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Behavior of prestressed stayed steel columns under fire conditions

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Abstract

Prestressed stayed steel columns experience loss of strength and stiffness when exposed to fire conditions. This paper presents results from experimental studies on the behavior of prestressed stayed circular steel columns under fire conditions. Two full scale prestressed stayed steel columns were tested by subjecting the columns to simultaneous gravity (mechanical) loading and fire conditions. In these fire tests, the varied parameters include load level and level of prestressing. Cross sectional temperatures, axial deformations, as well as fire resistance during the fire tests were recorded and measured. The results indicate that prestressed stayed steel columns undergo various failures modes under different combinations of load and prestress ratios. Specifically, load level significantly influence the fire response of prestressed stayed steel columns with higher load level leading to higher contraction and lower fire resistance.

Keywords: Prestressed stayed columns; Fire tests; Fire resistance; Pretension; Failure

1.0 Introduction

Prestressed stayed steel column refers to a special type of steel column whose ends are held through a set of tension cables. A typical prestressed stayed steel column comprises of a core steel column, two stayed cables attached to each end of the core column and two cross-arms to provide lateral support at mid-height of the column (as shown in Fig. 1). In this type of column, the presence of

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stay and cross-arms offers lateral restraint at mid-height of column, thus decreasing effective length of column which can result in higher load carrying capacity.

Due to the advantageous increase in sectional capacity, aesthetic architectural appearance and ease of installation, prestressed stayed steel columns are increasingly used in high-rise buildings for supporting glass panels (Belenia 1977, Smith 1985, Zhong 1986., Jemah and Williams 1990). Figure 1 illustrates a recent application of prestressed stayed steel columns with multiple cross-arms used in Qingdao North Railway Station, China in which the roof of this station is supported by a number of prestressed stayed steel columns.

In the last decade, a number of researchers have investigated the behavior of steel columns. These studies mainly focused on the structural behavior of prestressed steel columns at ambient conditions. For instance, Hafez et al. (1979) derived a linear relationship between initial pretension and buckling load of an ideally straight prestressed stayed steel column. Temple (1977), based on finite element analysis, proposed a procedure to calculate buckling load of a "stayed column". Jemah and Williams (1990) tested a column supported by planar stay frame consisting of pretension stays and short arms. They found that the calculated collapse load gets increased by several times than that of the theoretical optimum prestress load predicted, if imperfections were to be ignored. Smith et al. (1975) studied various parameters influencing the stability and buckling load of prestressed steel columns. Based on these results, the authors inferred that critical load associated with symmetric mode of instability in column was highly influenced by small changes in the length of cross-arm and diameter of stay. This aforementioned symmetric mode of instability is through half-sine wave buckling pattern.

Temple et al. (1984) found that prestressed stayed steel columns with initial pretension lower than the optimum pretension force produced higher failure loads than those predicted by theoretical equations. The optimum pretension force corresponds to a pretension force level, beyond which any additional pretension has negative effect on the critical buckling load of the stayed column. Smith (1985) studied the effect of initial tension on the buckling load of prestressed steel columns and indicated that columns in which stays lose their pretension force before buckling occurs have a lower buckling load than similar columns in which stays maintain the level of pretension force. Saito and Wadee (2008), investigated the buckling behavior of prestressed stayed columns using Rayleigh-Ritz method, and inferred that the post–buckling behavior was strongly linked to the level of the initial prestress force.

Similarly, Wong and Temple (1982) through nonlinear finite element analysis found that the presence of initial out-of-straightness, leads to rapid rise in deflection and can decrease the buckling load. Chan et al. (2002) presented a second-order stability analysis of prestressed cable-stayed columns, with initial imperfections, and indicated that initial imperfections significantly alter the buckling load in the pretensioning and loading stages. Saito and Wadee (2009) developed a nonlinear model that accounts for geometrical imperfections in evaluating buckling behavior of a stayed column. This study indicated that prestressed stayed column tends to be most sensitive to imperfections during prestressing stage.

The above reviews clearly indicate that there have been numerous studies on the behavior of prestressed stayed steel columns at ambient conditions. Throughout these studies, the effects of buckling load, initial imperfections on axial load-carrying capacity of columns have been

quantified. However, there is very limited information in the literature on the behavior of prestressed stayed steel columns under fire conditions. To better understand the behavior of spacious prestressed stayed steel columns under fire conditions, an experimental program was undertaken at Wuhan University of Technology (WHUT) in collaboration with Michigan State University (MSU). Two prestressed stayed steel columns were tested under fire conditions. Results from these fire resistance tests are utilized to investigate the failure modes and fire resistance of the prestressed stayed columns under different combinations of load level and degree of prestressing.

2.0 Experimental details

Two full-scale prestressed stayed steel columns were tested under standard fire conditions. The core steel tube and cross-arms of prestressed stayed steel columns were fabricated with Grade Q235 steel, equivalent to that of S275 covered in BS 5950-1 (BSI 1990), while the stay of the columns was fabricated with Grade 1860 steel strand in Chinese Standard, equivalent to that of Standard Strand covered in BS 5896—2012 (BSI 2012). The test variables included load level and extent of prestressing force in cable strand. Further details on mechanical property parameters of the tested columns are given in Table 1.

2.1 Fabrication of test specimens

Seamless steel tube of ϕ159×6 mm cross section and 3 m long was used as a core tube for the two columns. In addition, steel bars of 40 mm diameter were used for the cross-arm of columns. In addition, steel cable strands of 15.24 mm diameter were used in the fabrication of stay of prestressed stayed steel columns (shown in Fig. 2).

