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Study on shear performance and residual shear strength of concrete members after fire

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ABSTRACT
Some concrete structures collapse during fire accidents, implying that it is of great significance to evaluate residual strengths of concrete members after fire-damage. And many studies have been conducted on axial and flexural property, but the studies on the shear property are imperfection. An experiment was designed for discussing the shear behavior of fire-damaged concrete frame structure, including restrained beam ,cantilever and fire-damaged frames. The shear resistant mechanisms of fire-damaged concrete beams were also discussed based on experiments. The following conclusions can be drawn from test and analysis :The shear bearing capacity declined after fire. The concrete slab (flange), in a certain extent, can also improve the shear capacity of the beams, and with the increase of shear span ratio, the flange's strengthen effect on the shear capacity of the beams weakened gradually. The shear carrying capacity of fire-damaged frame beams with shear span ratio 2.0 are much lower than that of ones with shear span ratio 1.0. The strong column frames in normal temperature would change into strong beam frames after fire. The stiffness and energy-dissipating capacity declined after fire. The deformability and bearing capacity also declined after fire.

KEYWORDS: *concrete structure, fire behavior, shear property, residual bearing capacity*

1. GENERAL INSTRUCTIONS

In the past years, a great deal of new knowledge has been generated to understand the fire behaviors of shear performance and residual shear strength of concrete members after fire. Khan et al conducted shear tests of 111 Beams without web reinforcement[1]. The results showed that the shear strength of the beams has been found to increase by up to 10% at lower temperature cycles of 100 and 200°C but reduces by up to 14% at higher temperature (300°C) depending on the severity of thermal loading, and it is also found that shear bearing capacity decline slowly, with the increase in diameter of longitudinal reinforcement. Shear bearing capacity theory is constantly updated and improved, Professor Collins et al. introduces such a theory and explains the simple design models derived from the theory, which include a strut-and-tie model for disturbed regions and a sectional model for flexural regions[2]. Shear tests of simply supported beams after fire were carried out by Lu[3], then shear tests of restrained beams and cantilever after fire were also conducted by our research group. Fire tests and numerical simulations of these tests were conducted to investigate the fire behaviors of concrete beams, the effects of axial and rotational restraints on beams were also investigated[4]. Recently, some researchers have paid their attentions to fire behaviors of the whole frames[5]. However, the fire behaviors of the whole frames are hard to handle at present. So it is of great importance to pay attention to the basic test data of fire-damaged concrete frames. Although fire behaviors of concrete beams, simply supported or frame beams, have been studied by many researchers[6, 7], more research of the performance of fire-damaged concrete frames is needed. This is because the frame after fire is more likely broken down by shear-bond failure. In this paper, the experimental programme of restrained beams and cantilever after fire and fire-damaged concrete frames will be introduced , and the experimental results will also be described. Based on these results, several conclusions are given.

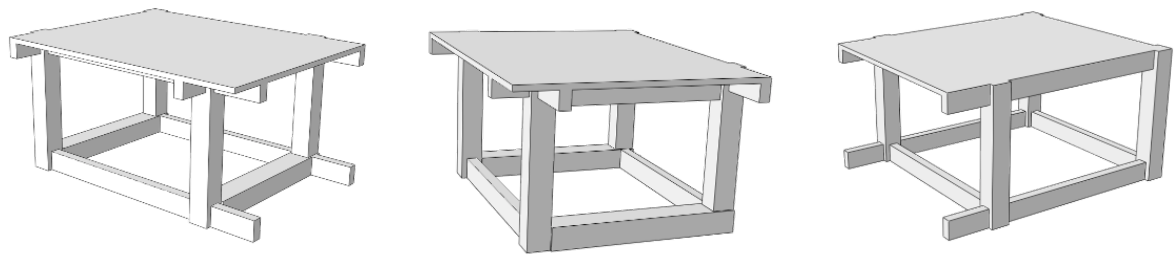
2. EXPERIMENTAL INVESTIGATION

2.1.Test specimens

To ensure that shear failure of the frame beams occurs, the stirrup requirements with a diameter of 8mm and a spacing of 200mm were employed, and the beams for tests were all reinforced using four longitudinal steel bars with a diameter of 14mm. All the slabs were 60mm thick and the longitudinal and transverse reinforcements in the slabs were steel bars of 6mm diameter at 150mm interval. Thickness of a protective layer was 10mm.

