PREDICTING THE COLLAPSE BEHAVIOUR OF LIGHT STEEL FLOOR TRUSSES IN FIRE

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Abstract

This paper investigates the performance of individual floor trusses of the type used at the World Trade Center using software developed over a number of years to model the behaviour of steel and composite framed structures in fire. A variety of scenarios are considered, including different truss forms, degree of protection, and span length, and load ratio. Issues which are considered include the identification of critical truss members, the interaction between flexural behaviour and catenary action at increasing levels of deflection, and the forces which develop in the connections. Results are generally presented as graphs of increasing deflection with temperature or time, and recommendations are given regarding the use of such structural forms for different fire resistance requirements.

Keywords: Fire, Steel trusses, Collapse, Deflection

1. Introduction

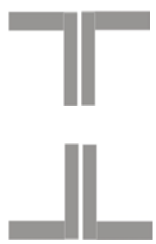
Lightweight steel floor trusses offer a very efficient alternative to conventional beam systems, in relation to both material and ease of construction. This is especially so for longer spans which are becoming increasingly popular as the value of flexible floor space is recognised. Design procedures for such trusses are well developed, and these should include considerations of fire resistance, which, as for other structural steel beam types, can be very important. Indeed, the whole issue of fire resistance has assumed greater significance since the collapse of the twin towers of the World Trade Center.

This paper considers the performance of such trusses in fire using the *Vulcan* software specifically developed for the analysis of steel and composite structures in fire (Huang et al 2000a, 2001). Generalised cross-sections were introduced by Cai et al (2002), and the influence of the concrete slabs included by Huang et al (2000b). Most recently the effects of membrane action in the slabs has been included (Huang et al 2002a, b) and the software extensively validated against test data (Huang et al 2002c) The initial study examines the performance of the floor trusses used at the World Trade Center, where one of the issues was the integrity of the fire protection. This leads into a more generic study in which robust fire protection strategies are examined. These trusses are designed to act compositely with the floor slab, bringing advantages for both the normal ambient temperature design condition, and the fire design case. A target fire resistance of 60 minutes is considered, as this is likely to be the typical specification for a multi-storey building. Whilst the ideal would be a bare, totally unprotected truss, it is recognised that some protection will be necessary. In the UK increasing use is being made of intumescent paint applied off-site as this reduces site work, and improves construction efficiency. For greatest advantage, the paint thickness needs to be limited to a single coat, and in the current study, protected parts of the truss are assumed to be treated in this way.

The purpose of the paper is not to explain the mechanisms by which the World Trade Center towers collapsed, but rather to consider the performance of such floor systems in a more general manner, with the intention of providing designers and control authorities with a greater understanding of the issues involved.

2. Analysis of the truss types used at the World Trade Center

The World Trade Center trusses were non-composite with a span of 60ft (18.3m), and the form of the truss is illustrated in section in figure 1. For the purpose of this analysis, the trusses were assumed to have no applied fire protection, and subject to a standard fire. The development of temperatures in the different parts of the truss was determined using the approach outlined in EC3: Part 1.2 and the results are shown in figure 2. These temperature profiles were then used to perform the structural modelling using *Vulcan* to analyse the behaviour of the trusses during the progression of the fire under a constant load of 5.46kN/m^2. The results are shown in figure 3. This shows a steady increase in deflection with temperature, accelerating a little above 500°C, and reaching 900mm (span/20) at approximately 670°C. This corresponds to a time of just less than 20 minutes, suggesting that this form of truss will invariably require some applied. fire protection.



Member	Section	Steel grade
Top chord	2x1.5x0.25	A50 (345N/mm^2)
First diagonal	1.14 ϕ bar	A50 (345N/mm^2)
Other diagonals	1.09 ϕ bar	A46 (317N/mm^2)
Bottom chord	3x2x0.37	A36 (250N/mm^2)

Figure 1. Cross-sectional details of the World Trade Center trusses used in the *Vulcan* analysis,

