The Behaviors of H-section Steel Beam in Fire

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ABSTRACT

In this paper, the behaviors of 14 H-section steel beams in fire are investigated. The test is carried out by use of the self-made fire-test furnace. The lengths of beams are 4200 mm and 3600 mm. The temperature field and deflection of beams in fire are measured. The effect of connection, boundary condition and axial restraint on the behaviors of steel beams is also studied. The test indicates that local buckling and lateral torsional buckling of H-section steel beams occur in fire. The behaviors of H-section steel beams in fire are influenced by connection and axial restraint. When the beams undergo a large deflection, compression force then quickly changes into tension force, which supports the beam and reduces further deflection, and the catenary action takes place. The test results in this paper may be used to guide the design of this type of steel beams in fire.

KEYWORDS: fire, H-section, steel beam, temperature field, deflection, experimental tests

INTRODUCTION

The behaviors of steel beams have been analyzed numerical [1-3]. Some test results are available in the literature [4] and will be considered in the comparison. To avoid possible bias in the conclusions, it is desirable to obtain test results as wide as possible, in relation to the total number of tests as well as the number of different independent sources. As it was not possible to find all the necessary experimental cases, it was decided to perform the new series of experimental full-scale tests.

EXPERIMENTAL EQUIPMENTS AND INSTRUMENTATION

The test program was intended primarily to assess the benefits of structural continuity in fire, on the basis of results of a series of fire tests which include typical beam-to-column connections and different boundary conditions. The beam specimen was subjected to a constant vertical load, and the furnace (Fig. 1) was programmed to follow the temperature-time curve as in Fig. 2. Six thermocouples of K type were installed around the furnace to measure the fire temperatures. Some 21 other K-type thermocouples were installed on the specimen beam to record the temperature distribution (Fig. 3).

The behaviors of the beams were assessed in terms of the moment and thrust resisted by the connections and the mid-span deflection of the beam. The moments and thrusts transmitted by the connections to the columns were determined by measuring the horizontal reaction forces at the top and bottom of the columns using calibrated-pin load cells. Displacement transducers were placed near to the ends of the beam and on the column to measure rotations. Figures 4 and 5 show the locations of some basic instruments on the test beam. The loading was applied manually at room temperature. It was then maintained at the same level during the fire test.

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Fig. 1. Test furnace.

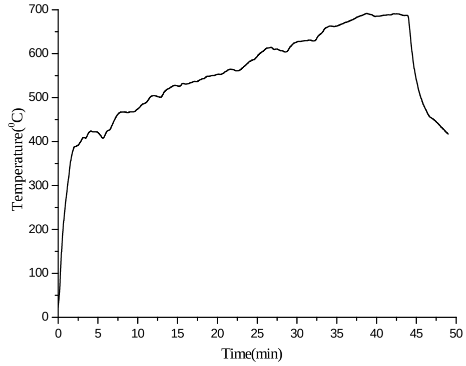


Fig. 2. Temperature-time curve.

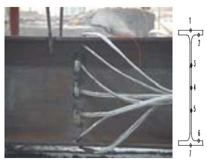


Fig. 3. Thermocouple distribution.

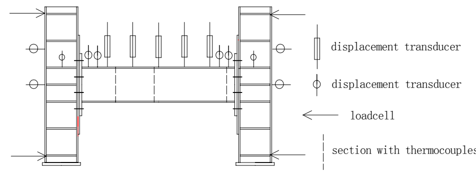


Fig. 4. Instrumentation in the test assembly.

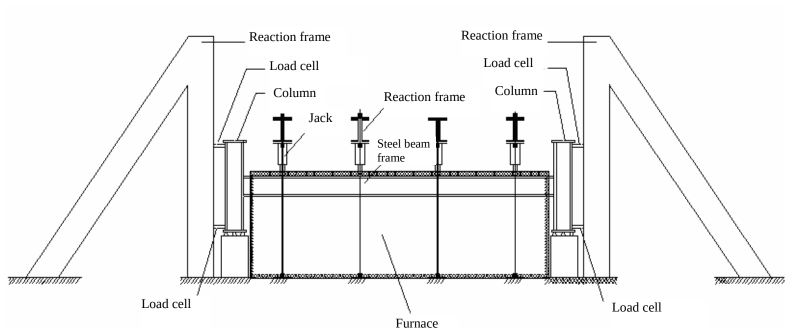
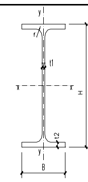


Fig. 5. Test set-up diagram.