The cross-arms of the column were fixed to the core column at one end and pinned to the cable

strands at the other end. For this purpose, the cross-arm were welded perpendicular to core column by using angular welds. This rigid connection can transfer compression from the cross-arm to the core column (See Fig. 2). Tension forces in the each pair of cable strands were unequal as shown in Table 1. The plane with higher magnitude of tension force (North-West direction as per Fig. 2) is the major plane, and the perpendicular plane is the minor plane. For instance, the pretension force on the major plane (North-West direction) in column PC1 is 102 kN and the pretension force on the minor plane (East-West direction) in this column is 76 kN.

For installing cable strands, two identical steel endplates with four annularly-distributed punched holes of 16 mm diameter were prepared (as shown in Fig. 2(b)). Then these two steel plates were welded to both ends of the core steel tube while paying special attention to centering them at mid-height of columns. The basic information relating to tested prestressed stayed steel columns, such as buckling load and maximum bearing capacity, is tabulated in Table 1.

2.2 Properties of steel

The mechanical properties of steel used in seamless steel tube were determined through tensile strength tests on steel coupons. Three coupons were prepared in accordance with the Chinese Standard GB/T 228.1-2010 (2010) and were tested for tensile strength and Yong's modulus. The tensile strength and Yong's modulus of the coupons was found to be 240 MPa and 206 GPa, respectively. The tensile strength and Yong's modulus of the steel cable strands was also determined in the same fashion and was found to be 1870 MPa and 195 GPa respectively. The variation of properties of core steel and strand with temperature is assumed to follow relations specified in EC3 (1993) and this is discussed in details by Zhou (2008).

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2.3 Test equipment

The prestressed stayed steel columns were tested in a specially built fire test facility. The test equipment, located in structural fire laboratory at Qingdao Technology University, China, comprises of a vertical heating furnace, a load equipment and data acquisition system (as shown in Fig. 3).

The furnace chamber had a floor area of 3×4 m^2 and a height of 3.8 m. Six thermocouples were mounted on three walls of the chamber to measure temperatures in the heating chamber. The total length of prestressed stayed steel column was 3 m with a length of 2.2 m exposed to fire. The bottom 0.3 m of the specimen was protected by fire insulation, and the top of 0.5 m of the column was outside the heating area, as shown in Fig. 3 (b).

A hydraulic jack, located at the top of the tested column, applied an axial load onto the test specimen. To control the magnitude of the axial load and to maintain a constant load during fire test, a force transducer connected to the hydraulic jack was used. Tension force in cable strands was measured by load sensors mounted the top plate of the column (see Fig. 4). The axial deformation in the column was monitored through the displacement of the top end plate of the column which was measured by installing a displacement transducer as shown in Fig. 5.

2.4 Test conditions and procedure

Both columns were tested under pin-pin end conditions. The displacements at bottom end of the test column were restrained in three directions, i.e., on plane normal to the axis of the column and in direction of the longitudinal axis of the column. The displacement at the top-end was restrained only on the plane normal to the axis of the column and the displacement along the longitudinal axis was kept free.

Both columns were tested under the application of a concentric load through a hydraulic jack, located at the top of the loading rig, with capacity up to a maximum of 2000 kN. The load ratio, defined as the ratio of the applied loading to buckling load of core column (Osofero, 2012), was calculated for pin-pin end conditions. This load ratio on columns PC-1 and PC-2 was 0.13 and 0.22, respectively.

In the tests, the load on columns was applied in increments about 30 min prior to the start of fire exposure, until the target load level was reached. This load level was maintained for 20 minutes until no further increment of axial and horizontal deformation could be measured. Then, the fire was ignited in the furnace. It should be noted that the heating rate and the load level on column were controlled manually and temperature and displacement data were collected using a programmable data acquisition system. The columns were considered to have failed and the test was terminated when they cannot sustain the applied loading (hydraulic jack could not maintain the load), reach a deformational rate of 9 mm/min according to Chinese code (2008), or reach a total axial deformation reached 200 mm which was the stroke of hydraulic jack.

2.5 Displacement and temperature measurements

Linear Variable Displacement Transducers (LVDTs) were installed to measure the axial displacement and lateral deflections of the tested columns. The instrumentation layout for measuring displacements and cross-sectional temperatures is shown in Fig. 5. Since LVDT's cannot withstand high temperatures, a fire-proofing molybdenum wire was placed horizontally as shown by the dashed line in Fig. 5. One end of the molybdenum wire was linked to the mid-height of the tested column, and the other end, outside the fire furnace, was connected to the different LVDT's. To obtain the lateral deflection at the mid-height of the column inside the fire exposure

zone, LVDT's D1 and D2 were used. Similarly, LVDT's D3 and D4 used to measure the lateral deflection through the molybdenum wire as aforementioned. Cross sectional temperatures on the columns were monitored through six thermocouples (TH1 to TH6) located along the height of the column as shown in Fig. 5.

3.0 Results and discussion

Data generated in fire resistance tests is utilized to evaluate the response of prestressed stayed steel columns under fire conditions. The test variables included loading level and level of prestressing. A comparative evaluation of fire response of columns is carried out by comparing progression of temperatures, deflections and failure patterns, as well as failure times.

