A NEW DESIGN METHOD FOR INDUSTRIAL PORTAL FRAMES IN FIRE

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ABSTRACT

For single-storey steel portal frames in fire, especially when they are situated close to a site perimeter, it is imperative that the boundary walls stay close to vertical, so that fires which occur are not allowed to spread to adjacent properties. A current UK fire design guide requires either that the whole frame be protected as a single element, or that the rafter may be left unprotected if column bases and foundations are designed to resist the forces and moments generated by rafter collapse, in order to ensure the lateral stability of the boundary walls. This can lead to very uneconomical foundation design and base-plate detailing.

In previous studies carried out at the University of Sheffield it was found that a fundamental aspect of the collapse of a portal frame rafter is that it usually loses stability in a "snap-through" mechanism, but is capable of re-stabilising at high deflections, when the roof has inverted but the columns remain close to vertical. Numerical tests performed using the new model show that the strong base connections recommended by the current design method do not always lead to a conservative design. It is also found that initial collapse of the rafter is always caused by a plastic hinge mechanism based on the frame's initial configuration. If the frame can then re-stabilize when the roof is substantially inverted, a second mechanism relying on the re-stabilized configuration can lead to failure of the whole frame.

In this paper, a portal frame with different bases is simulated numerically using *Vulcan*, investigating the effect of different base strength on the collapse behaviour. The test results are compared with the failure mode assumed by the current design method. A new method for the estimation of re-stabilized positions of single-span frames in fire, using the second failure mechanism, is discussed and calibrated against the numerical test results.

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1. INTRODUCTION

In the UK, fire protection of portal frames differs from that of other structures. Normally the roof of the building need not be protected in fire, but the portal frame is considered as a single structural element due to the rigid connection between rafters and columns, so if any part of it needs to be protected against fire, the whole frame has to be protected. It is also specified by current building regulations [1] for fire safety that the boundary wall of a building has to be fire-protected to stop fire spreading from one building to the next. As a result, the whole portal frame needs to be fully protected in fire as the boundary wall.

The design of pitched portal frames for fire conditions usually follows the SCI document P087 [2]. This design guide allows portal frames to be unprotected if the column bases have the sufficient resistance to the overturning moments (OTM) caused by the collapse of the rafters, in order to guarantee the longitudinal stability of the boundary wall. The calculation of the OTM at the column bases is based on the symmetrical failure mechanism shown in Fig. 1. In this design process, assumptions about haunch length, rafter elongation, fire hinge moments and the limiting inclination of the column are applied to simplify the calculation model. It is worth noting that these assumptions are inconsistent. The assumed rafter elongation implies a steel temperature of 1400°C, while the strength degradation of the rafter fire hinge moments relates to 900°C. The method also implies that the stability of the boundary wall is assured if the inclination of the column is less than 1°. However, the maximum horizontal movement of eaves is specified as 1/300 of the height of column in a well known design guide for masonry walls [3], which means the column inclination should remain within 0.2°. This method also assumes that the haunch length is one tenth of the span, although haunches may be of any size, if they exist.

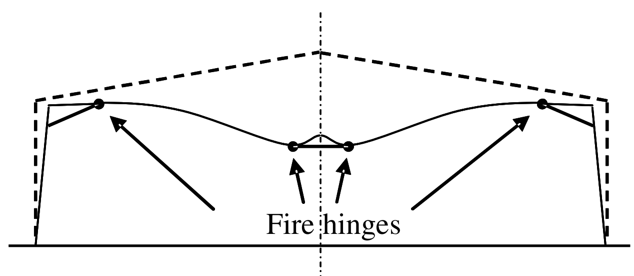

Fig. 1 – Failure mechanism used in current design method

The numerical tests on a typical single-storey industrial frame [4] will show that this symmetric failure mechanism does not always happen in fire, and even nominally rigid bases can not guarantee that the column inclination is always within 1°. The OTM required for a nominally rigid frame could be much bigger than the design value, so the assumptions used to simplify the current design model may not always lead to conservative results in fire.

