A NEW DESIGN METHOD FOR INDUSTRIAL PORTAL FRAMES IN FIRE

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INTRODUCTION

For single-storey steel portal frames in fire, especially when 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 [1] requires either that the whole frame be protected as a single element, or that the rafter can be left unprotected but the column bases and foundations should be designed to resist the forces and moments necessary to prevent collapse of rafter, in order to ensure the lateral stability of the boundary walls. Some arbitrary assumptions regarding the behaviour of the frame in fire, which are used to simplify this current design model, can lead to very uneconomical foundation design and base-plate detailing. Further understanding of the behaviour of portal frames in fire is required, to provide other design options so that over-design of column bases and foundations can be avoided, and a more reasonable prediction of real critical temperatures can be made.

On the basis of fire tests, a simplified method to estimate the critical temperatures of portal frames in fire was developed by Wong in 2001 [2] for single-span portal frames with simple base connections. It was shown by numerical modelling that this method could predict the temperature at which the rafters initially lose stability in fire. A recently developed quasi-static analysis [3], implemented in the program *Vulcan*, using a combination of static and dynamic solvers, has also shown that the strong base connections recommended by the current design method may not always lead to a conservative design. A second-phase failure mechanism observed in numerical modelling corresponds with the failure mode shown in one of the previous fire tests. The critical temperature at which run-away collapse occurs may be higher than that at which the roof initially loses its stability, because of re-stabilisation.

In this paper, a new method for estimating critical temperatures of single-span frames in fire, using these two failure mechanisms, is presented. Numerical tests on typical industrial frames are used to calibrate this new method against the current design method.

1 BEHAVIOUR OF SINGLE-STOREY PORTAL FRAMES IN FIRE

As early as 1979 the behaviour of steel portal frames in accidental fires was described in the report of a study [4] of fires in a number of portal frames in the UK. A typical variation of the overturning moment at the column base with time, after a fire is ignited in a pitched-roof portal frame is described in the CONSTRADO design guide [5]. It was believed that the stability of the column was mainly determined by the resistance provided by the column base connections. However, the fire test on a scaled pitched-roof portal frame performed in 1999 [2] showed that the steel columns, connected to their foundations by a fairly flexible connection, could stand almost upright throughout the fire while the rafters snapped through to a inverted shape. This indicates that the strong column bases are not always essential to the stability of an industrial frame under fire conditions.

It has been postulated by O'Meagher *et al.* [6] that unaffected parts of a building can anchor the collapsing parts, provided that the forces developed in the purlins is small and that they have sufficient capacity at high temperatures. The results of a series of parametric studies [2] using the two- and three-dimensional modelling showed that that the initial collapse of a portal frame with semi-rigid bases initially loses stability in a combined mechanism, which differs from the assumption used in the current design method. Further deformation could not be simulated because of the limitations of the static solver.

In a previous paper the behaviour of single-span pitched portal frames was simulated using the recently-developed quasi-static solver [3] in *Vulcan*. This showed that collapse of the frame happens in two phases [7]. It was also found that initial collapse of the rafter is always caused by a plastic hinge mechanism which is based on the frame's initial configuration. If the frame can re-stabilize when the roof is substantially inverted, a second plastic mechanism based on the re-stabilized configuration leads to eventual failure of the whole frame.

2 NEW DESIGN METHOD

A single-span portal frame fails either in the first-phase mechanism when it initially loses stability, or may re-stabilise for a while before collapsing in the second-phase mechanism. The simple method developed by Wong [2] is based on the initial configuration of the frame. Hence, it is capable of explaining the reason why frames initially lose stability in fire, but is not valid for frame collapse in the second mechanism, in which the deformation of the frame is significant. The estimation of the critical temperatures for a two-phase failure mechanism should be based on different initial configurations for each of the two phases.

2.1 Estimation of First-Phase Failure

When the roof of the frame starts to deform downward under the loading and fire temperature, the columns are pushed outward due to the change of geometry and to thermal expansion of the rafters. For a portal frame with frictionless pinned base connections, high rotations can be generated at these bases, caused by either elastic or plastic deformation. These rotations, together with the fire hinges formed at the apex and eaves, can generate a "combined" plastic mechanism. Wong's simple model, as shown in *Fig. 1*, 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 loss of stability at relatively low deflections. According to plastic theory, for the mechanism shown in *Fig. 1*, the fire hinge moments at corners 1 and 2 can be calculated. The ratio of the fire hinge moment to the normal moment capacity is given by the strength reduction factor at the critical temperature, so the critical temperature of this frame can be interpolated from stress-strain curves defined in Eurocode 3 Part 1. 2 [8].

