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Research Paper

Behaviour of Concrete and Steel in Fire

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ABSTRACT: *Fire is recognised as a significant hazard during the lifecycle of a structure, and it is a regulatory requirement of any building design to maintain structural integrity during a fire attack. Design techniques and construction materials greatly reduce the risk of fire injury or death from fire. This is with a view to understanding the ability of structural elements made from different materials to survive under fire and continue in service. In this study, we presented detailed analysis of the fire resistance behaviour of different structural elements like CFST columns [1], Composite members [2], cellular beams [3], CTSRC columns [4], and stainless steel members [5]. And the results and observations of various experimental studies are discussed in this, which can tell us about the behaviour of concrete and steel in fire.*

KEYWORDS: *Fire resistance, Fiber reinforced concrete, concrete filled steel tubular columns , circular tubed steel reinforced concrete columns .*

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NOMENCLATURE

CFST	:	Concrete filled steel tubular
CTSRC	:	Circular tubed steel reinforced concrete
CIF	:	Cool in furnace
E	:	Young's modulus
f0.2	:	0.2% proof strength
fu	:	Ultimate strength
η_{fi}	:	Design load level in fire condition
R	:	Structural fire resistance
T_C	:	Thermocouple in concrete
T_S	:	Thermocouple in the steel tube
Θ exp	:	Target temperature

I. INTRODUCTION :

Current fire resistance strategies for buildings often include both active and passive protection methods. Fire alarms and sprinklers, etc., are examples of proactive measures that require either human intervention or automatic activation and help control the spread and effects of fire when necessary. The construction system includes passive fire prevention measures by selecting appropriate materials, measuring component dimensions ,structural element shapes and cross-sections, and constructing compartments. They control the spread of fire and its effects by providing sufficient fire resistance to prevent loss of structural stability within a given time period determined by building occupancy and fire safety objectives. Therefore, this study describes an experimentaltests for various components in case of fire.

Information in this study is available only for passive fire protection methods. Active fire protection system area important part of a fire protection plan, but activeSystems are prone to failure, and human intervention may not be enough to prevent fires, leaving only passivefire protection measures. Since these guidelines only deal with structural fire resistance calculations and not the broader structural fire protection methods, the issue between active and passive fire protection measures is not addressed.Theserecommendationscontainonlyconcrete and steel-related information. Other common building materials such as these are not mentioned, given that stone and wood are unlikely to be used as primary structural materials in high-rise buildings.

The design recommendation for concrete-filled tubular steel (CFST) columns in a fire isand is based on the results of a standard laboratory fire test of CFST members in which the columnwas subjected to the same temperature throughout the height of the column. However, this study is not representative of his CFST columns in a typical building. The CFST columns are continuous between floors, so if one floor catches fire, the floors above and below will remain cold [1].On the other hand, most studies do not consider the interaction of columns with the surrounding building structure. The response of these columns when inserted into the structure of the buildingis different than when they are isolated. Therefore, in this study, we presentresearch papers , in which axially loaded with constrained thermal expansion by a three-dimensional (3D) steel frame to evaluate the effect of ambient axial and torsional stiffness [2].

Later in this study,discussed the behavior of cellular beams . Steel beams with Web Openingsis commonly used in truss construction, allows for lighter building and longer spans, large column free interior space, high flexibility in cable routing allows forand reduced floor-to-floor height. The focus of thisstudy [3] is the fire performance of perforatedstainless steel beams, which combines the attractive properties ofstainless steel with the static efficiency of perforated beams. A fire test has conducted which is described & discussed[3] . As mentioned earlier, the shape and cross-section of structural elements are measures of resistance, and in this study we performed a detailed experimental study of circular cross-section columns in the case of fire. The study also includesexperimental processes on his CTSRC (Circular Tubed Steel Reinforced Concrete) columns in fire[4].In addition, stainless steel has been found to perform better than carbon steel in fire conditions because it offers excellent mechanical properties and corrosion resistance and retains strength and stiffness over time. Research also focuses on post-fire conditions, which have received limited attention from the research community. The motivation for this work is to demonstrate that fire-vulnerable stainless steel elements can be repaired in a short period of time with minimal additional cost[5].