2. S.Y. WONG'S SIMPLE METHOD

To reveal the true failure mechanism of portal frames in fire, three natural fire tests on a scaled portal frame were carried out at Sheffield University [5]. It can be seen from inspection of one of the tests that the roof frame deflected downward after the rafters initially lost stability in fire and rested in a stable position near to eaves level until the end of the test. It seems that, after initial snap-through of the rafters, the portal frame has re-stabilized with high deflection, and that the inclination of the column can stay within a relatively small region. A simple method, used to predict the critical temperature of single-span portal frames with simple base connections, was developed by S. Y. Wong [5]. According to plastic theory for the mechanism shown in Fig. 2, an equation can be set up equating the work done by the external and internal forces:

$$wL^2\theta = M_p[\theta + \eta_1\theta + \eta_2(\varphi + \theta)] \tag{1}$$

where η_1 and η_2 are the strength reduction factors for the fire hinges at corners 1 and 2 in Fig. 3. If a localized fire is assumed, the strength reduction factor on the unheated corner is set as 1. If the whole roof is assumed to be heated by fire, the critical temperature of the frame can be predicted according to a single strength reduction factor η using Eqn.2.

$$\eta = \frac{wL^2}{M_p(3 + \frac{h_2 + 2h_1}{h_2})} \tag{2}$$

A series of parametric studies on the single-span pitched portal frame performed by Wong [5] using *Vulcan* [6-8] showed that the critical temperatures calculated by this simple method agree with both the numerical and fire test results in predicting the initial collapse of the roof in fire. However, due to the limitations of the iterative solver used in the static model, the numerical analysis was unable to continue beyond this point, so the re-stabilized position observed in the fire test could not be traced.

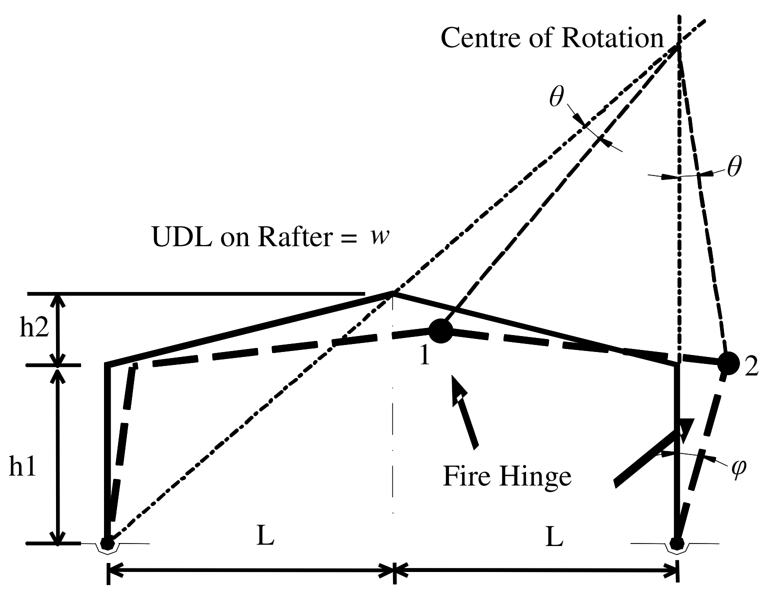

Fig. 2 – The model in Wong's simple design method

3. NUMERICAL TESTS

Recently a new dynamic solver has been incorporated into *Vulcan* [6-8] and a quasi-static solution procedure has been adopted to overcome the problem described above.

This new model has been validated on benchmark problems [9] using commercial software, and calibrated against Wong's fire test result [10]. It is evident that the new model is capable of dealing with partial failure and temporary instabilities encountered in structural analysis.