In the frame members, the dimensions of all beams used for the test investigation is 120mm × 250mm, column size is 250mm × 250mm, Thickness of a protective layer was all 15mm.Three-dimensional model is shown in, Figure 2.1(a) shows a normal model, Figure 2.1(b) shows a high temperature model frame 1, Figure 2.1(c) shows a high temperature frame 2 models. During the test, by cutting the concrete slab, the restriction effect of slab for some test beam is removed. To investigate the shear behavior of frame beams after high temperature, static load test on the frame beams is conducted. And quasi-static test for fire-damaged concrete frames is also conducted in order to investigate the shear behavior of frame columns after fire.

The test-day (100mm× 100mm×300mm) prism compressive strength of concrete was 43.9MPa.The measured yield strength and ultimate strength of the steel bars with a diameter of 6 mm were respectively 413.4MPa and 510.2MPa, those of the steel bars with a diameter of 14 mm were respectively 589.9MPa and 701.7MPa, those of the steel bars with a diameter of 16 mm were respectively 484.0MPa and 597.3MPa, and those of the steel bars with a diameter of 20 mm were respectively 446.5MPa and 582.3MPa.



(a) Frame under normal temperature (b) Frame 1 after fire (c) Frame 2 after fire
Figure 2.1 Three-dimensional model

Five parameters were considered in the static load tests, including constraint type, connected to the slab or not, Span(mm), shear span ratio and stirrups. Details of specimens under normal temperature are listed in Table 2.1, details of specimens after fire are listed in Table 2.2. In the Table 2.1, the letter "a" means the distance between the loading point and the support; "h" means the height of the compression zone; "C" means normal temperature; "KL" means frame beam; "XL" means cantilever; the numbers behind "-" means the shear span ratio; "L" means it can be regarded as a L-shaped beams; "T" means it can be regarded as a T-shaped beams.

Two parameters were mainly considered in the quasi-static tests, including fire-damaged or not and the relative stiffness of beams and columns. Details of fire-damaged concrete frames are listed in Table 2.3.

Table 2.1 Details of specimens under normal temperature

Specimen	constraint type	connected to the slab or not	Span(mm)	a(mm)	shear span ratio(a/h0)	stirrups
CKL-2.0	restrained	not	2350	450	2.0	R6@100/ R6@150
CKL-3.0	restrained	not	2350	675	3.0	R6@100/R6@150
CKL-4.3	restrained	not	2350	925	4.1	R6@100/R6@150
CKL-4.3L	restrained	one side with slab	2350	925	4.1	R6@100/R6@150
CXL-1.0	cantilever	not	375	225	1.0	R6@150
CXL-2.2	cantilever	not	575	500	2.0	R6@150
CXL-2.2T	cantilever	two sides with slab	576	500	2.1	R6@150

Note: R is the mild steel round bars.

Table 2.2 Details of specimens after fire

Specimen	constraint type	connected to the slab or not	Span(mm)	a(mm)	shear span ratio(a/h0)	stirrups
KL-1.0L	restrained	one side with slab	2350	225	1.0	R6@100/R6@150
KL-2.0L	restrained	one side with slab	2350	450	2.0	R6@100/R6@150
KL-2.0T	restrained	two sides with slab	2350	450	2.0	R6@100/R6@150
KL-3.0L	restrained	one side with slab	2350	680	3.0	R6@100/R6@150
KL-1.0	restrained	not	2350	225	1.0	R6@100/R6@150
KL-2.0	restrained	not	2350	450	2.0	R6@100/R6@150
KL-2.0W	restrained	not	2350	450	2.0	R6@150
KL-3.0	restrained	not	2350	695	3.0	R6@100/R6@150
KL-4.3	restrained	not	2350	925	4.1	R6@100/R6@150
KL-4.3L	restrained	one side with slab	2350	925	4.1	R6@100/R6@150
XL-1.0	cantilever	not	375	225	1.0	R6@150
XL-1.4	cantilever	not	375	300	1.4	R6@150
XL-1.7	cantilever	not	575	380	1.7	R6@150
XL-2.0	cantilever	not	575	450	2.0	R6@150
XL-1.7L	cantilever	one side with slab	575	380	1.7	R6@150
XL-2.0L	cantilever	one side with slab	575	450	2.0	R6@150
XL-2.2	cantilever	not	575	500	2.2	R6@150
XL-2.2T	cantilever	two sides with slab	575	500	2.2	R6@150

Note: R is the mild steel round bars.

Table 2.3 Details of fire-damaged concrete frames (mm)

Specimen	column				beam				
	clear height	size	rebar	stirrups	clear span	size	upper rebar	lower rebar	stirrups
CKJ-1	1200	250	9T16	R6@150	1850	150×350	1T20+2T14	2C20	R6@150
CKJ-2	1300	250	9T16	R6@150	1850	120×250	2T14	2C14	R6@150
KJ-1	1200	250	9T16	R6@150	1850	150×350	1T20+2T14	2C20	R6@150
KJ-2	1300	250	9T16	R6@150	1850	120×250	2T14	2C14	R6@150

Note: T and R is the high-yield deformed steel bars and the mild steel round bars, respectively.

