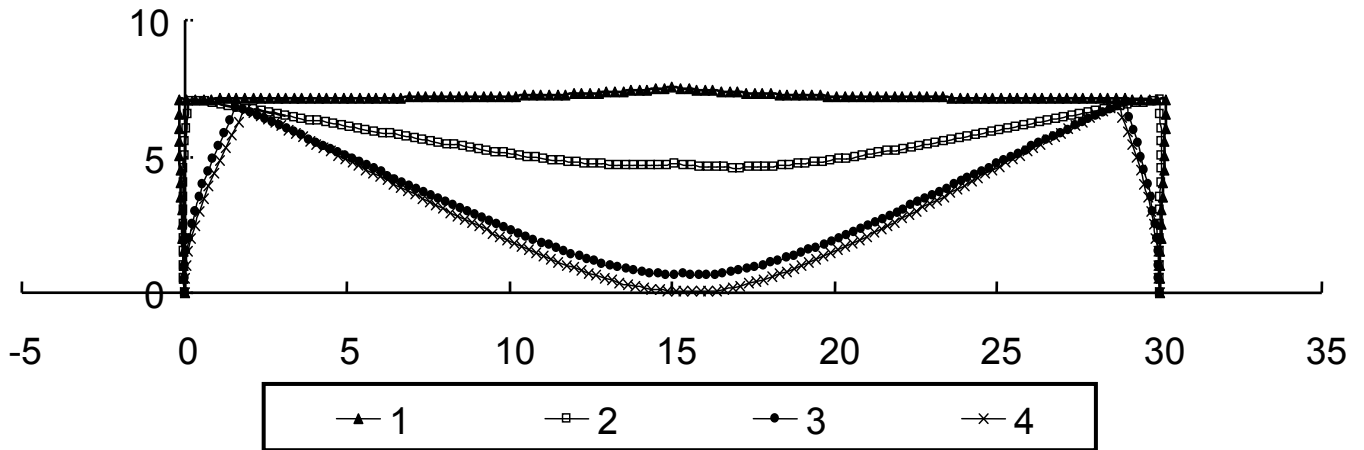


Figure 17. Failure Progress of Portal Frame with Nominally Rigid bases when only Rafters Heated

4.2 Frames with Columns Protected

The re-stabilisation after initial snap-through can be observed clearly for the cases in which the rafters are heated by the Standard Fire and the columns are protected. According to the load-deflection curves presented in Figures 18 to 20, the rotational stiffness of the column bases shows a significant influence in restraining the frame while the rafters are collapsing. A secondary stabilised position was found in all three models beyond the phase of rapid deflection.

Due to the expansion of the rafter, the apex initially rose, as the columns were pushed outward symmetrically. The apex then began to descend as the eaves continued to move outward, because of the degradation of the steel at elevated temperature and the vertical load on the roof. When one fire hinge had developed near each eaves connection, the peak load at snap-through was initiated, the roof frame lost stability and collapsed downward (Shape 1 in Figure 21), and the columns were pulled inward by the inverted rafters after the apex had fallen below the eaves level (Shape 2 in Figure 21). The frame later achieved stability once more when the rafters had almost completely inverted with the apex deflection being over twice its initial height above the eaves level (Shape 3 in Figure 21). With increasing temperature, the redistribution of internal forces led to an increase of curvature near to the apex. This caused the third plastic hinge to form at the apex. The frame was statically determinate at this stage, causing a plateau to form on the load-deflection curve, which ended as the fourth plastic hinge was generated at the left base to complete the mechanism (Shape 4 in Figure 21). Thereafter the left column was pulled inward rapidly until the apex reached column base level (Shape 5 in Figure 21).