3.1 NUMERICAL MODEL

To model the whole failure sequence of a portal frame in fire, a series of single-span pitched roof frames were tested using the new dynamic model. A uniform section was assumed and the frame was designed according to the load combinations in Table 1 and plastic theory [11], as shown in Fig. 3. As an imperfection to this symmetric structure, the left eave is 0.02m higher than the right eave. It is assumed that the rafters are heated by an ISO834 standard fire [12], and the steel temperature are calculated using the simple Eurocode 3 Part 1.2 method [13].

The semi-rigid base was modelled by dummy elements [14], and no axial restraint was applied at the outer ends of these dummy elements. The capacity of the semi-rigid bases was controlled by the moment resistance of the dummy element according to Eqn. 3.

$$S_{x,dummy} = kS_{x,column} \tag{3}$$

where S_x is the plastic modulus of the column about its major axis and k is the strength reduction factor. Eurocode 3 Part 1.8 [15] assumes that connections can be classified as nominally pinned if their design moment resistance is less than half of the design moment resistance required for a full strength joint, which has the same moment resistance as the connected members. This means that, for a full-strength column base connection, when the moment resistance of the dummy element is equal to the design strength of the column, it represents a full-strength connection, and when its design strength is less than 0.5 times the strength of the column, it can be classified as nominally pinned. As a result, the value of k should lie between 0.5 and 1.0. When the moment resistance of the dummy element is determined, the initial stiffness of the connection can be varied by changing the length of the dummy element, following Eqn. 4.

$$L_{dummy} = k_{flex} \frac{3I_{dummy} L_{column}}{I_{column}} \tag{4}$$

Here L_{dummy} and L_{column} are the lengths of the dummy element and column, I_{dummy} and I_{column} are the second moment of area of the dummy element and column sections, and k_{flex} is the initial stiffness reduction factor. According to the connection classification by stiffness in Eurocode 3 Part 1.8 [15], the initial stiffness of a nominally pinned connection should be less than $0.5EI_{column}/L_{column}$. The value of k_{flex} was assumed to be 2.0 in all the models used in this study.

Table 1: Load combinations

Load Type	Unfactored load kN/m^2	Ambient Load factors	Fire load factors
Permanent load	0.66	1.4	1.0
Variable load	0.60	1.6	0.5

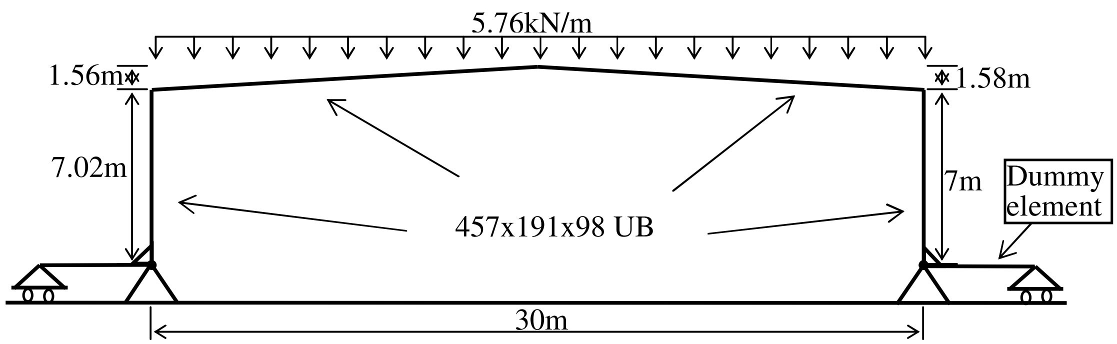

Fig. 3 – Numerical model of the pitched portal frame with semi-rigid bases

3.2 RESULTS OF NUMERICAL TESTS

It was found from the numerical tests that, no matter which base strength was applied to the frame, the roof initially lost stability in fire at almost the same temperature of around 563°C, but when the base strength was greater than 55% of the column capacity, the frame was able to re-stabilize, which is shown as the broken lines in Fig. 4.