And for the discussion of each and every experimental study we have given several tables and figures those will give the results and observations for the measures which were calculated during the tests. Finally , based on the obtained results and performance conclusions were given .

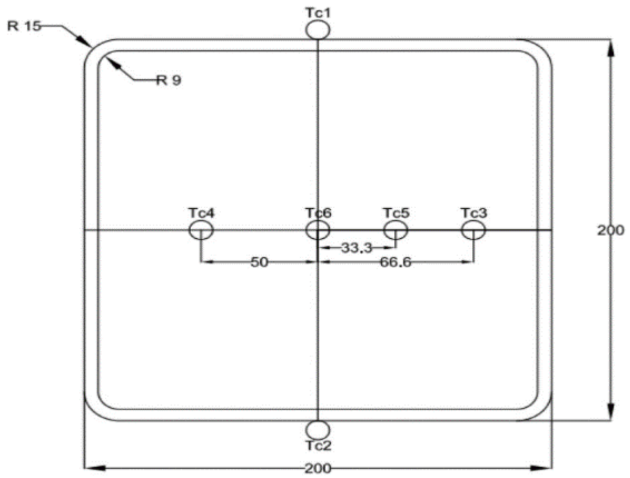
II. LITERATURE REVIEW :

Here , What is described and discussed in this study revolves around a keytopic –FIRE RESISTANCE BEHAVIOUR OF CONCRETE AND STEEL. The ability ofmaterials to withstand fire at different temperatures is called fire resistance, and how this concreteand steel resist fire is the main focus of this topic.

As we have discussed, current fire resistance strategies for buildings often include both active and passive protection methods. We do not discuss the active protection methods here. Support systems therefore include passive fire protection measures through selection of appropriate materials, measurement of component dimensions, and shape and cross-section measurement of components and structural sections. They protect against fire and its effects by providing sufficient fire resistance to prevent loss of structural stability within a given time period determined by building occupancy and fire safety objectives. Control the spread. Moving down this section you can clearly see the experimental testing of different structural elements. They are:

- CONCRETE FILLED STEEL COLUMNS (CFST) :

The experimental tests reported here dealt with partially heated lengths ofcontinuous columns and various concrete fillings with compressive strengthsranging from 80 MPa to 95 MPa [1] . A total of 10 tests were performed using only2m columns with a height of 3.2m. All tests were performed on square hollow sections ofsteel with nominal cross-sectional dimensions of 200 mm × 200 mm × 6 mm and 220 mm × 220 mm × 6mm, as shown in Figure (1.a). The tests included 3 types of fillers. Simple concrete, reinforced concrete, and steel fiber concrete. Fixed-fixed (F-F) and plug-in-fixed (P-F) end fittings were considered. From the experimentally measured structural fire resistance (R), the axial strength of the column in fire was calculated. The temperature was then calculated using thermocouples placed at different locations, as shown in Fig. (1.a). Finally,obtained the results and observations shown in Table (1.1). Therefore, after all tests, steel fiber concrete columns were found to have higher R compared to plain and bar reinforced concrete infills.Due to the tensile strength of concrete core thus increasing bending strength[1].