EXPERIMENTAL PROGRAM

The experiment included 14 fire tests of steel beams. In order to simulate the heat-sink effect of the concrete slab, the top flange is wrapped with ceramic fiber blanket. Table 1 lists the principal parameters of the 14 tests. Details of steel beams can be seen in Fig. 6. The load ratio is defined as the ratio of the applied load at fire limit state to the load-carrying capacity as a simply–supported beam at room temperature.

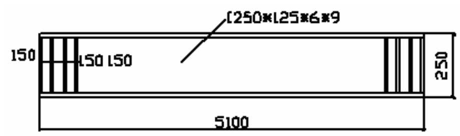
Table 1. Numbering schedule and details of specimens.

Specimen number	**Boundary condition**	**Span (mm)**	**Applied load(KN)**	**The load ratio(R)**	**Specification of H-section steel**
S-1	both ends hinged	4200	10.5	0.31	 H×B=250×125 t_1=6 t_2=8 r=14 f_y=330 N/mm^2 I_x=4080 cm^4 I_y=294 cm^4 A=37.87 cm^2
S-2			17.5	0.52	
SG-1	one end hinged, one end	4200	10.5	0.31	
SG-2			17.5	0.52	
G-1	both ends fixed	4200	16	0.48	
G-2			26	0.78	
R-1	web-cleat connection	4200	10.5	0.31	
R-2			17.5	0.52	
R-3		3600	12.3	0.31	
R-4			20.5	0.52	
SR-1	end-plate connection	4200	16	0.48	
SR-2			26	0.78	
SR-3		3600	18.5	0.48	
SR-4			30.5	0.78	

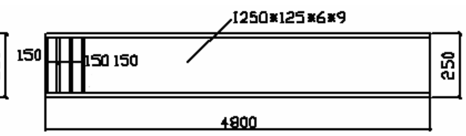
EXPERIMENTAL RESULTS

Temperature Distribution

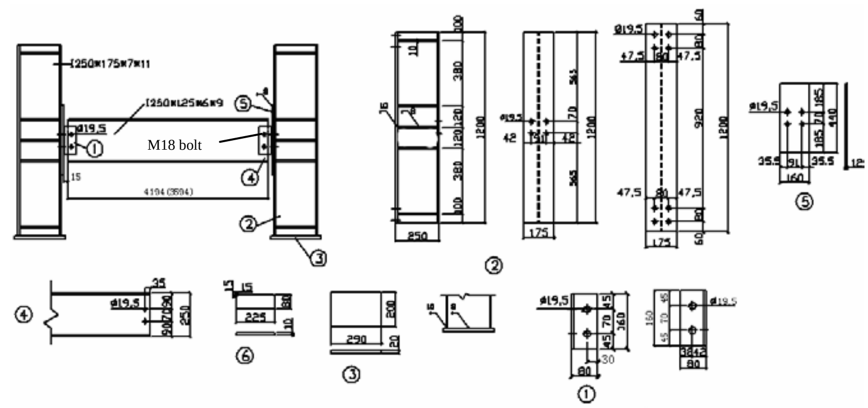
Figure 7 shows typical variations against time of the temperatures of the furnace atmosphere, during the heating and cooling phases. Due to the fire protection around the top flange its temperature initially rose much more slowly than the rest of the section. The difference between 1 and 7 increased to almost 135°C after about 7min. As the rate of rise in fire temperature reduced, the rate of rise of temperatures in the bottom flange as a result of the lower radiation reduced. However, conduction of heat into the top flange continued. The temperature difference between the top flange and the rest of the section was thereafter reduced to as little as 80°C when the bottom temperature exceeded 610°C. Indeed, the temperatures in the top flange continued to rise further at a low rate after the fire was switched off, and its temperatures were reduced more slowly than other parts of the section.