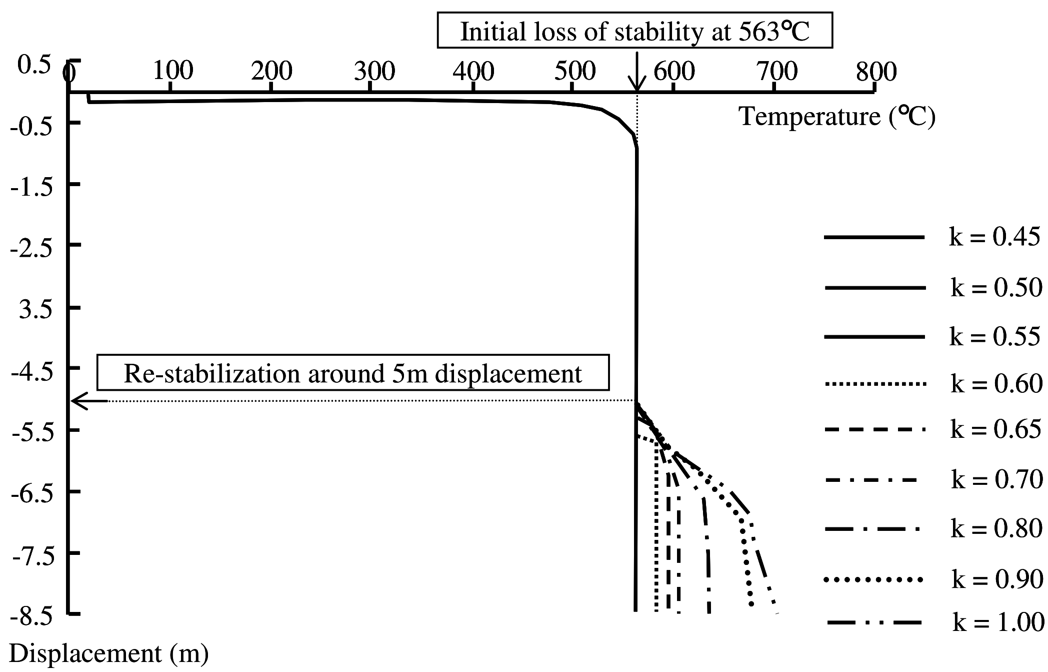

Fig. 4 – Vertical displacements of the apex from numerical modelling

The initial collapse of the pitched roof was caused by the occurrence of fire hinges at the eaves and apex. For the frames with column bases weaker than 55% of the column moment capacity, the roof continued to collapse to base level. This did not happen to the frames with stronger bases, which re-stabilized at an apex displacement of around 5m. In these cases the deformation remained relatively stable against temperature beyond this point. The frame lost stability again when one base began to plastically yield, so that a four-hinge mechanism (with hinges at the two eaves, apex and one base) formed. The temperature at which this second mechanism formed changed with the base capacity. Frames with stronger bases could remain stable to higher temperatures.

3.2 COMPARISON AGAINST CURRENT DESIGN MODEL

The numerical tests showed that collapse of the single-span portal frame is due to two phases of plastic mechanisms. In the current design model, only the symmetrical collapse mechanism of rafters, shown in Fig. 1, is adopted, and this only concerns the initial loss of stability of the frame in fire. It is worth finding whether the prediction from the current design model is always conservative compared with the numerical tests.