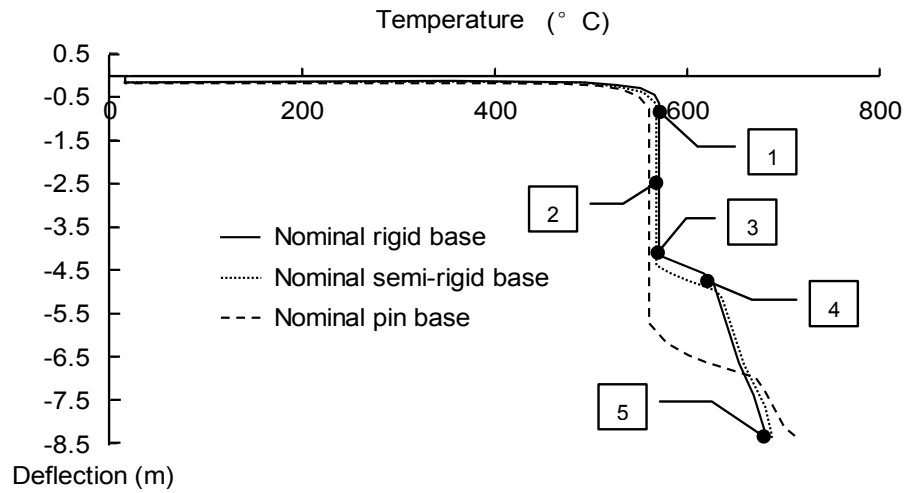


Figure 18. Vertical Deflection of the Apex (Numbers refer to Figure. 21)