2.2.Test steps

Temperature load is applied to all the specimens in a furnace chamber, as is shown in Figure 2.1. The surface of the slab is applied to 1 kN/ m^2 uniformly distributed load, as is shown in Figure 2.3. During fire tests water stains appeared one after another on the plate, as is shown in Figure 2.4 and Figure 2.5. After fire, concrete spalling was found at the bottom of the slab, steels were exposed, and there were a few cracks on the surface of the slab, as is shown in Figure 2.6 and Figure 2.7. Then static load test and Quasi - static tests were conducted. Figure 2.8 and Figure 2.9, respectively , shows the loading schematic for restrained beam and cantilever. Figure 2.10 shows the quasi-static test for fire-damaged concrete frames. Temperature was increased by the furnace according to ISO834 standard heating curve, Figure 2.11 shows a comparison between the ISO834 standard fire curve and the measured time-temperature curve in the furnace. Both the thermal and structural responses were measured during the tests and collected automatically by the computers.

Figure 2.2 the furnace chamber

Figure 2.3 uniformly distributed load

Figure 2.4 Water stains of frame 1

Figure 2.5 Water stains

Figure 2.6 The bottom of slab after fire

Figure 2.7 The surface of slab after fire

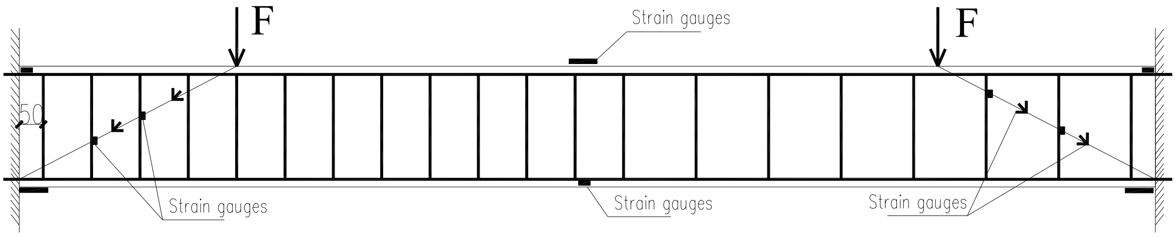


Figure 2.8 Loading schematic for restrained beam

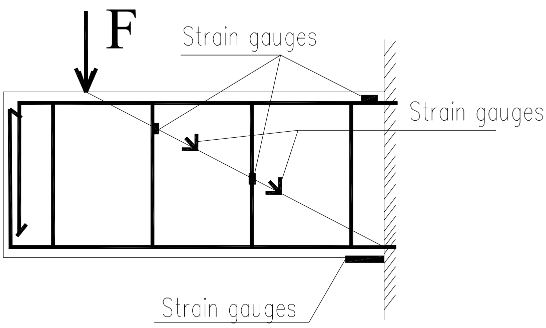


Figure 2.9 Loading schematic for cantilever

Figure 2.10 Quasi-static test for fire-damaged concrete frames

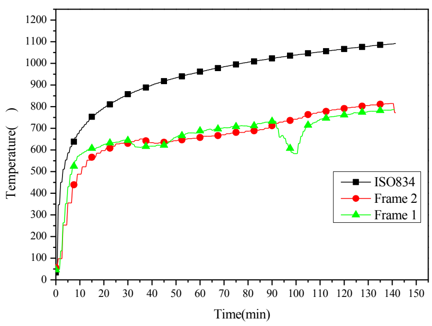


Figure 2.11 Measured time-temperature curve in the furnace

3. TEST RESULTS

3.1. The results of static load test

Load Statistics of specimens under normal temperature and specimens after fire are shown in Table 3.1 and Table 3.2. It can be seen that: (a)the shear bearing capacity of specimens after fire declined. For example, the specimen CKL-2.0 and KL-2.0 are same in cross-sectional dimensions and reinforced beams, but ultimate bearing capacity of the specimen CKL-2.0 is lower. (b)The concrete slab (flange) can also improve the shear capacity of the beams, such as the specimen XL-2.2 and XL-2.2T.