3.1 Temperatures

The measured furnace temperature and temperature rise of prestressed stayed steel columns is plotted in Fig. 6 for columns PC-1 and PC-2. Figure 6 shows that there are slight differences between the measured furnace temperature and the ISO-834 standard temperature-time curve. This difference between generated temperature in furnace and required fire temperature as per ISO834 specifications can be attributed to malfunctioning of some of the burners in the furnace. The trends in Fig. 6 showed that there was a plateau in steel temperature at initial stages of fire exposure. This low temperature results from initial non-uniform temperature distribution along the column length. After attaining this plateau, the temperature in the tested column increases at a higher pace with fire exposure time. This is due to high thermal conductivity of steel and also high slenderness (thin wall) of the core column.

3.2 Axial deformations

When a column is exposed to fire, deformation in the column results from mechanical and thermal

strains. In the initial stage of fire exposure, axial deformation of the tested column mainly results from thermal strain (because the initial time is taken from the ignition time after undergoing contraction at room temperature). As steel temperatures increase, strength and elastic modulus of steel (in cable strands and core steel tube) decrease, leading to higher mechanical (compressive) strains. The compressive strain from mechanical (structural) loading neutralizes part of the thermal expansion, and this results in total axial deformation in column to be lower with exposure time in the mid-stages of failure. With further rise in steel temperature, the net strain (addition of thermal and mechanical strains) transforms deformations from elongation to contraction. Finally, large contraction results due to significant loss of stiffness in the columns and this finally leads to run-away failure in columns.

The variation of axial deformation with time for columns PC-1 and PC-2 is shown in Fig. 7. Data from column PC-2 is utilized to illustrate the typical progression of axial deformations in prestressed stayed columns. In the initial stage of fire exposure, the columns underwent elongation resulting from thermal expansion of steel (as shown in Fig. 7b). At 37 min into fire exposure, the elongation reached a peak of 26 mm. After about 37 min into fire exposure, the expansion of column begin to decrease and after about 97 min, expansion turns into contraction. Following this, Column PC-2 experienced large contraction in final stage and failed suddenly. This behavior is similar to that of a conventional steel column subjected to fire exposure and concentric loading (Wang and Kodur 2000, Wang and Li 2009, Li et al. 2010, Moura Correia and Rodrigues 2011, Correia and Rodrigues 2012, Han et al. 2013, and Osofero et al. 2013).

The peak value of elongation experienced by column PC-1 and column PC-2 are 26 and 24 mm,

respectively as shown in Figs. 7 (a) and (b). Average temperatures corresponding to the peak elongation value are 540 and 493 ˚C for the same columns as shown in Fg.8. In addition, the maximum value of contraction in prestressed stayed steel columns PC-1 and PC-2 were 45 mm and 52 mm (see Fig. 7), which correspond to a load level of 70 and 122 kN (see Table 1) respectively. This indicates that the larger the load ratio on the column, the greater the axial deformation would be.

Figure 7 also reveals contrasting difference in the progression of axial deformation in columns PC-1 and PC-2. For instance, peak value of axial deformation in column PC-1 occurred at 80 min of fire exposure when the temperature in the column reached 540 ˚C. Within very short duration, the column reached its maximum contraction at 87 min when steel temperature reached at 559 ˚C. Whereas the peak deformation in column PC-2 occurred at time of 37 min and steel temperature of 493 ˚C. Unlike column PC-1, the maximum contraction occurred at 52 min when steel temperature was 700 ˚C. It is clear that column PC-2 took significantly longer duration of time (15 min) as oppose to 7 min that took column PC-1 to reach maximum contraction. This can be attributed to the effect of lateral restraint at mid-height exerted by tension force of cable strands on the columns. As an illustration, it can be seen that the tension force in column PC-1 in minor plane relaxed at time of 15 min and the tension force in column PC-1 was substantial until critical (buckling) stage (see Fig. 9).

3.3 Lateral deflections

Figure 9 shows progression of lateral deflection at mid-height of columns PC-1 and PC-2, along major and minor planes, with fire exposure time. It can be seen that the lateral deflection of

columns in major plane is much lower than that in minor plane for both columns. For example, the maximum deflection in minor and major plane in column PC-1 is 160 and 100 mm, respectively. This variation is due to the fact that the magnitude of tension in major plane is greater than that in minor plane as shown in Table 1. This resulted in greater restraint stiffness at mid-height in major plane of the column than that in minor plane.

Lateral deflection in both prestressed stayed steel columns increased gradually with fire exposure time until failure occurred. Towards later stages of fire exposure, lateral deflection rapidly increased, leading to abrupt failure. This run-away failure is similar to failure in an axially loaded steel column under fire conditions (Kodur and Dwaikat 2009, Dwaikat et al. 2011, Mao and Kodur 2011). Moreover, the lateral deflection in the tested columns resulted from out-of-straightness of the columns (due to practical errors in fabrication and placement of columns) and unintended minor eccentricity in the application of loading. With rising temperature, elastic modulus and strength of steel degrade gradually and this leads to reduction in axial and lateral stiffness of column. Once the capacity of the steel column decreases to a level lower than that of applied loading, state of the column transforms into an unstable mode that is followed by an abrupt rise in lateral deflections.

A comparison of plotted lateral deflections in Fig 9 (a) and (b) show contrasting differences in deflection trends in columns PC-1 and PC-2. This is due to the fact that failure patterns are different in these two columns. The failure modes experienced in fire tests, as well as that predicted in

numerical analysis are compared in Fig. 11. Full details of numerical analysis is beyond the scope of this paper.