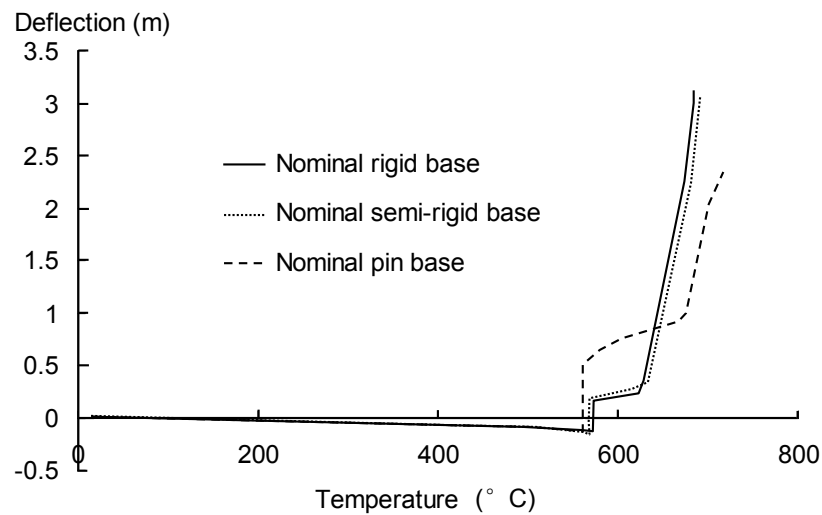


Figure 19. Horizontal Deflection of the Left Eave

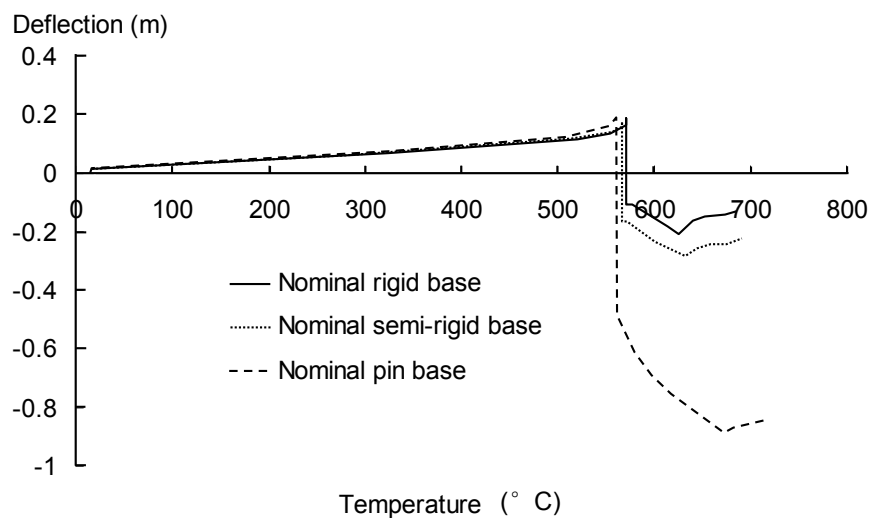


Figure 20. Horizontal Deflection of the Right Eave

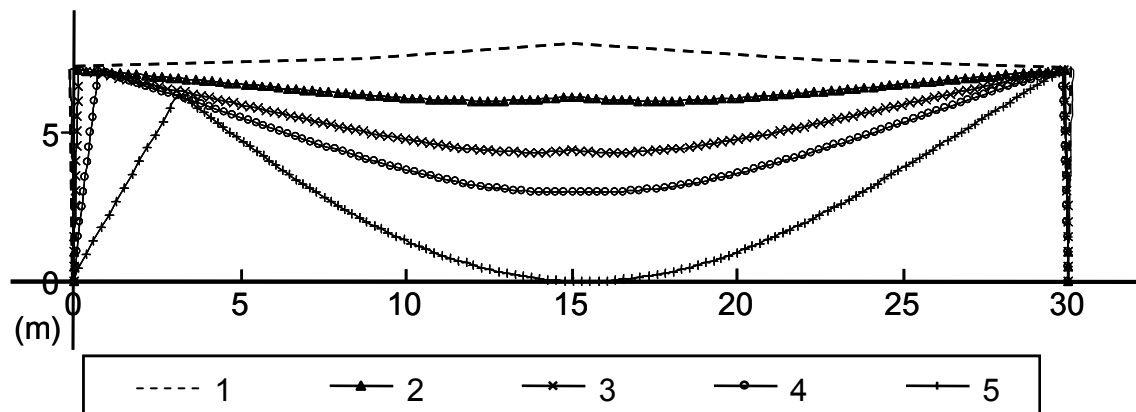


Figure 21. Failure Progress of Portal Frame with Nominally Rigid bases with only Rafters Heated

5. CONCLUSION

The simplified dynamic model and quasi-static solution procedure developed for this study is suitable to overcome the temporary instabilities encountered in numerical analysis of structures. It could also be used to accurately predict progressive collapse of portal frames and other steel frames in fire.

For the simplified portal frame case study presented here, the rotational stiffness of column bases had no significant effect on the failure mode of the portal frames in two heating profiles. For the whole-frame-heated cases, when the two fire hinges formed near the eaves connections, the frames began to lose stability and a full mechanism formed at the same temperature, so that collapse occurred to the base level almost immediately. However, for the column-protected cases, after the initial mechanism which caused the collapse of the rafters, as for the previous heating profile, a second failure mechanism was created at a higher temperature, which led to the final collapse of the frame to base level.

The different base flexibilities did not show a very significant influence on the structural behaviour in cases where the whole frame was uniformly heated. However the rotational stiffness of column bases showed a significant influence on the re-stabilised positions after snap-through of the roof frame for the roof heated case (see Fig. 18). Critical temperatures will be lower for frames with less stiff base connections, if the critical temperature is defined as the temperature at which the structure finally begins to lose stiffness as the final hinge develops, shown as Point 4 on Figure 18. This study reveals that initial instability of the rafters does not always lead to collapse of the frame, because relatively stiff and strong columns may be able to re-stabilise the structure for a while in fire. All frames lose stability in a four-hinge mechanism, and very large deformations of the rafters were experienced. It seems reasonable to calculate the critical temperature based on plastic theory and the thermal properties of the steel for fire safety design rather than design the moment resistance base connection to prevent inclination of the columns.