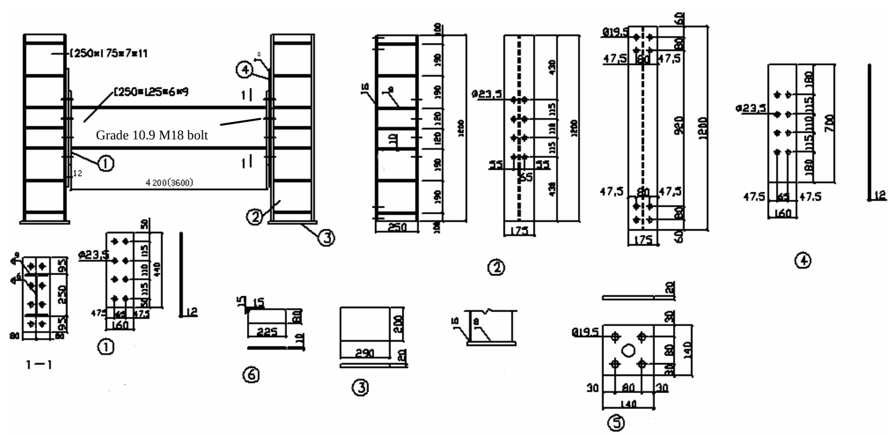
G-1



SG-1



R-1



SR-1

Fig. 6. Details of steel beams.

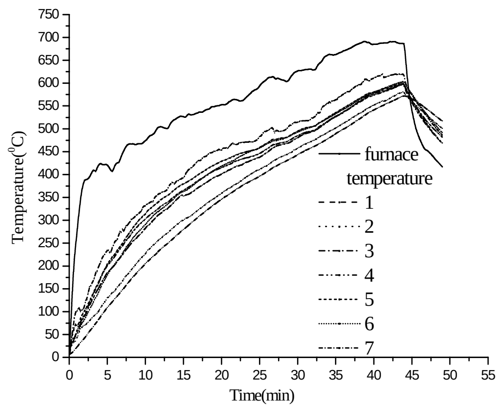


Fig. 7. Typical temperature distribution.

Deflection Characteristics

Figure 8 shows the temperature-deflection curves. Most of them were able to sustain the load without excessive deflection before limit temperature. There was the reversal of thermal bowing temperature during the heating phases, leading to a slight reduction of deflection. Further rise in temperature led to a progressive run-away of beam deflection as loss of stiffness and strength accelerated.

The lateral torsional buckling was found after completion of the tests (Fig. 9).