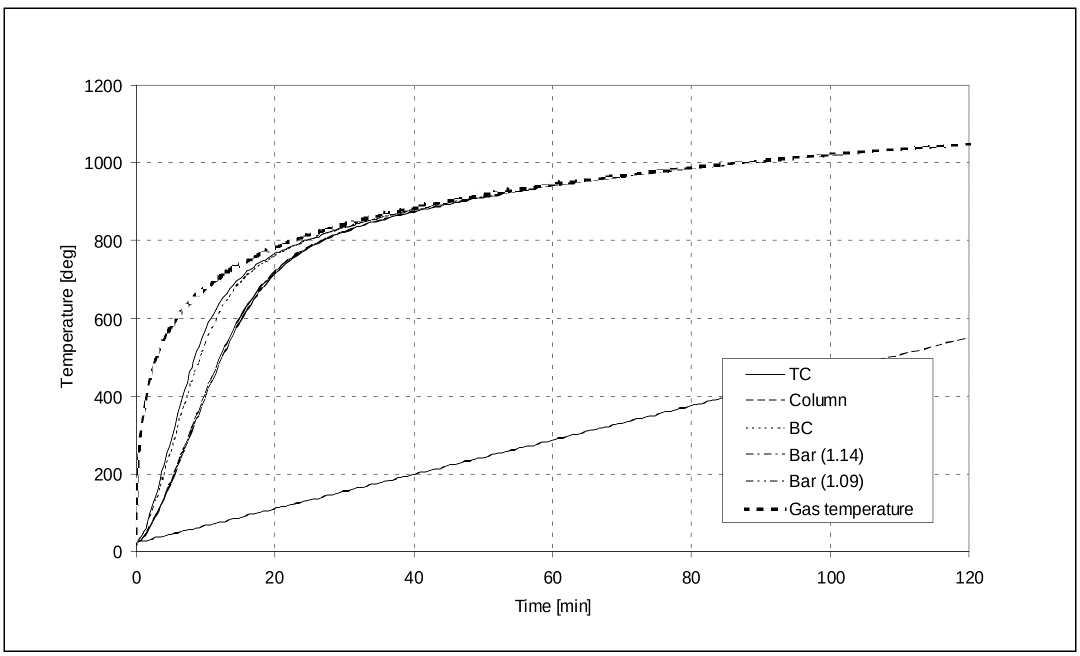


Figure 2. Variation of temperature with time for different structural elements according to EC3: Part 1.2

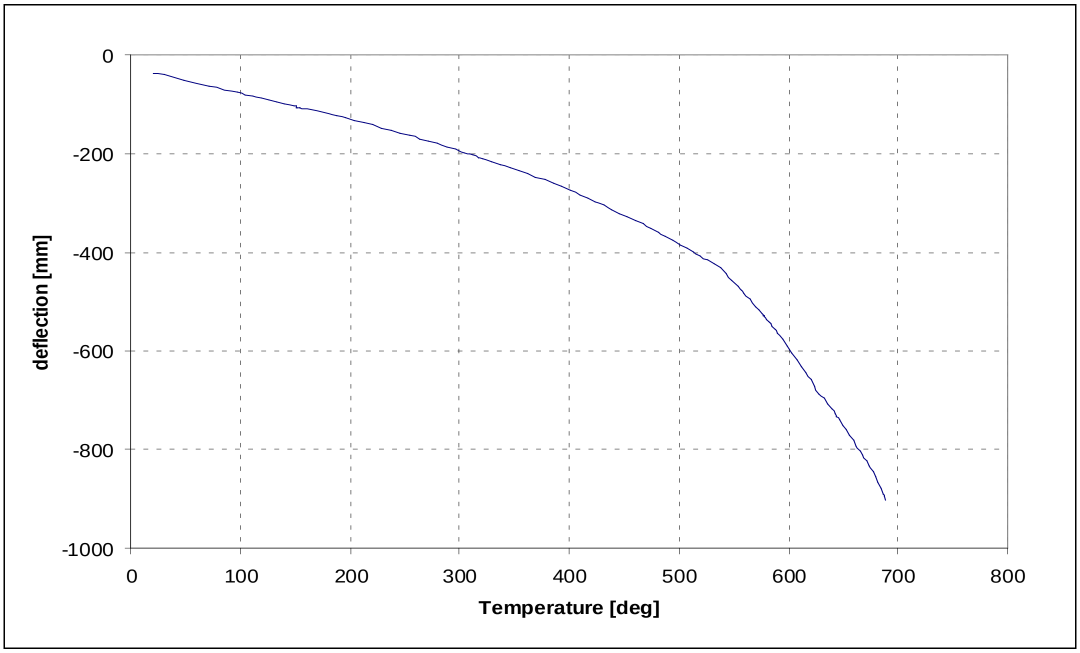


Figure 3. Results of *Vulcan* analysis showing increasing deflection with temperature for the World Trade Center trusses