REFERENCES

- [1] Huang, Z., Burgess, I.W. and Plank, R.J., "Modeling Membrane Action of Concrete Slabs in Composite Buildings in Fire. I: Theoretical Development", *Journal of Structural Engineering*, ASCE, 2003, Vol. 129, No. 8, pp. 1093-1102.
- [2] Huang, Z., Burgess, I.W. and Plank, R.J., "Modeling Membrane Action of Concrete Slabs in Composite Buildings in Fire. II: Validations", *Journal of Structural Engineering*, ASCE, 2003, Vol. 129, No. 8, pp. 1103-1112.
- [3] Huang, Z., Burgess, I.W. and Plank, R.J., "3D Modelling of Beam-columns with General Cross-sections in Fire", Paper S6-5, Third International Workshop – Structures in Fire, Ottawa, Canada, 2004, pp. 323-334.
- [4] ABAQUS, "ABAQUS Analysis User's Manual-Version 6.5", ABAQUS INC., USA, 2004.
- [5] ANSYS, "ANSYS User's Manual for Revision 8.0-Volumn IV-Theory", Swanson Analysis SYSTEM, INC., Houston USA, 1992.
- [6] SAFIR, "A Thermal/Structural Program Modelling Structures under Fire", Jean-Marc Franssen, Proc. NASCC 2003, A.I.S.C. Inc., Baltimore, 2003.
- [7] Morris, L.J. and Plum, D.R., "Structural Steelwork Design to BS 5950 (2nd edn.)", Addison Wesley Longman, England, 1996.
- [8] The Building Regulations 1991, "Fire Safety, Approved Document B", Department of the Environment and The Welsh Office, HMSO, 1991.
- [9] Newman, G.M., "Fire and Steel Construction: The Behaviour of Steel Portal Frames in Boundary Conditions (2nd edn.)", The Steel Construction Institute, UK, 1990.
- [10] Song, Y., Huang, Z., Burgess, I.W. and Plank, R.J., "The Design of Pitched-Roof Steel Portal Frames Against Fire", *Proceeding of 5th International Conference on Advances in Steel Structures*, Singapore, 2007, Vol. 3, pp. 728-733.
- [11] Wong, S.Y., "The Structural Response of Industrial Portal Frame Structures in Fire", PhD Thesis, University of Sheffield, 2001.
- [12] Song, Y., Huang, Z., Burgess, I.W. and Plank, R.J., "A New Numerical Model for the Industrial Steel Frames in Fire", Research Report F/08/01, University of Sheffield, 2008.
- [13] Bathe, K.J., "Finite Element Procedures", Prentice-Hall, New Jersey, 1996.
- [14] Crisfield, M.A., "Non-linear Finite Element Analysis of Solids and Structures, Volume 2: Advanced Topics", John Wiley & Sons Ltd., Chichester, 1998.
- [15] Franssen, J.M. and Gens, F., "Dynamic Analysis Used to Cope with Partial and Temporary Failures", Paper S6-3, Third International Workshop - Structures in Fire, Ottawa, Canada, 2004, pp. 297-310.
- [16] Vassart, O., Brasseur, M., Cajot, L.G., Obiala, R., Spasov, Y., Griffin, A., Renaud, C., Zhao, B., Arce, C. and De La Quintana, J., "Final Report: Fire Safety of Industrial Halls and Low-rise Buildings", ECSC Steel RTD Program, CEC Agreement 7210-PR-378, 2005.
- [17] Horne, M.R. and Morris, L.J., "Plastic Design of Low-Rise Frames", Granada, UK, 1981.
- [18] CEN, "Eurocode 3: Design of Steel Structures: Part 1.2: General Rules – Structural Fire Design", European Committee for Standardization, 2005.
- [19] Salter, P.R., Malik, A.S. and King, C.M., "Design of Single-Span Steel Portal Frames to BS 5950-1:2000 (2004 Edition)", SCI Publication P252, The Steel Construction Institute, Ascot, United Kingdom, 2004.