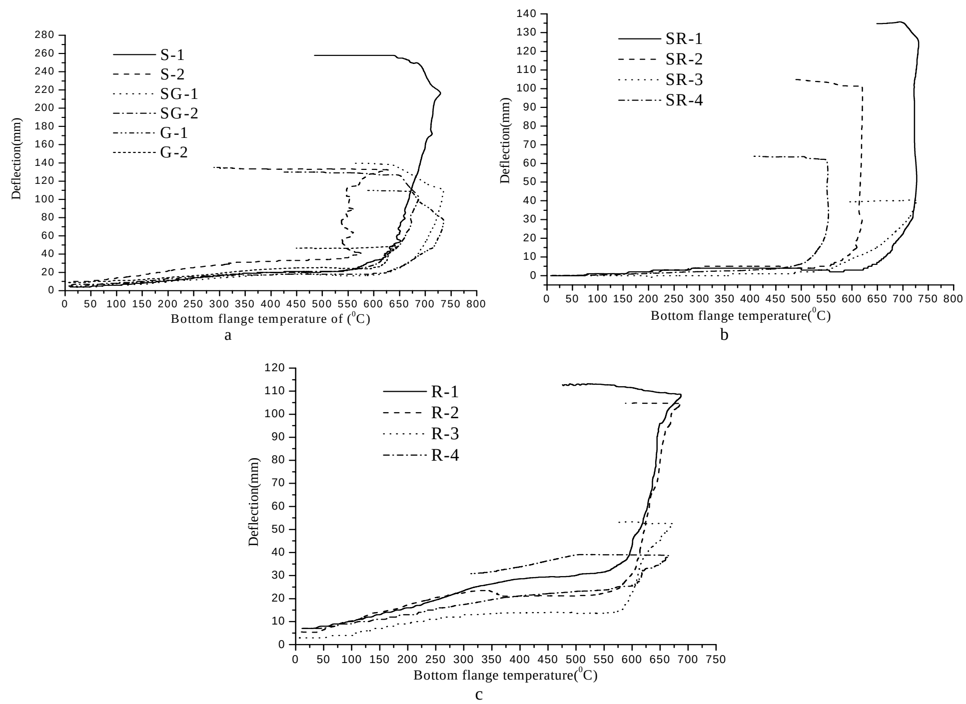


Fig. 8. Temperature-deflection curves.

S-1 SG-1

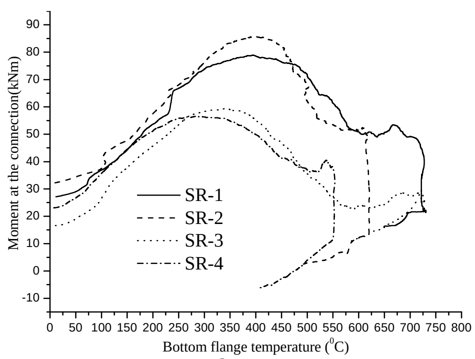
R-2 SR-1

Fig. 9. Steel beam after fire.

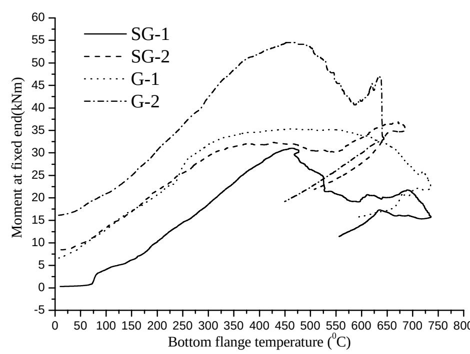
Effect of Connections

End-plate Connections and Fixed Ends

The top flanges of the beams were protected from direct heating. The increase of temperature at other parts of the section leads to a thermal bowing towards the fire, which induces an increase in moment at the connections and moment at fixed end. The connections, including the bolts and the endplates, were fire-protected by ceramic blanket. Fig. 10 shows the variation with temperature of the connection moment (the fixed-end moment). It has been found that the bending moments were particular sensitive to the actual temperature distribution at the sections near to the connection. Despite the large variation, the general trends are very clear. The beams have similar rates of increase in connection moment and fixed-end moment, which are irrespective of loading level and span. As temperatures rose further, the bending moments started to decline. The reason affecting the point at which the bending moment started to drop is the reversal of thermal bowing temperature.



a



b

Fig. 10. Temperature- moment curves.

In these tests, there was a phenomenon of a sudden reduction of moment. The reason is that the high stress due to the combinations of connection moment and axial thrust leads to early and progressive material softening in the vicinity of the connections. When plastic hinge occurs, it indicates local buckling takes place (Fig. 11). Because the fire protection of the connection leads to a lower temperature in this zone, such plastic hinges were not formed at the connections.

G-1 SR-1

Fig. 11. Local flange buckling.

Web-cleat Connections

Though normally regarded as a pinned joint, the web-cleat connection inevitably resisted a minimal amount of moment. Figure 12 shows the moment-temperature curve for web-cleat connections under various conditions. As temperature increased, the connection moment increased subsequently, but remained fairly low. At 600°C, the connection moment increased suddenly. The reason is that the gap between the beam and the column had closed.

There was no sign of local buckling in the lower flanges of these beams near to their ends. However, there were clear indications that the bolt holes in the web of the beams had been elongated (Fig. 13). This is because of the lower value of moment capacity of such connections but large bearing forces.

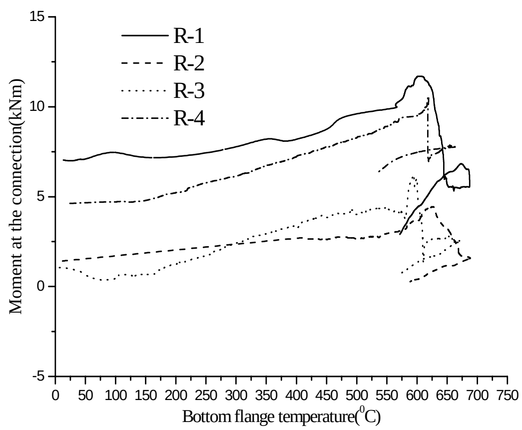


Fig. 12. Temperature-connection moment curves. Fig. 13. Local photo.