3. Generic studies

There are clearly advantages in systems which do not require applied fire protection – in relation to both economy and robustness. A modified form of floor truss using more substantial diagonal bracing members than the World Trade Center trusses has been used for the purpose of this study. The truss, which is 750mm deep, is assumed to be simply supported and acting compositely with a 130mm thick composite deck floor (comprising a rib depth of 60mm and a topping of 70mm). A range of spans is considered, but a typical spacing between trusses of 2.5m is used. The truss details have been determined by a conventional design process for ambient temperature using standard Metsec profiles in S350 steel. Concrete grade is C30, and A142 anti-crack mesh reinforcement is included in the slab. Additional reinforcement is also included in the slab immediately over the truss to enhance the catenary action at very high deflections. Details of the floor truss arrangement are shown in figure 4.

In modelling the truss, the top and bottom chord members are assumed to be continuous, but the diagonal bracing members are assumed to be pinned. All loads are applied at the node points of the truss.

The high temperature properties of the steel and concrete are calculated in accordance with the recommendations of EC3: Part 1.2 and EC4: Part 1.2. Temperatures in the individual truss elements were calculated using the incremental method described in EC4: Part 1.2 accounting for the section factor. The corresponding temperatures in the slab were estimated from experimental data.

Previous studies have shown that, if all members are unprotected, the truss can only survive for relatively short fire durations, confirming the indications from the studies on the World Trade Center trusses. Detailed examination of the truss behaviour identified the diagonal bracing members in compression adjacent to the supports to be the critical elements, failing by buckling and precipitating collapse of the complete truss. These studies have therefore assumed some applied protection to these critical members, using intumescent paint. For those members protected in this way, a two stage temperature development has been assumed, the temperature rising to 200°C at the same rate as an unprotected member, and then increasing linearly until it reaches the design temperature of 650°C at 60 minutes. The temperature development for different parts of the structure is illustrated in figure 5.