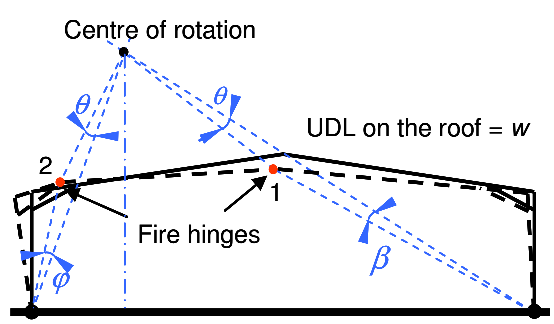

Fig. 1. The model of Wong's simple design method.

2.2 Estimation of Second-Phase Failure

The initial collapse of the roof frame may initiate a "combined" mechanism leading to collapse of the whole frame, or the columns may be pulled back towards the upright position (see shape ABCDE in *Fig. 2*) due to the collapse of the rafters.

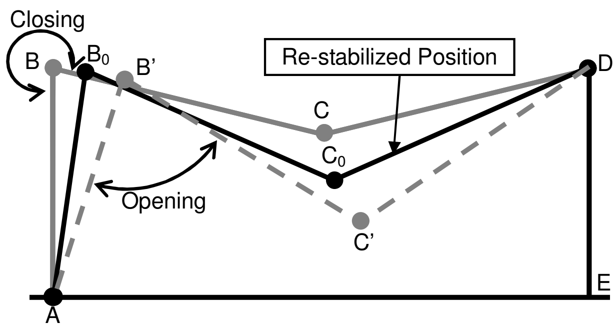

Fig. 2. Illustration of the second phase mechanism.

In the latter case the change of direction of the column's rotation causes elastic unloading of the base moments of the columns, so that the plastic hinges developed at the bases are effectively locked. When the pitched roof deflects further than the position BCD, column AB is pulled inward, so that the base moments of the columns increase again. When the rotation of the column AB is faster than the rotation of the adjacent rafter, the moment on one eave (corner B) starts to reverse, leading to locking of the adjacent plastic hinge. This causes the frame to re-stabilise at this position (Shape AB_0C_0DE in *Fig. 2*), at which the internal angle (AB_0C_0) between column AB_0 and the connected rafter BC_0 stops closing and starts opening. With further increase of the pulling force at the column top caused by 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. 2*), and a new mechanism, referred to as the second-phase failure mechanism, is established which leads to complete collapse of the frame.

The new design method developed here focuses mainly on collapse caused by the second-phase mechanism in fire, and aims to predict the critical temperatures which initiate formation of the second-phase failure mechanism. The method is based on calculating the strength reduction factor of the fire hinge moment according to the work balance within the frame.

Because of the significant deformation of the roof frame before the start of the second failure mechanism, this new model has to identify the re-stabilised position of the frame and its critical position at the start point of the second-phase mechanism. Both the thermal elongation of rafters and degradation of fire hinge moments at elevated temperatures are considered under some temperature assumption. Moreover, because a plastic hinge at one column base is essential to generate a second-phase mechanism, the strength of the column bases is also included in this new method.

When the second-phase mechanism of the frame is established, the elongation of the rafters is significant. This should not be ignored when the work balance is calculated within this system. Estimation of the critical temperature for the second-phase mechanism of the frame is also based on the reduced moment capacity of the rafters. Because the strength reduction factor and the steel elongation at the critical temperature are both unknown, an iterative solution procedure, illustrated by the flow chart in *Fig. 3,* is required.