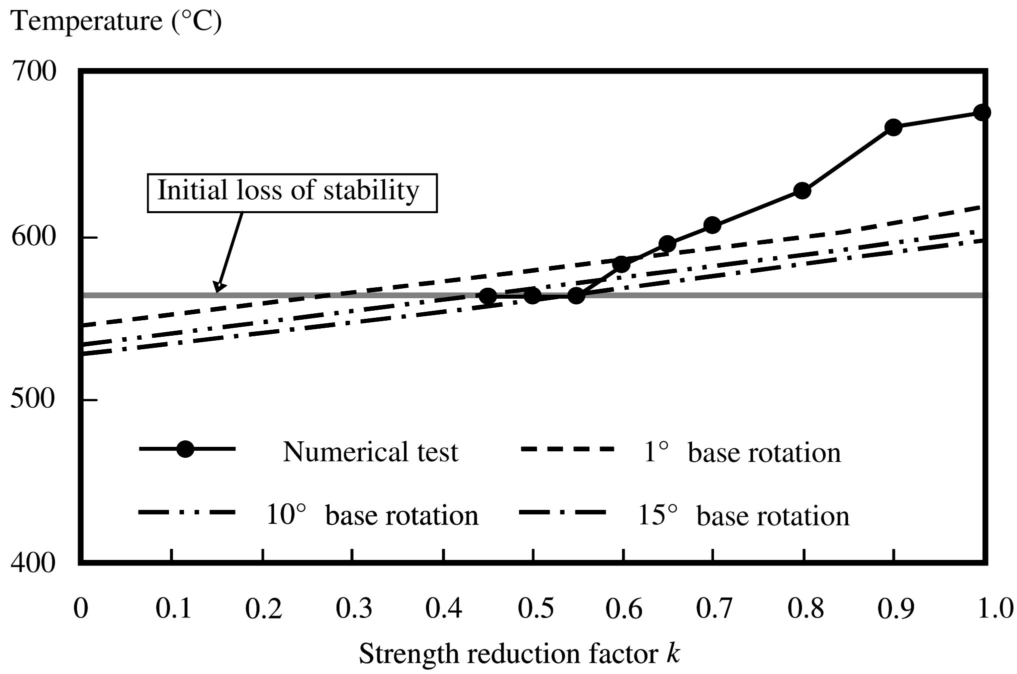

Fig. 5 – Comparison between the numerical results and current design model

In the current design model the effect of haunches, rafter elongations, fire hinge moment and column inclination are known values, so reactions at the base can be calculated from force equilibrium. To compare with the test results, the haunch length was assumed as zero in the design model and the steel properties defined in Eurocode 3 Part 1.2 [13] were used for the rafter elongation and moment capacity at elevated temperatures. It was assumed that the base capacity in the numerical test just equals the OTM required by this model, so the OTM at the base could be given by multiplying the moment resistance of the column by the appropriate strength reduction factor. Applying different assumptions of base rotation, the relationship between the temperature and strength reduction factor can be represented as the broken lines in Fig. 5.

According to the numerical modeling, all frames initially lose stability at almost the same temperature, which is marked by the grey line in Fig. 5. The temperature that causes the frame either to collapse to ground level directly, or to lose stability a second time before collapsing to ground level, is defined as the critical temperature of the frame in fire. The critical temperatures of frames with different bases are represented in Fig. 5 by the solid line and points.

The critical temperatures predicted by the current design model increase with the base strength, but decrease with higher base rotation. Comparing the current design model with the numerical modeling in predicting initial loss of stability, when the moment capacity of the base is greater than 30% of the column capacity the design model with 1° column inclination gives higher critical temperature for rafter collapse, which suggests longer fire resistance than the numerical model. However when the strength reduction factor is less than 0.3, the collapse temperature from the design model should be lower than the numerical results. If re-stabilization of the roof frame is considered in this comparison, the prediction from the current design model gives a higher critical temperature than the numerical modeling when the base strength reduction factor is between 0.3 and 0.6.

For this specific portal frame, the current design model seems unsuitable for the frames with bases stronger than 30% of the column strength, provided no fire protection is assumed. For frames with simple bases, if fire protection is designed according to the critical temperature, the current design model may not always give conservative results.

4. NEW DESIGN METHOD

A new design method which attempts to predict the collapse of portal frames with simple base connections is being developed as an extension to Wong's work. The heating profile adopted in the current design guide [2] is assumed for this simple method; this is similar to the temperature distribution in a large compartment fire. Two failure mechanisms can be identified during the collapse sequence of pitched portal frames.