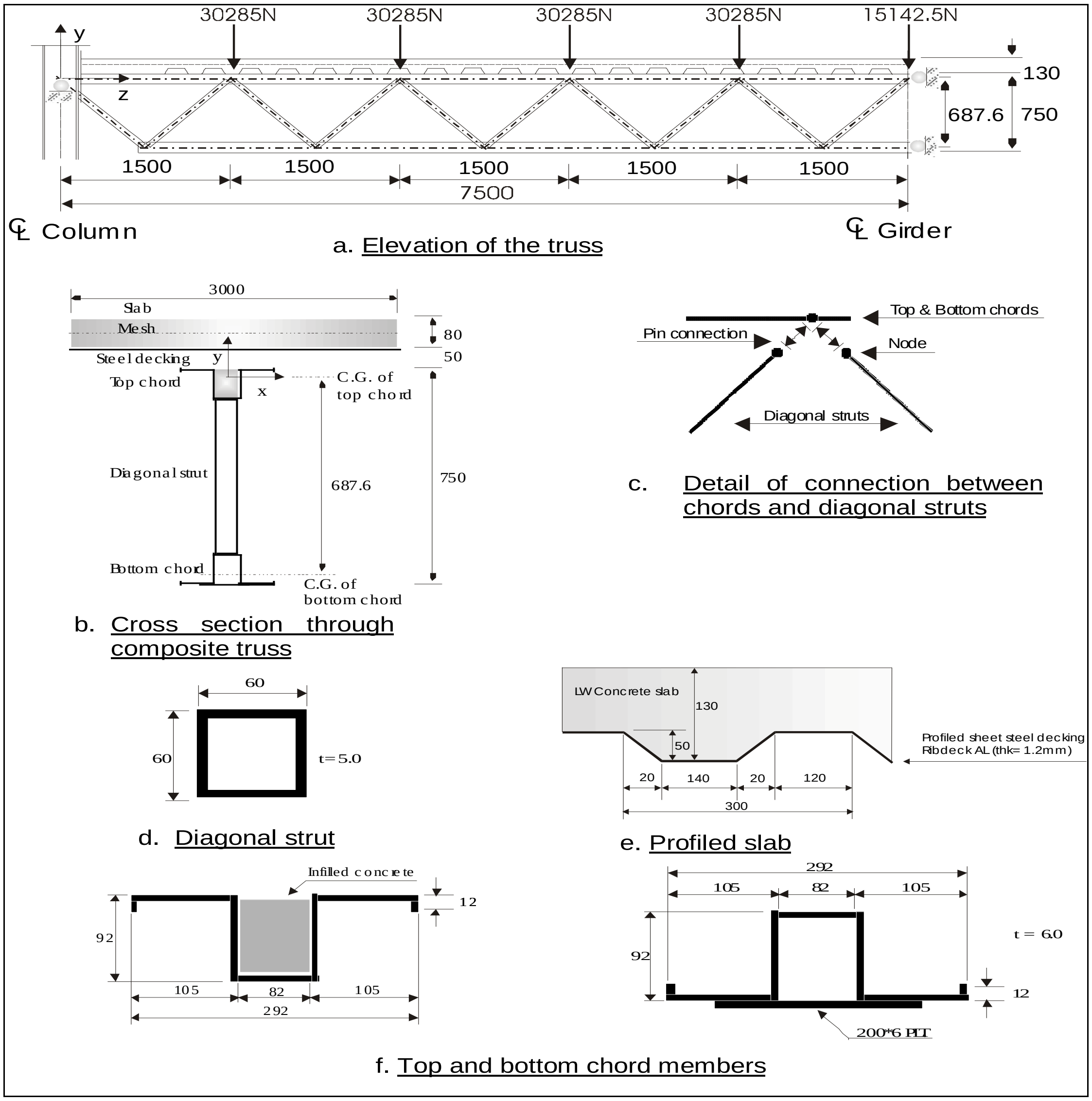


Figure 4. Details of truss used in generic studies

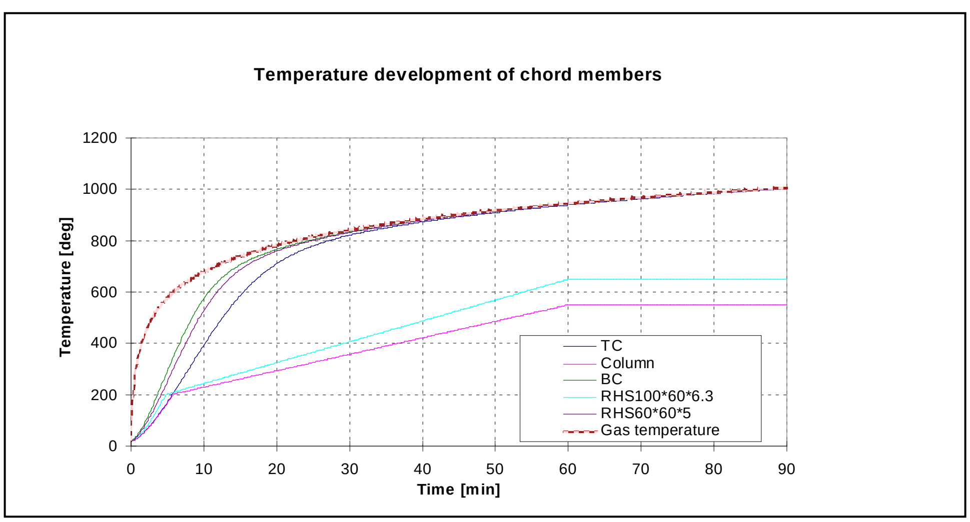


Figure 5. Development of temperature in different structural elements of the generic truss.

3.1 The effect of slab reinforcement on the performance of composite trusses

It is relatively easy to incorporate additional reinforcing bars within the slab immediately over the truss. This will potentially increase the strength of the truss, particularly at high deflections when catenary action becomes more important. In order to study this, the standard truss has been analysed, but with varying degrees of additional reinforcement. The results are shown in figure 6. This demonstrates that two 32mm bars are required in order to achieve 60 minutes fire resistance, and that smaller amounts of reinforcement have relatively little effect. Unfortunately, even with this arrangement, the deflections at 60 minutes greatly exceed what might be regarded as acceptable, even under fire conditions. Further studies have therefore been conducted to examine the effect of end fixity where the truss connects to the column, and how trusses might perform over shorter spans.