The tested columns PC-1 and PC-2 failed in different failure modes under fire exposure. In column PC-1, the tension force in cable strands in minor plane relaxed earlier to that of column PC-2 (see Fig. 9). This level of relaxation in cable strands could not produce sufficient support lateral restraint in the column PC-1 at mid-height (by the cross-arm). So, column PC-1 failed as a conventional steel column (See Fig. 11). However, tension force in cable strands of column PC-2 lasted longer than that in column PC-1, and hence cable strands could provide longer lateral restraint at mid-height (through cross-arm during) during fire exposure. So, column PC-2 failed in a symmetric mode, namely, through buckling pattern of a full-sine wave (See Fig. 10 (b) and Fig. 11 (b)).

3.4 Tension in cable strands

Evolution of tension force in cable strands of columns PC-1 and PC-2 is shown in Fig. 9 as a function of fire exposure time. The progression of tension force with fire exposure time can be grouped into three stages. In Stage I, which corresponds to the initial stage of fire exposure, the temperature variation is very marginal. This caused negligible elongation in cable strands, and the tension force in the cable strands remained basically invariant.

Stage II corresponds to the growth stage of fire, at which temperature in the cable strand rose at a faster rate. This led to significant thermal expansion in cable strands. At the same time, the strength and stiffness properties of steel degrade at a faster pace with rising steel temperature. These two

factors led to stress relaxation exceeding the mechanical strain at high temperature. Therefore, the tension force in cable stands decreases rapidly with fire exposure time in Stage II.

In Stage III, the remaining tension force in cable strands for columns PC-1 and PC-2 are shown in Fig. 9. It can be seen that the tension forces in cable strands in major plane of columns PC-1 and PC-2 are relatively high. However, the residual tension force in cable strands in minor plane is zero for column PC-1, in contrast to that of columns PC-2 which remained high. So, columns PC-1 and PC-2 still had enough lateral restraint at mid-height in major plane and this was facilitated by tensional cable strands through the cross-arm. This was the reason why columns PC-1 and PC-2 almost had no deflection in major plane. Figure 9 shows that cable strand in of column PC-1 relaxed in minor plane and the cable strand of column PC-2 possessed some level of tension. This difference led to different lateral restraint to be exerted at mid-height. Namely, for column PC-1, no lateral restraint exist due to relaxation of cable strands in minor plane, while lateral restraint at mid-height of column PC-2 decreased but still some level of restraint exists due to tension in cable strands in minor plane. Therefore, the deflection of the column PC-1 is larger than that in column PC-2 at mid-height in minor plane.

3.5 Failure temperatures and failure patterns

The temperatures and time at which failure occurred in two prestressed stayed steel columns is tabulated in Table 2. Failure temperature is defined as temperature at which a column cannot sustain the applied load. The time to reach failure temperature is taken as the fire resistance of the column.

It could be seen that the failure temperature in column PC-1 is much lower than that of column PC-2. This difference of failure temperature between the two columns can be explained as

discussed below. According to the stability theory of steel structures, a simply supported pin-pin compressed column has the lowest buckling load when the column buckles in half-sine wave pattern at ambient temperature due to the shortest effective length. In the tested columns, at elevated temperature, the effective length factor of the column PC-2 approached to 0.5 due to the lateral restraint at mid-height, while the effective length factor of the core column PC-1 was only 1.0 due to the disappearing lateral restraint at mid-height in minor plane. Due to the aforementioned analysis, the tested columns of PC-1 and PC-2 experienced different failure patterns, as shown in Fig. 11. It should be noted that in order to fully understand the fire behavior of prestressed stayed column, additional testing and numerical simulation is needed.

4.0 Conclusions

Based on the results presented, the following conclusions could be drawn.

- A prestressed steel column with restraints in two perpendicular prestress planes experiences failure on the plane with the lower tension.
- A prestressed stayed column experiences negligible lateral deflection in major plane and large deflections in minor plane.
- The failure in fire exposed prestressed stayed columns vary depending on prestess level and external load at high temperature. A stayed column fails in a half-sine wave buckling pattern when the lateral restraint exerted at mid-height by the tensional force in the cable strands disappears. Otherwise, a stayed column experiences failure through buckling in a full-sine wave when cable strands are in tension.
- Under fire conditions, the longer the tension in the cable strands is maintained, the longer

is the lateral restraints at the mid-height of a prestressed stayed column. This lateral restraint influences failure pattern in the prestressed stayed columns at elevated temperature.

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Parameter	PC-1	PC-2
The minimum effective tension force (kN)	77	77
Linear optimum pretension (kN)	144	144
Pretension in the cable strands in major plane (kN)	102	156

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Pretension in the cable strands in minor plane (kN)	76	123
The maximum buckling load of core column with prestressed stay (kN)	1900	3030
Critical load of core column without prestressed stay (kN)	544	544
Applied axial compression load (kN)	70	122
Ratio of the applied loading to buckling load	0.13	0.22

Table 2 Critical temperature and failure time in tested columns PC-1 and PC-2

Column	$T_w(kN)$	Load ratio	Time to failure (min)	Failure steel temperature(℃)
PC-1	76	0.13	89	559
PC-2	123	0.22	105	700

