Journal of Constructional Steel Research 65 (2009) 1896–1903

Contents lists available at ScienceDirect

Journal of Constructional Steel Research

journal homepage: www.elsevier.com/locate/jcsr

Horizontal forces in steel structures tested in fire

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ARTICLE INFO

Article history:
Received 13 November 2007
Accepted 21 April 2009

Keywords:
Structural engineering
Steel structures
Fire test
Full-scale test
Fire design
Eurocodes
Connections
Robustness
Horizontal forces

ABSTRACT

Fire tests carried out on the eight-storey steel framed building at the Building Research Establishment's Cardington laboratory have shown that the connections are subject to large axial force. These forces are the result of thermal movements of the structure during heating and cooling and in some cases can result in failure of the structure. This paper describes a fire test carried out on the steel frame at Cardington on 16th January 2003 and a fire test carried out on a structure in Ostrava on 16th June 2006. In both cases the tests were designed to measure the forces generated in the connections.

1. Introduction

Full-scale fire tests carried out on the eight-storey steel framed building at the Building Research Establishment's Cardington facility have shown that the connections are subject to high axial forces as a result of thermal movements during heating and cooling, see [1]. Current design methods do not check the behaviour of the connections under these conditions which can be critical, particularly in the cooling phase of the fire. Determining these forces is not easy as they are dependent on the form of the structure, the boundary conditions, the time–temperature curve, the thermal movements and local failures during heating and the behaviour of the deformed structure during the cooling phase of the fire. Observations from the fire tests at Cardington also showed that a partially protected composite flooring system deforms to such an extent that it supports the applied load in a combination of bending and catenary actions transferring significant axial forces to the supporting connections.

Although the axial capacity of a connection is not routinely checked under the fire situation the axial capacity of a connection is checked as a way of providing a structure with adequate robustness against disproportionate collapse in the event of an accidental action such as a gas explosion or impact from a vehicle, see [2].

Early work on robustness required the connections in a steel framed structure to have at least two M 16 bolts in tension to ensure structural integrity., This was replaced by a estimation of the tie forces in the connections based on catenary behaviour of the supported beams. see [3–5]. There are many similarities between the catenary action of a flooring system under fire and that under accidental actions. It is postulated that the design of the connections for axial tying capacity will allow the connections to resist the axial forces generated as a result of the floor going into catenary under fire. If this could be proven no further checks at elevated temperature would be required. The aim of this paper is to determine the axial forces that connections are subject to under heating and cooling and to compare these with the tying forces used in design for progressive collapse.

Annex A of EN 1991-1-7: 2006 [6], gives the following expression for calculating tie forces:

$$T_i = \min[k(g_k + \psi q_k)sL; 75\ \text{kN}] \tag{1}$$

where

- k is the transformation factor; for internal ties $k = 0.8$; for perimeter ties $k = 0.4$,
- g_k is the characteristic value of permanent action,
- ψ is the combination factor according to the accidental load combination,
- q_k is the characteristic value of variable action,
- s is the spacing of the ties, and
- L is the span of the tie.

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0143-974X/$ – see front matter
doi:10.1016/j.jcsr.2009.04.020

Nomenclature

f_{ub}	is the ultimate strength of the bolt
f_y	is the yield stress at ambient temperature
g_k	is the characteristic value of permanent action
k	is the transformation factor; $k = 0.8$ for internal ties; $k = 0.4$ for perimeter ties
$k_{E,\theta}$	is the reduction factor for the slope of the linear elastic range at the steel temperature
$k_{y,\theta}$	is the reduction factor for the yield stress at the temperature of the steel
s	is the spacing of the ties
q_k	is the characteristic value of the variable action
A_s	is the tensile stress area of the bolt
L	is the span of the tie
E	is the elastic modulus of the steel
$F_{t,b}$	is the design tension resistance of the bolt
$F_{t,2}$	is the horizontal force to the column on the second floor
$F_{t,3}$	is the horizontal force to the column on the third floor
$F_{t,4}$	is the horizontal force to the column on the fourth floor
T_i	is the tie force
W_y	is the section modulus to y-axis, major axis
W_z	is the section modulus to z-axis, minor axis
γ_{M2}	is the partial safety factor for the bolt
ψ	is the combination factor according to the accidental load combination
ε_θ	is the strain at elevated temperature

2. Seventh large-scale fire test on a steel frame in Cardington

A structural integrity fire test was carried out on the eight-storey steel framed building at Cardington on 16 January 2003, see [1]. The main purpose of this test was to collect data on the behaviour of typical beam-to-column and beam-to-beam connections subjected to a natural fire, see [7]. The test was carried out in a compartment on the fourth floor enclosing a plan area of 11×7 m, as shown in Fig. 1. The internal walls of the compartment were made of three layers of plasterboard (15 mm + 12.5 mm + 15 mm) with a thermal conductivity of 0.19–0.24 W m^{-1} K^{-1}. The external wall was a 0.9 m brick window sill and 1.95 m plasterboard wall. A 1.27 m high and 5.7 m wide opening simulated an open window and created ventilation for the compartment. The size of the opening was designed to produce a fire with a temperature exceeding 1200 °C and a duration of 60 min.