3.2. The results of quasi-static test

The order of plastic hinge and failure mode fire-damaged concrete frames are shown in table 3.3,and some important ductility parameters of the frames are shown in table 3.4. The hysteretic curves of fire-damaged concrete frames are shown in Figure 3.1. The skeleton curves of fire-damaged concrete frames are shown in Figure 3.2. The failure mode of the fire-damaged concrete frames is shear-bond failure. Shear-bond failure is an important failure mode for reinforced concrete (RC) structures subjected to seismic actions. Compared with flexural-shear failure, the specimens with bond failure have clearly "pinching effect", poor ductility and energy dissipating capacity and faster stiffness degradation, which leads to a non-ductile behavior of the specimens. It can be seen that: (a) The pinching effect of fire-damaged frames' hysteretic curves is more pronounced, the less full hysteresis loop of fire-damaged frames indicates that seismic performance of the frames declined after fire, as is shown in Figure 3.1. (b) The strong column frames in normal temperature would change into strong beam frames after fire, as is shown in Table 3.3. (c) The deformability and bearing capacity also declined after fire, as is shown in Table 3.4.

Table 3.1 Load Statistics of specimens under normal temperature(kN)

	CKL-2.0	CKL-3.0	CKL-4.3	CKL-4.3L	CXL-1.0	CXL-2.2	CXL-2.2T
Initial crack	25	30	20	30	15	25	14
Longitudinal reinforcement	140	100	175	170	120	60	100
Stirrups yield	140	135	not yield	175	not yield	70	75
Ultimate load	175	200	185	175	145	76.7	102

Table 3.2 Load Statistics of specimens after fire(kN)

	KL-1.0L	KL-2.0L	KL-2.0T	KL-3.0L	KL-4.3L
Initial crack	60	40	50	30	28
Longitudinal reinforcement	not yield	not yield	not yield	130	/
Stirrups yield	250	130	140	130	/
Ultimate load	275	145	200	156.7	140
	KL-1.0	KL-2.0	KL-2.0W	KL-3.0	KL-4.3
Initial crack	80	30	32	30	18
Longitudinal reinforcement	not yield	not yield	not yield	not yield	/
Stirrups yield	225	110	/	100	/
Ultimate load	240	155	67.2	150	132
	XL-1.0	XL-1.4	XL-1.7		XL-2.0
Initial crack	15	20	15		10
Longitudinal reinforcement	60	not yield	55		70
Stirrups yield	70	55	40		50
Ultimate load	73	68.8	55		71
	XL-1.7L	Xl-2.0L	XL-2.2		XL-2.2T
Initial crack	15	20	14		28
Longitudinal reinforcement	55	45	not yield		84
Stirrups yield	45	35	/		/
Ultimate load	75	70	63		84.000

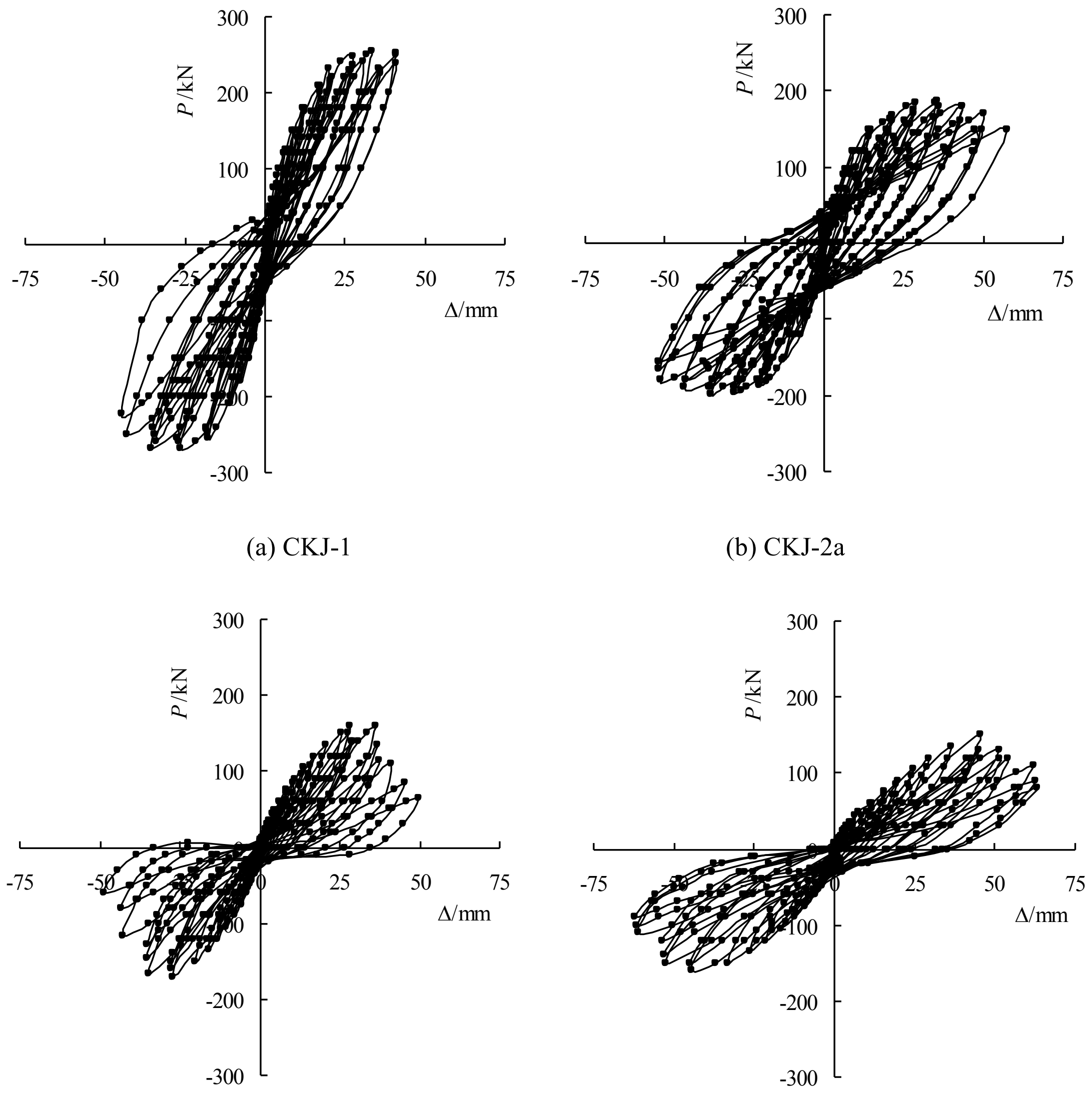
Table 3.3 Failure mode of fire-damaged concrete frames