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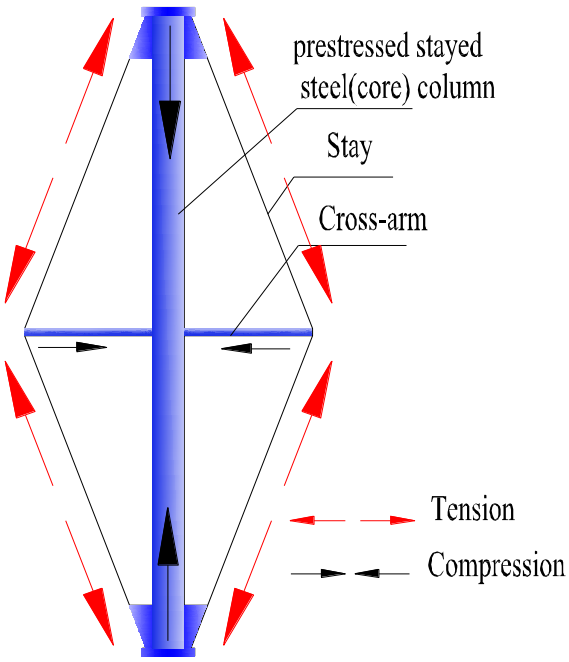
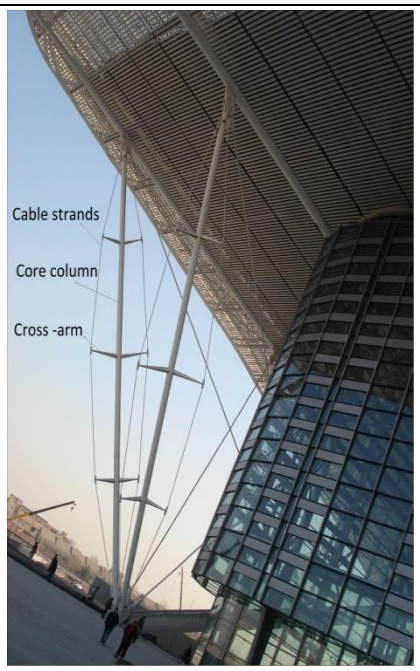


(a) Components of prestressed stayed steel column



(b) Typical application

Fig. 1 Principle of prestressed stayed steel columns and its engineering application

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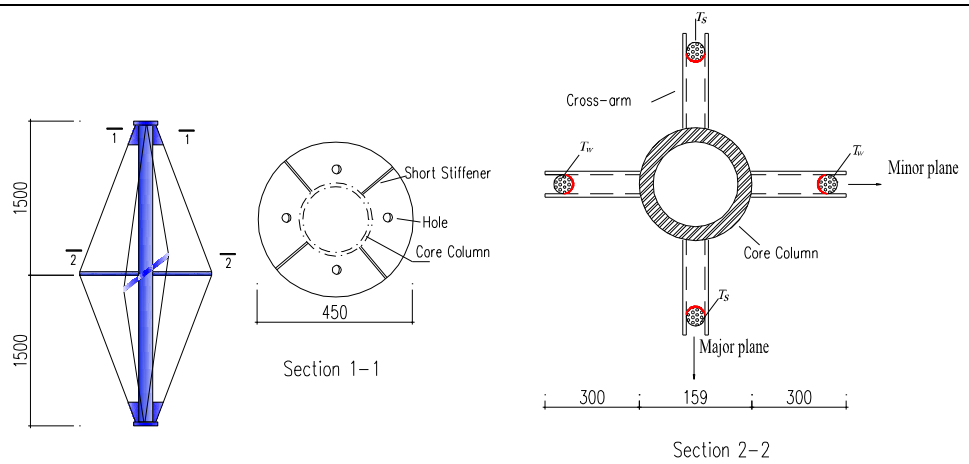

(a) Elevation (b) Cross section at top (c) Cross section at mid-height

Fig. 2. Elevation and cross-section of a typical prestressed stayed steel column

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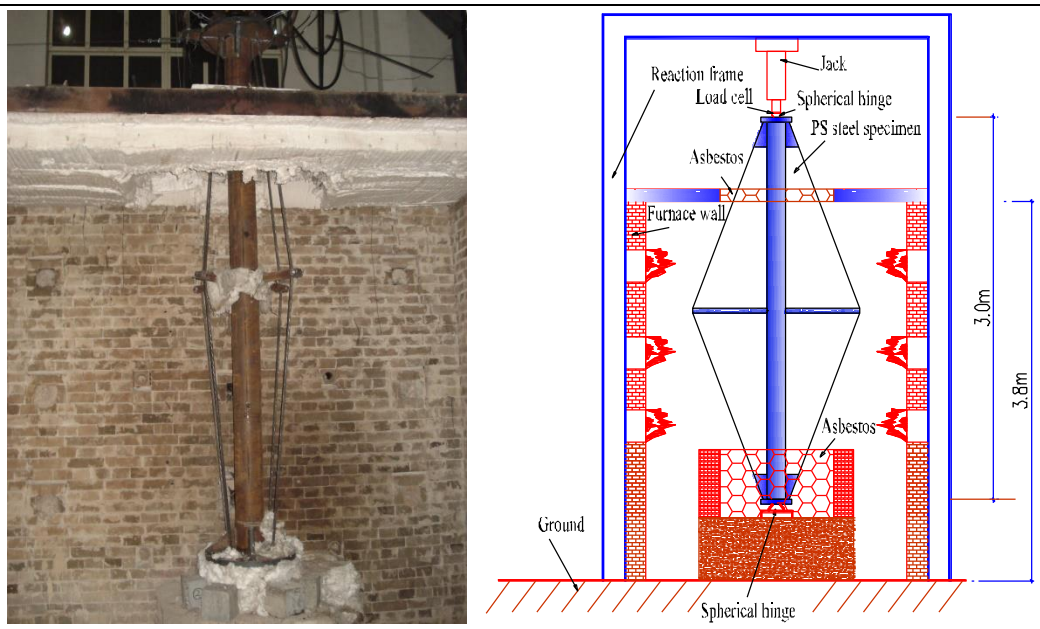