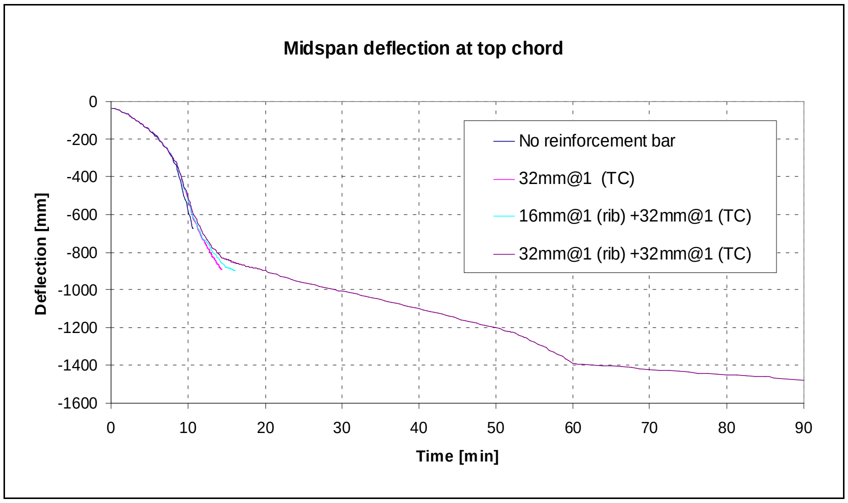


Figure 6. Deflection profile for increasing exposure to the standard fire for the generic truss with various slab reinforcing details

3.2 *The influence of end fixity at truss supports*

It is recognised that even simple connections are able to transmit some bending, and in the case of floor trusses, an end fixity of 10% of full fixity has been suggested. Assuming this degree of restraint, corresponding to an end moment of 10% of the fixed end moment, a rotational stiffness, K_s (= 5.5×10^9Nmm/rad), was calculated by using the moment-area theorem as follows:

$$K_S = \frac{10\%\_of\_M_{fixed}}{\theta} = \left(\frac{wl^2}{12} \times \frac{1}{10}\right) / \theta$$

in which θ

$$= \frac{1}{2EI}\left(-\frac{2}{3} \times l \times \frac{wl^2}{8} + \frac{1}{10} \times \frac{wl^2}{12} \times l\right)$$

This was then included in the structural arrangement to model the standard composite truss. This resulted in buckling of the second compressive diagonal bracing member, and so it was also increased in size to that of the end diagonal and the truss reanalysed. The effects of this on mid-span deflection are compared in figure 7 with the standard result (excluding the additional slab reinforcement).

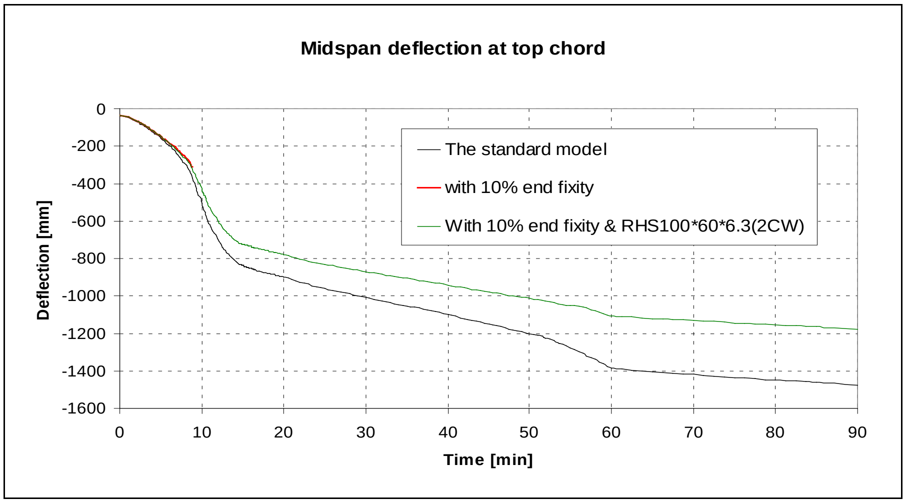


Figure 7. Deflection profile for increasing exposure to the standard fire for the generic truss with end fixity

Clearly there is some reduction in deflection as a result of the end fixity as would be expected. However, this is unlikely to be sufficient to achieve acceptable performance in a fire, with deflections of approximately 1100mm (~span/14) at 60 minutes.