	order of plastic hinge	failure mode
CKJ-1	the top of column- the end of beam- the bottom of column	column broken
CKJ-2	the end of beam- the bottom of column- the top of column	beam broken
KJ-1	The function of the plastic hinge did not play	shear-bond failure
KJ-2	the bottom of column- the end of beam- the top of column	shear-bond failure

Table 3.4 Ductility parameters of the frames

	P_y/kN	Δ_y /mm	P_{max} /kN	P_u /kN	Δ_u /mm	Δ_u/Δ_y
CKJ-1	210/180	16.7/7.7	254/269	252/269	41.1/35.7	2.5/4.6
CKJ-2	150/150	14.0/12.2	186/198	170/180	50.0/51.0	3.6/4.2
KJ-1	135/120	20.8/20.1	160/170	160/166	36.2/34.9	1.7/2.7
KJ-2	105/120	24.5/21.5	150/160	130/150	51.5/54.2	2.1/2.4

Note: Py is the yield load; Δy is the yield displacement; Pmax is the peakload; Pu is the ultimate load; Δu is the ultimate displacement; Δu/Δy is the ductility factor.



(a) CKJ-1 (b) CKJ-2a

(c) KJ-1 (d) KJ-2

Figure 3.1 Hysteretic curves of fire-damaged concrete frames

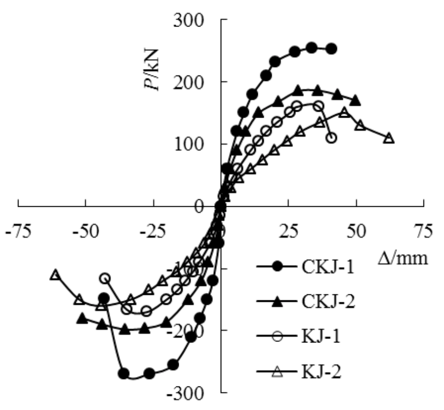


Figure 3.2 Skeleton curves of fire-damaged concrete frames

4. CONCLUSIONS

The following conclusions can be drawn based on the results of this experimental study:

1. The shear bearing capacity declined after fire.
2. The concrete slab (flange), in a certain extent, can also improve the shear capacity of the beams, and with the increase of shear span ratio, the flange's strengthen effect on the shear capacity weakened gradually.
3. The shear carrying capacity of fire-damaged frame beams with shear span ratio 2.0 are much lower than that of ones with shear span ratio 1.0.
4. The frame after fire under cyclic loading is more likely broken down by column exfoliation. The strong column frames in normal temperature would change into strong beam frames after fire.
5. The stiffness and energy-dissipating capacity declined after fire. The deformability and bearing capacity also declined after fire.

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