Fig(1.a)

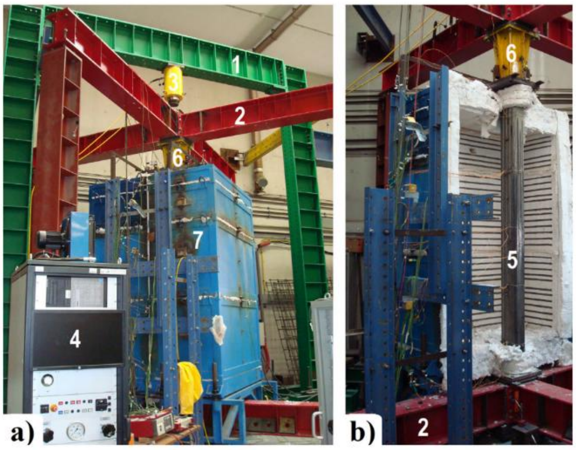
Details of test specimens.

Specimen	Column size	End conditions	f_s N/mm^2	f_c N/mm^2	$E_{fi,Rd}$ kN	η_{fi}	δ	λ_r	R Min	Failure mode
a) Plain concrete										
P1	200 mm × 200 mm × 6 mm	F-F	569	86.9	1378.4	0.37	0.54	0.55	37	Flexural
P2	200 mm × 200 mm × 6 mm	F-P	569	81.33	1378.4	0.38	0.56	0.66	26	Flexural
P5	220 mm × 220 mm × 6 mm	F-F	461	84.4	1415.2	0.35	0.47	0.48	38	Local
b) Conventionally reinforced concrete										
R1	200 mm × 200 mm × 6 mm	F-F	569	86.1	1485.6	0.37	0.51	0.56	46	Flexural
R2	200 mm × 200 mm × 6 mm	F-P	525	94.5	1485.6	0.37	0.47	0.67	23	Flexural
R5	220 mm × 220 mm × 6 mm	F-F	461	86.05	1604.3	0.35	0.41	0.49	72	Plastic
c) Fibre reinforced concrete										
F1	200 mm × 200 mm × 6 mm	F-F	525	90.9	1378.3	0.37	0.51	0.54	24	Flexural
F2	200 mm × 200 mm × 6 mm	F-P	525	91.9	1378.4	0.37	0.51	0.66	25	Flexural
F5	220 mm × 220 mm × 6 mm	F-F	461	79.86	1415.2	0.36	0.49	0.47	85	Flexural
F6	220 mm × 220 mm × 6 mm	F-P	461	94.3	1415.2	0.33	0.45	0.59	51	Flexural

Table(1.1)

- COMPOSITE COLUMNS- CONCRETE &STEEL :

Here, the experiment proceeds as follows.longitudinal rebar and steel hooks were welded to the S355 steel plate and then also to the ends of hisprofiles in steel tubes [2]. Lateral reinforcement was performed on each specimen by an 8 mm diameter stirrup at a distance of 150 mm. The concrete was ready-mix concrete and was poured vertically into the hollow column specimens. Once the column is ready, it should be tested in a 3D setup as shown in Figure (2.a). The tested columns were placed in the centre of the 3D restraining frame and tested accordingly. So after placing the specimen, it was tested in two stages. They are; loading and heating stage. Finally , the results obtained and are shown in Table(2.1)[2] . And they observed that, although the furnace temperature was higher than 700 °C for over 30 min, the temperature in the column core (T_C1) and on the longitudinal steel reinforcing bars (T_S1a) did not exceed 200 and 350 °C, respectively [2]. This means that the loss of load-bearing capacity of CFST columns is mainly due to deterioration of the mechanical properties of the tubular steel profile and the concrete between the tube and the rebar[2].



Fig(2.a)

Main experimental results of the fire resistance tests on CFST columns.