(a) Picture of furnace (b) Scheme diagram of test setup

Fig. 3 Test set-up of fire resistance tests

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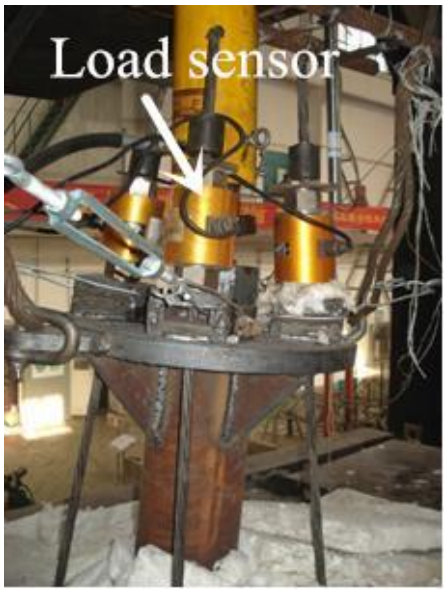

Fig. 4 Details of loading sensor at top of columns

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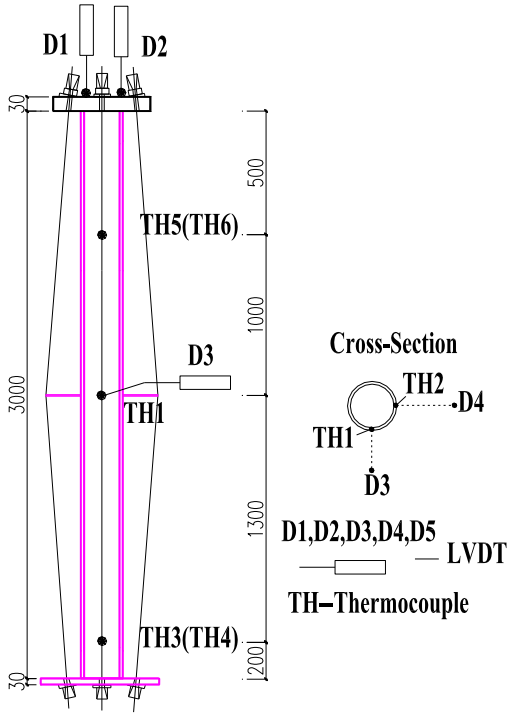

Fig. 5 Tested column with position of LVDTs and thermocouples

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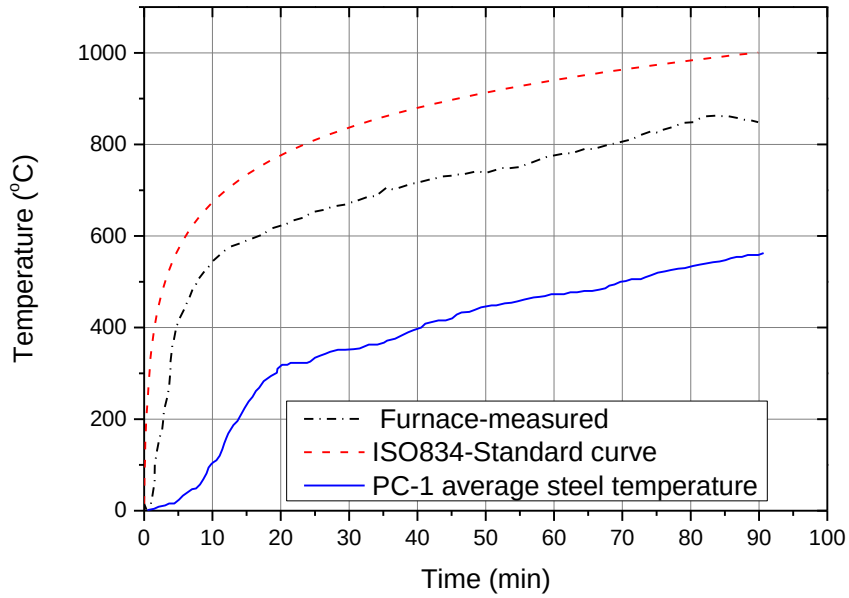
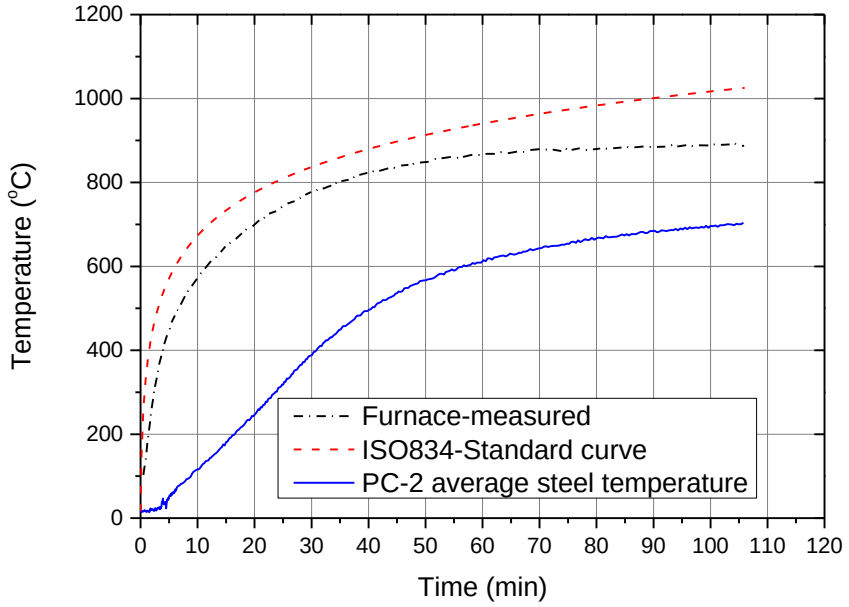