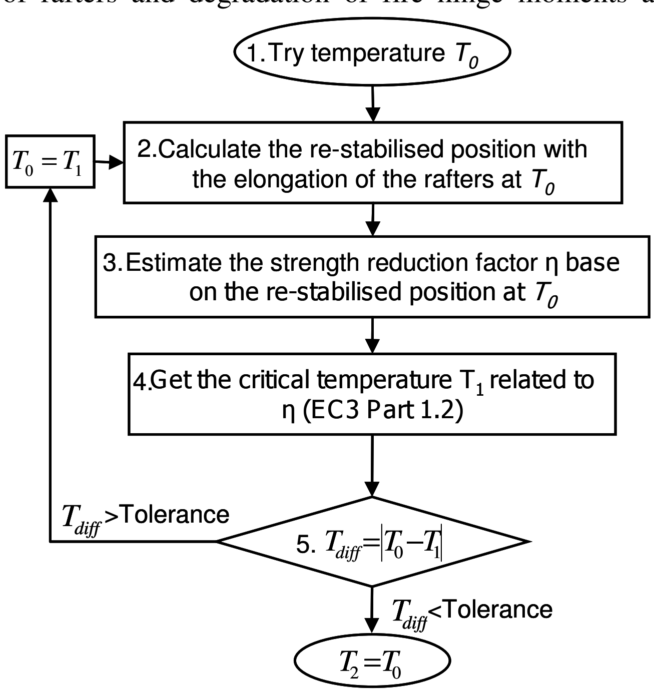

Fig. 3. The procedure for estimating the critical temperature of second phase failure.

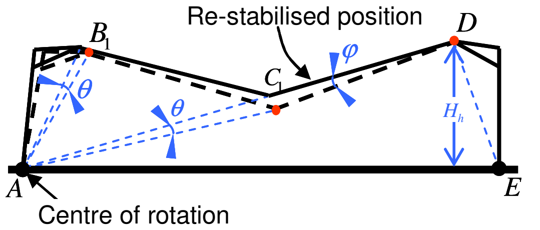

Fig. 4. The model of the second-phase failure

At the beginning of the second-phase calculation, an initial temperature T_0 is assumed, so the re-stabilised position can be estimated on the basis of the geometry of the frame, including the elongation of the rafter, at temperature T_0. The fire hinge moment can also be calculated on the basis of the configuration of the frame at the re-stabilised position (as shown in *Fig. 4*) and the work balance based on plastic theory. The critical temperature T_1 can be obtained from the strength reduction factor given by dividing the fire hinge moment by the moment capacity of the rafter section, and relating this to the corresponding temperature, as defined in Eurocode 3 Part 1.2 [8]. If the difference between T_0 and T_1 is larger than the tolerance required, Steps 2 to 5 as defined in *Fig. 3* are repeated, using the elongated lengths of the rafters at T_1, until T_{diff} is smaller than the tolerance required. The temperature T_1 estimated from the final iteration is the critical temperature of the frame at the beginning of the second-phase mechanism.

3 VALIDATIONS AGAINST NUMERICAL TESTS

In order to validate the new design method a series of comparison between critical failure temperatures predicted using the new design method and those obtained from previous numerical tests [9], have been conducted. Because the two-phase mechanisms are included in this method, critical temperatures for the creation of both the first- and the second-phase mechanisms are compared.

Figs. 5 and *6* compare the critical temperatures predicted by the new design method and numerical analysis results for two typical portal frames. The results presented in *Fig. 5* are for the portal frame designed without haunches but with varying base strength. Results for the other portal frame, which is designed with typical-sized haunches and modelled with different base strengths, are shown in *Fig. 6*.

The first re-stabilised position of the frame is reached when the rafter is deformed into the inverted position and the vertical displacement of the apex is around 5m. The prediction of the new design method is 5m. This confirms that re-stabilisation during the collapse of the

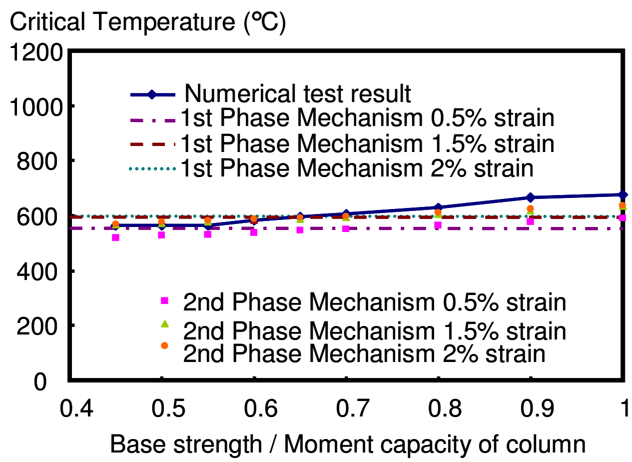

Fig. 5. Portal frames without haunches.