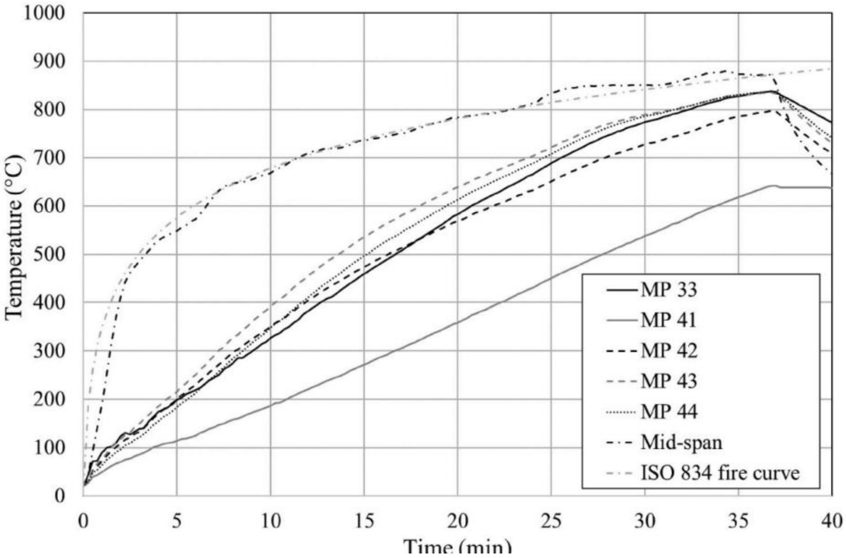
Test reference	At the maximum axial compression force					At critical time		
	t_{P_max} (min)	T_S1a$_{P_max}$ (°C)	T_S2$_{P_max}$ (°C)	P_{max} (kN)	P_{max}/P_0	t_{cr} (min)	T_S1a$_{cr}$ (°C)	T_S2$_{cr}$ (°C)
CC273-30 ka	19.6	54	486	1426	1.25	36.5	144	717
SC220-30 ka	21.2	96	499	1511	1.34	32.2	150	688
CC194-30 ka	16.6	75	464	842	1.43	34.7	162	761
RC350-30 ka	20.6	102	421	1307	1.12	28.4	152	595
RC250-30 ka	19.5	104	446	1220	1.35	32.5	186	693
SC150-30 ka	14.7	59	405	772	1.45	24.5	137	662
EC320-30 ka	24.2	98	499	1393	1.43	42.5	205	754
CC273-110 ka	17.4	73	444	1911	1.67	35.6	152	740
SC220-110 ka	19.5	79	428	1974	1.75	32.3	150	685
CC194-110 ka	15.2	70	401	1035	1.76	31.3	154	727
RC350-110 ka	19.5	94	401	2041	1.75	32.0	181	639
RC250-110 ka	18.6	77	439	1769	1.96	29.5	160	658
SC150-110 ka	13.6	54	374	963	1.81	23.4	125	625
EC320-110 ka	20.5	55	444	2008	2.06	39.9	160	733

Table (2.1)

- CELLULAR BEAMS :

A perforated beam was installed in the furnace room. The total length and span of the beam were 5000 mm and 4300 mm respectively, and the length exposed to fire was 4000 mm. The circular opening had a diameter of 200 mm. The loads were placed using the 4-point method and these loads were suspended using hydraulic jacks. In , additional concrete blocks were placed on the beams to simulate floor slabs and provide his three-sided heating of as shown in Figure (3.a) [3].Intended to understand the fire performance of bare stainless steel girders, the girders were not rated fire rating. Oven temperature was measured using eight plate thermometers. The result was points. Graphical plot (3.b), recorded between time (min) and temperature (c), shows that the beam responded linearly to vertical midspan deflection up to about 24 min. shown. It had risen to over 800°C [3].

At the end of this part , it was concluded that load placement does not significantly affect the length of time an unprotected perforated stainless steel girder survives.And one of the most important results of the current part is that stainless steel girders significantly outperform carbon steel hollow section girders in terms of service life and deflection[3].