4.1 FAILURE MECHANISMS

When the roof begins to deform downward in fire under the effect of the degradation of the steel and gravity load, the columns are pushed outward due both to the change of geometry of the rafters and to their thermal expansion. For a portal frame with frictionless base connections, high rotations can be generated at these bases, caused by either elastic or plastic deformation, and these rotations, together with the fire hinges at the apex and eaves, can establish a sway mechanism. Wong's simple model uses this mechanism, whose kinematics is referred to the initial configuration of the portal frame. This method can only apply to the frame's initial lose the stability at relatively low deflections. This is referred to as the first phase failure mechanism.

The initial collapse of the roof frame may initiate a sway mechanism leading to collapse of the whole frame, or the columns may be pulled back towards the upright position (see shape ABCDE in Fig. 6) due to the collapse of the rafters in fire. In the latter case the change of direction of rotation causes elastic unloading of the base moment and the base rotation is effectively locked. When the apex deflects to below eaves level and the columns are pulled inward this causes the base moments to increase again, and due to the inclination of the column relative to the rotation of the adjacent rafter, the moment on one eave starts to reverse, leading to the locking of the hinge at this eave. This causes the frame to re-stabilise at the position (Shape AB_0C_0DE in Fig. 6) where the internal angle between one column and the connected rafter stops closing and starts opening. With further increase of the pulling force at the column top caused by the catenary action of the inverted roof, the fire hinges at the eave and column base can be mobilised again (shape AB'C'DE in Fig. 6) and a new mechanism is established, which allows complete collapse of the second phase failure mechanism to take place.

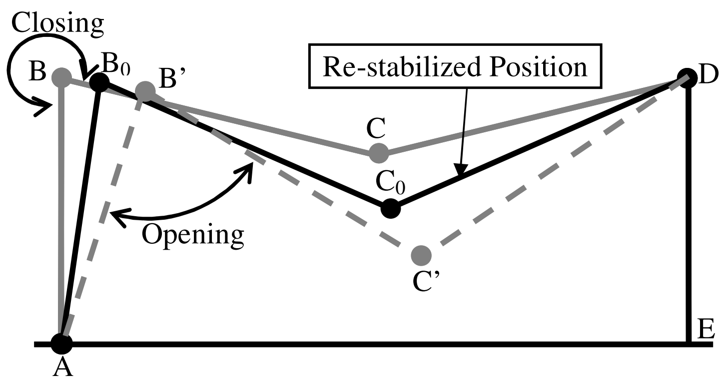

Fig. 6 – Illustration of the second phase failure mechanism

This new design method mainly focuses on the collapse of portal frames caused by the second phase mechanism in fire conditions, and aims to predict the critical temperatures by calculating the strength reduction factor of the fire hinge moment based on a work balance in the second phase failure mechanism. Because of the significant deformation of the roof frame before the mechanism forms, unlike Wong's simple method which can simply refer to the initial configuration of the frame, this method needs to find the re-stabilised position and the critical position at which the second phase mechanism is established including both the elongation of the rafter and the degradation of the moment resistance of the rafter section. Moreover, because a plastic hinge at one column base is essential to the second phase mechanism, the strength of the column bases is included in this new method.