The steel structure within the compartment consisted of four columns (internal columns were 305 × 305 × 198UC section, edge columns 305 × 305 × 137UC section, steel grade S355), two primary beams (336 × 171 × 51UB section, steel grade S350), two secondary beams (305 × 165 × 40UB section, steel grade S275) and edge beam (356 × 171 × 51UB section), see [1]. Flexible end plates were used for beam-to-column connections and fin plates for the beam-to-beam connections. In both cases, the plates were made from steel grade S275 and M20 bolts, grade 8.8, were used. To prevent the collapse of the structure, the columns were fire protected by a 20 mm thick layer of Cafco300 vermiculite-cement spray with a thermal conductivity of 0.078 W m^{-1}K^{-1}. In addition, protection was also applied to joints on external columns and parts of the primary beams (approximately at a distance 1.0 m from the joints), see Fig. 2.

A lightweight concrete slab cast on the profiled metal decking was supported on the primary and secondary beams. 19 mm diameter shear studs were used on all the beams.

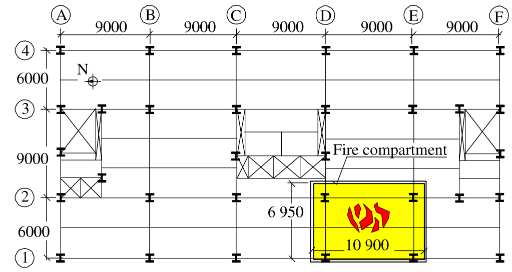

Fig. 1. The position of the fire compartment of the seventh large-scale fire test on the plan of the Cardington frame.

The applied load was applied using sandbags distributed over an area of 18 m by 10.5 m on the floor above the fire compartment. This load represented the permanent action (including floor layers and partition walls) and 56% of the variable action. Wooden cribs with a moisture content of 14% provided a fire load of 40 kg/m^2. According to analytical and finite element simulations, failure of the concrete slab was expected during the fire test.

Thermocouples, strain gauges and displacement transducers were used to measure various data during the test. A total of 133 thermocouples monitored the temperature of the connections and beams within the compartment, the temperature of the concrete slab and the gas temperature within the compartment. Additional 14 thermocouples were used to measure the temperature of the columns, see Fig. 3. Nine high-temperature strain gauges were used to measure the strains in the unprotected fin-plate and end-plate–minor-axis joints. A total of 47 ambient-temperature strain gauges were attached to the protected columns and to the concrete slab. See Fig. 4 for the location of the strain gauges. The vertical deformations of the concrete slab were measured by 25 displacement transducers installed on the fifth floor. Additional 12 transducers measured the horizontal movement of the columns and the slab. Ten video cameras recorded the fire and smoke development and the deformations, and two thermo-imaging cameras were used for measuring the temperature distribution in the steel elements.

3. Horizontal forces measured during the Cardington test

High-temperature strain gauges were attached to the beams next to the connections to measure the strains induced by the fire. These strains were used to determine the axial forces in the connections during the fire. The high-temperature strain gauges are capable of measuring strains up to a temperature of 1200 °C. The stress σ_θ at the elevated temperature was derived from the measured strain using modulus of elasticity reduced for the steel temperature ($E_{a,\theta} = k_{E,\theta}\, E$). The corresponding steel temperature was recorded by the thermocouple attached to the structure near the strain gauges. The calculations are given in Tables 1 and 2. The stress is limited by yielding of the steel. The resulting stress is calculated as

$$\sigma_\theta = \min(k_{E,\theta} E\varepsilon;\ k_{y,\theta} f_y) \tag{2}$$

where

$k_{E,\theta}$	is the reduction factor for the slope of the linear elastic range at the corresponding steel temperature, see [8]
E	is the elastic modulus of steel
ε	is the measured strain
$k_{y,\theta}$	is the reduction factor for the yield stress at the corresponding steel temperature, see [8]
f_y	is the yield stress at ambient temperature, 396 MPa based on coupon tests of beams for Cardington frame [7]
θ_a	is the steel temperature measured near the strain gauge.