3.3 *The effect of span length and load ratio*

Although floor trusses have particular advantages for longer spans, they may still be economically attractive for shorter spans, for which deflections are likely to be reduced. As deflection seems to be the limiting factor in relation to the fire performance of the trusses discussed above, shorter span trusses have also been studied. Span lengths of 12.0m and 9.0m were considered, using the same basic arrangement for the standard model (15.0m span). The same basic truss configuration and geometry was maintained simply by removing pairs of internal bracing members. In addition, changes in design rules indicate that reduced levels of imposed loads may be allowable when considering the fire limit state. A partial load factor of 0.5 has been proposed, compared with a current value of 0.8. The effects of shorter spans and lower load factor are shown in figure 8. The corresponding deflection limit of span/20 for each case is also indicated.

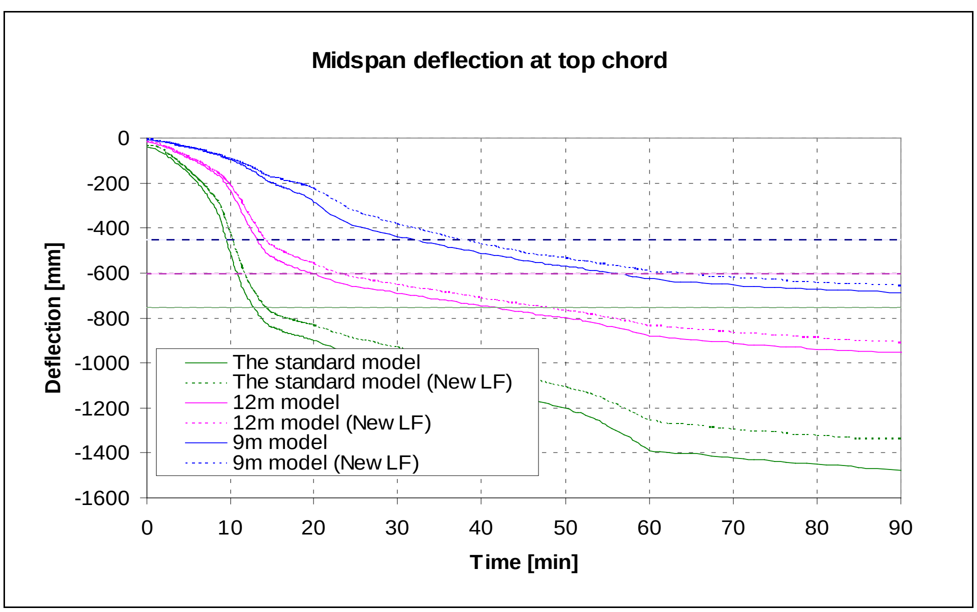


Figure 8. Deflection profile for increasing exposure to the standard fire for the generic truss with various spans and for different load factors

The results show that deflections are indeed reduced for shorter spans and load factors as would be expected. However, at 60 minutes these are still somewhat greater than span/20, the traditional failure criterion for beams in fire.

4. Conclusions

Floor truss systems provide an efficient alternative to conventional beam systems, but fire resistance requirements are an important design consideration. The preliminary results presented here suggest that it may not be practical to detail such trusses to avoid applied fire protection altogether, particularly for longer span conditions. However, the introduction of additional slab reinforcement and partial protection applied to critical parts of the truss may be sufficient to achieve the necessary fire resistance.

The use of advanced analysis using software such as *Vulcan* enables the details to be refined to achieve an optimum design. This will, in turn, enable guidance to be given regarding the extent and nature of fire protection, and in this context work is progressing to examine how partial protection of the bottom chord might help reduce deflections.

4. Acknowledgements

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