4.2 ESTIMATION OF THE RE-STABILIZED POSITION

As shown in Fig.7, the re-stabilized position of the frame is determined by the movement of hinges B_0 and C_0. When the angle AB_0C_0 is smallest, the apex (point C_0 in Fig. 7) should lie on the line AD. The coordinates of the point $C_0(x_{C0}, y_{C0})$ and $B_0(x_{B0}, y_{B0})$ can be calculated as follows:

$$x_{B0} = h_1 \sin\left\{\arctan\left(\frac{S}{L}\right) - \arccos\left[\frac{\left(\sqrt{h_1^2 + L^2} - S\right)^2 + h_1^2 - S^2}{2h_1\left(\sqrt{h_1^2 + L^2} - S\right)}\right]\right\} \tag{5}$$

$$y_{B0} = h_1 \cos\left\{\arctan\left(\frac{S}{L}\right) - \arccos\left[\frac{\left(\sqrt{h_1^2+L^2}-S\right)^2 + h_1^2 - S^2}{2h_1\left(\sqrt{h_1^2+L^2}-S\right)}\right]\right\} \tag{6}$$

$$x_{C0} = L\left(1 - S/\sqrt{h_1^2+L^2}\right) \tag{7}$$

$$y_{C0} = h_1\left(1 - S/\sqrt{h_1^2+L^2}\right) \tag{8}$$

in which h_1 is the column height, L is the span of the frame and S is the length of a rafter.

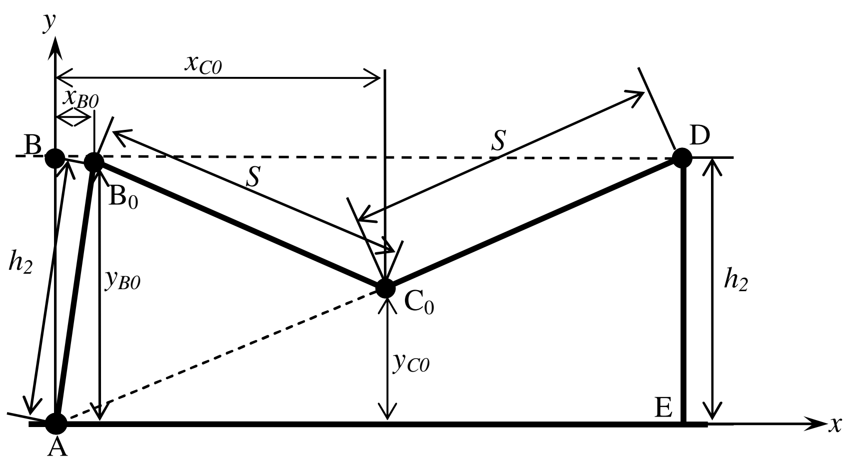

Fig. 7 – Mechanical model for the estimation of the re-stabilized position

This method shows that the frame of Fig. 3 can re-stabilize when the apex deforms to about 3.43m below the eaves level; adding the distance from the apex to the eave level which is 1.58m, the total vertical displacement of the apex should be around 5m, which matches very well with the numerical results presented in Fig. 4. This confirms that the re-stabilization during the collapse of the portal frame is caused by the locking of the plastic hinge near to an eave joint as it unloads. Once the opening of the locked angle exceeds the elastic rotation limit, the frame loses stability again.

5. CONCLUSION AND FUTURE WORK

In this paper, numerical experiments on a single-spanned pitched portal frame with different semi-rigid bases are presented. Numerical results are compared with a current design model which is widely used in the UK. The study shows that re-stabilization of the portal frame at high deflections can postpone the collapse of the whole frame, and its fire resistance may exceed the prediction given by the current design method.

It has been observed in previous physical and numerical tests that the collapse of the portal frame might be caused by a second phase mechanism which happens after inversion of the roof. Based on this failure mechanism, a new approach to estimate the re-stabilized configuration has been developed and the prediction from the new method has been confirmed by the numerical test results.

In the next step of this study, this new design method will be improved by including the prediction of the final (second phase) collapse mechanism. It is expected that the fire resistance of the portal frame with simple base connections can be evaluated using this new method. Instead of only having a single option, of designing moment-resisting base connections, engineers will then have more options to guarantee the fire resistance of portal frames under fire conditions.

The initial rotational stiffness of a "simple" column base can be much greater than that assumed in the numerical testing here. Numerical tests on frames with different initial base stiffness are in progress, and its effect will be reported in due course.

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