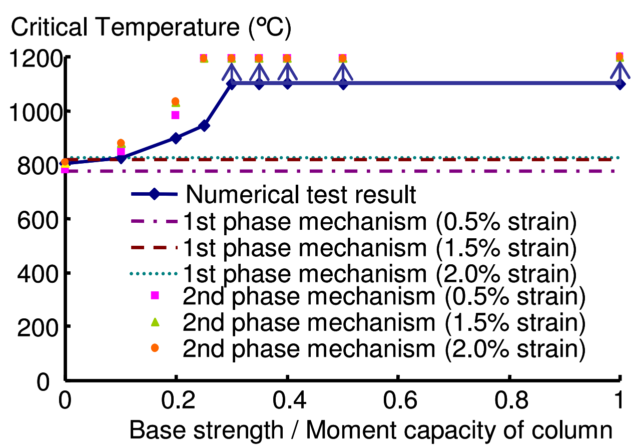

Fig. 6. Portal frames with haunches

portal frame is caused by locking of the plastic hinge near to an eave, which disables the first-phase mechanism. Once the opening of the locked angle exceeds the elastic rotation limit, the frame loses its stability again.

For the frame with haunches, the re-stabilised position predicted by the new design method is about 0.4m lower than for the numerical results. This is because, in the new design method, a re-stabilised position is assumed to occur when, in the first-phase mechanism, the rafter ceases to rotate relative to the column at hinge B (*Fig. 4*) and this hinge consequently locks itself. When the hinge finally begins to rotate again, in the opposite sense to its original rotation, the second-phase mechanism is created and failure occurs. Locking of the plastic hinges developed at one of the column bases could also be encountered when the column is pulled inward and passes its original position, so collapse is arrested until the plastic hinge at the base A is unlocked. When this hinge is mobilised, the frame is capable of continuing to collapse until the fire hinge near the end of the haunch locks itself. This explains the difference between the re-stabilised positions predicted by numerical tests and this new design method. It is worth noting that the main purpose for estimating the re-stabilised position is to determine the critical configuration of the frame on the basis of the principle of work balance. Therefore the final equilibrium position of the frame before it collapses again should be adopted in order to estimate the critical temperature.

From the previous numerical studies, the portal frame would fail in its first-phase mechanism when the ratio between the base strength and moment capacity of the column is lower than 0.55 for the unhaunched case and 0.1 for the haunched case. The reduced "yield stress" at 0.5% steel strain leads to a better prediction of the critical temperature of the frame without haunches in the new design method. As shown in *Fig. 5* the critical temperatures predicted by the new method for the second-phase mechanism according to the strength reduction factors at 1.5% and 2.0% strain show a similar trend to the numerical results.

From *Fig. 6* it is evident that, for a haunched frame with base strength equal to 10% of the moment capacity of the column section, the critical temperatures predicted by the new design method for the first-phase mechanism are very close to the numerical results. However, the numerical analyses give higher limiting temperatures for cases with base strength greater than 20% of the moment capacity of the column.

In the new design method the strength reduction factor obtained from equilibrium on the basis of the second-phase mechanism, becomes negative when the base strength is higher than 30% of the moment capacity of column. This means that the work done by the plastic hinges developed at the bases exceeds the work done by the external forces, and hence the second-phase mechanism can not happen, and the frame will stand in its re-stabilised position under further temperature increases.

4 CONCLUSION

A new design method, extended from Wong's model is presented in this paper. In this method, instead of relying on a single failure mechanism, the two most common failure mechanisms for pitched portal frames under fire conditions are considered to predict the critical temperatures of the frame. The critical temperatures predicted by the new design method on the basis of the first-phase failure mechanism show very good agreement with the numerical results at which a typical frame initially loses stability in fire. Very reasonable predictions about the re-stabilised position and the final collapse temperatures of the frames were achieved using this new design method. It is evident from this study that the initial collapse of the frame due to the first failure mechanism is often a temporary instability, and that after this the frame can experience a second-phase mechanism with a higher critical temperature, which depends on the base strength and the loading conditions of the frame.

When the frame collapses at the beginning of the first-phase mechanism, the inclination of the columns may be relatively small, so this could be a lower bound for the design of portal frames in a fire boundary condition. The re-stabilisation after the initial loss of stability of the frame can be estimated from the critical temperature predicted on the basis of the second-phase mechanism. In this new design method, when the strength reduction factor, calculated from work equilibrium for the second-phase mechanism of the frame, becomes negative it is possible that the portal frame could remain in its re-stabilised state after snap-through of the pitched roof, and may not collapse until a very high temperature is achieved.

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