(a) Evolution of temperature in column PC-1



(b) Evolution of temperature in column PC-2

Fig. 6 Measured temperature as a function of time in columns PC-1 and PC-2

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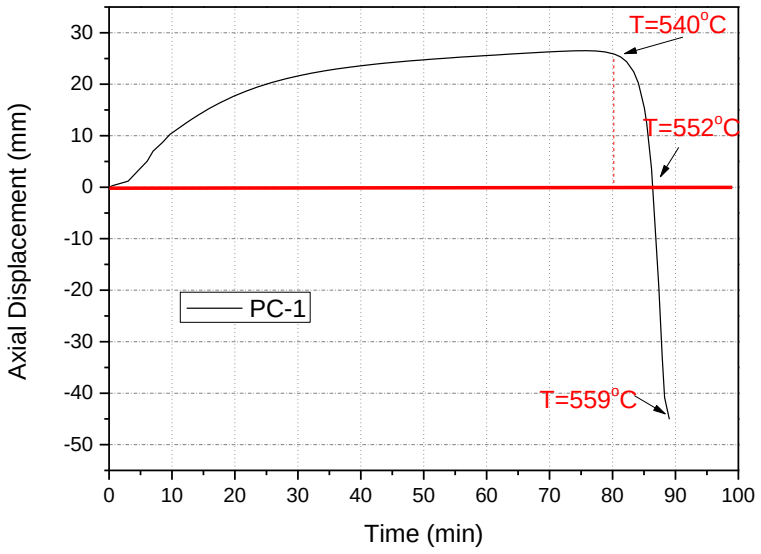
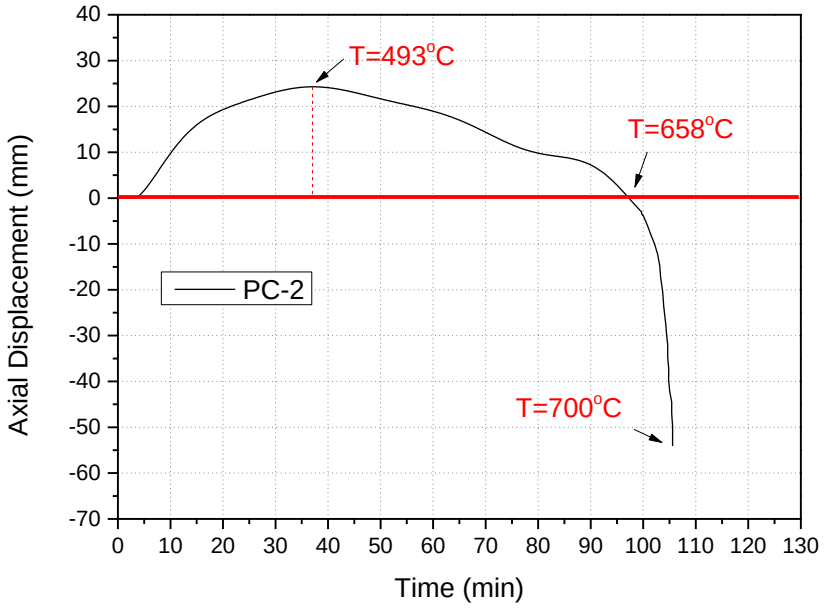


(a) Column PC-1.



(b) Column PC-2.

Fig. 7 Progression of axial deformation in tested columns PC-1 and PC-2.