Effect of Axial Restraints

Figure 14 shows the axial force-temperature curves under various conditions. Axial compression force developed when the thermal expansion in the heated steel beam was resisted by the column. As the beam temperature increased further, the beams underwent a large deflection, and compression force then quickly changed into tension force, which supported the beam and reduced further deflection and catenary action took place.

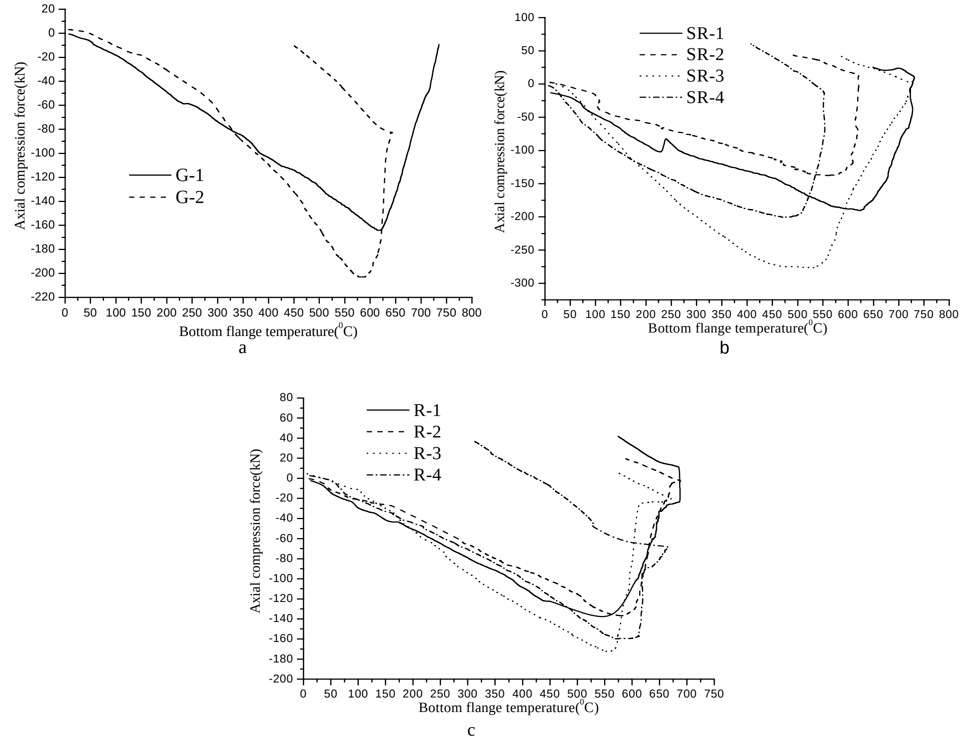


Fig. 14. Temperature-axial force curves.

CONCLUSIONS

In this paper, the behaviors of 14 tests of H-section steel beams in fire are investigated. All the beams are full scale with 4200 mm and 3600 mm length. In order to simulate the heat-sink effect of the concrete slab, the top flange is wrapped with ceramic fiber blanket. The test results are following:

1. When H-section steel beams are in fire, lateral torsional buckling takes place.

2. As the thermal bowing changes, the mid-span deflection, connection moment and moment at fixed end change.

3. The combinations of connection moment (fixed-end moment) and axial thrust lead to early and progressive material softening in the vicinity of the end-plate connections (the fixed ends), and plastic hinge occurs.

4. When the beam undergoes a large deflection, compression force then quickly changes into tension force, which supports the beam and reduce further deflection, it indicates catenary action takes place.

ACKNOWLEDGMENT

This research was financially supported by the China NKBRSF project (No. 2001CB409603) and the China International Joint research project of NSFC (No.50320120156). The authors deeply appreciate the support.

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