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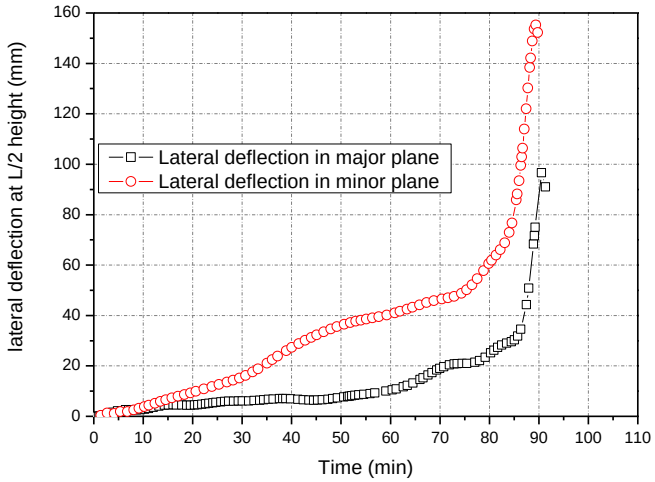
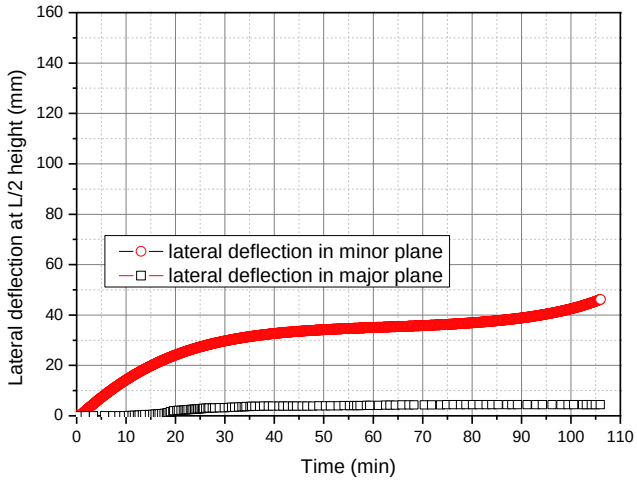


(a) Column PC-1



(b) Column PC-2

Fig. 8 Lateral deflection of the prestressed stayed steel columns PC-1 and PC-2

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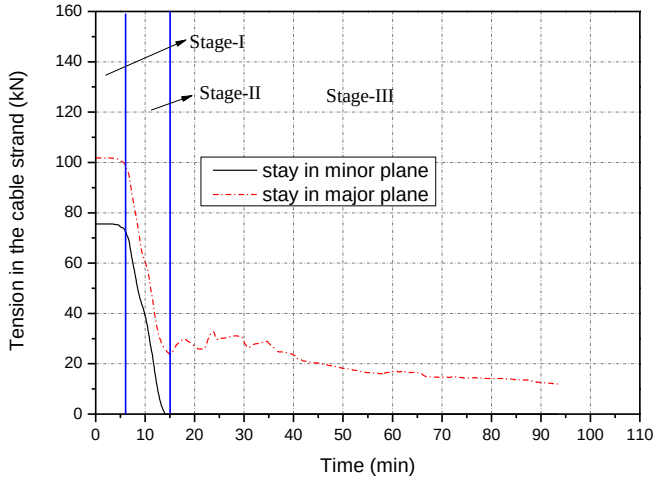
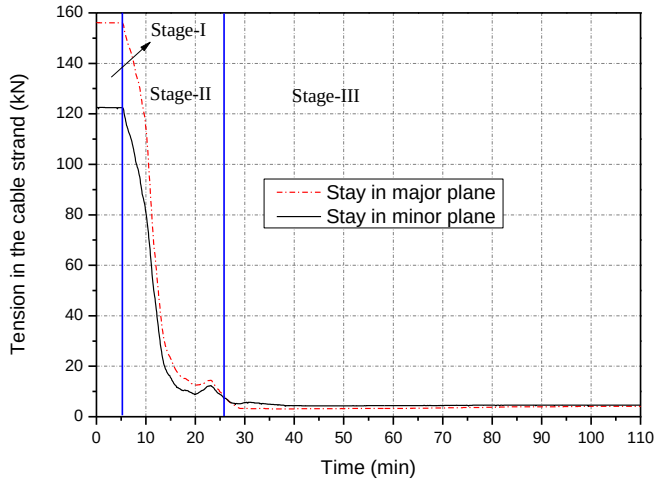


(a) Column PC-1



(b) Column PC-2

Fig. 9 Evolution of tension force in the tested columns P-1 and PC-2

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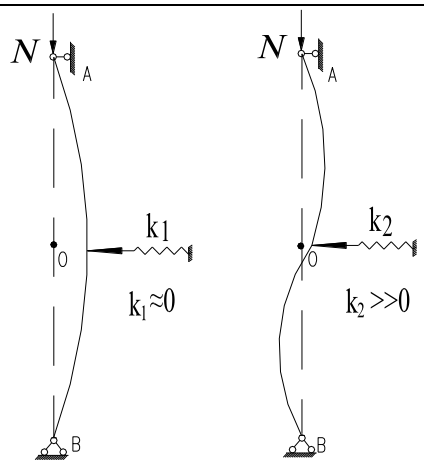

(a) Symmetric buckling (b) Non-symmetric buckling

Fig. 10 Buckling modes of columns along minor plane

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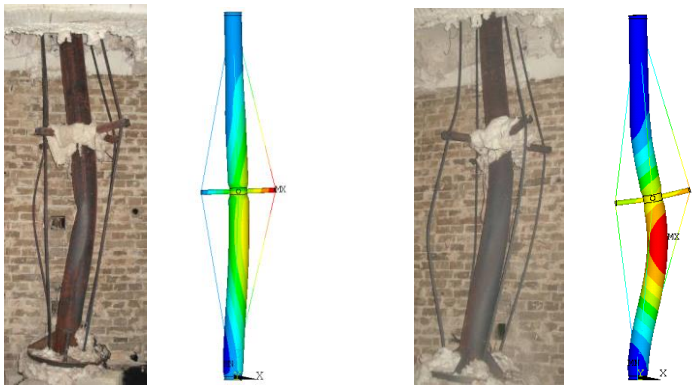

(a) Column PC-1 (b) Column PC-2

Fig. 11 Failure mode of tested columns PC-1 and PC-2 as per fire tests and numerical analysis