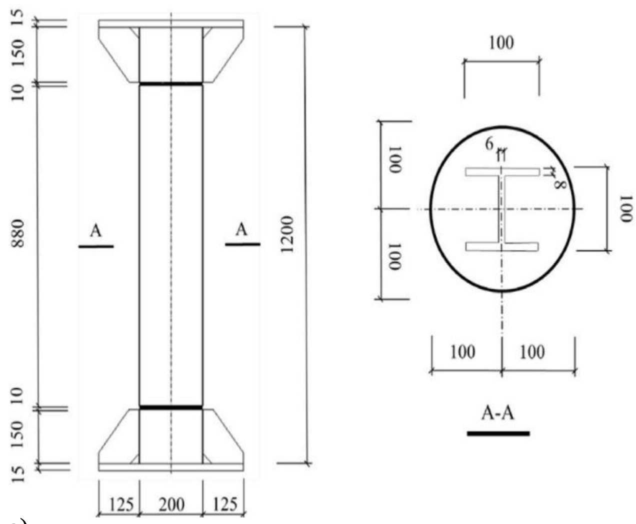
Fig(3.a)

Fig(3.b)

- <u>CIRCULAR TUBED STEEL REINFORCED CONCRETE COLUMNS (CTSRC) :</u>

For this purpose, a total of 6 specimens were made as shown in Figure (4.a).Outer steel pipe was cut in a circle so that he created two circumferential strips (10mm wide) 150mm apart from the edge of the specimen so that the load was fully transferred to the concrete core . Prior to testing, a 20 mm diameter hole was drilled in hisspecimen to mount a thermocouple. Then, concrete is poured into the hollow portionof the specimen. After the concrete had hardened, the holes were filled with a high-strength, non-shrinking grout. This tubular reinforced concrete column was placed on the structural drawing (4.b) [4] for further testing. The columns were then loaded. The facility was subjected to high temperature testing during loading using an electric furnace already installed.

After receiving the results, it can be seen that the temperature of the steel pipe is significantly lower than the furnace temperature for the first 25 minutes due to heat absorption by the concrete core. It was found that the temperature of the H steel generally depends on the temperature of the adjacent concrete and the flange temperature of the H steel is lower than the web temperature due to the distance from the outer surface of the specimen[4].



Fig(4.a)
Fig(4.b)

- <u>AUSTENITIC STAINLESS STEEL :</u>

The experimental tests reported here are two tests. First, a series of histensile tests were performed on specimens taken from his load stainless steel beams previously tested under fire conditions. Second, a set of virgin coupons (ie, not previously heated or tested) were subjected to various levels of elevated temperature and cooling procedures. The above coupon was obtained from a carrier exposed to a temperature of 785 °C at[5]. The beam was then naturally cooled in Furnace (CIF) . A total of 18 tensile bars were cut and tested. All the main properties measured during tensile testing are shown in Tables (5.1 & 5.2) as above. And the results obtained from both tests are presented in Tables (5.1-adjacent) and (5.2) [5]. In general, it has been observed that sample thickness, cooling method, and stress history have a significant effect on the remaining properties of the material after temperature exposure. It has also been shown that quenching specimens in water, rather than allowing the samples to cool naturally in air, improves strength retention [5] .

Test results of coupons out of heated beam.

Coupon	θ_{exp} (°C)	E (kN/mm^2)	E/E_{20}	$f_{0.2}$ (N/mm^2)	$f_{0.2}/f_{0.2,20}$	f_u (N/mm^2)	$f_u/f_{u,20}$
A1	838	209.2	1.11	262	0.99	616	0.91
A2	832	183.4	0.97	256	0.97	613	0.90
A3	838	189.5	1.00	261	0.99	615	0.90
B1	784	191.7	1.01	259	0.98	613	0.90
B2	775	194.5	1.03	278	1.05	621	0.91
B3	784	185.6	0.98	258	0.98	605	0.89
E1	642	191.4	1.01	263	1.00	607	0.89
E2	636	197.3	1.04	275	1.04	611	0.90
E3	642	195.1	1.03	285	1.08	613	0.90
F1	580	194.2	1.03	287	1.09	613	0.90
F2*	559	202.9	1.07	338	1.28	637	0.94
F3	580	196	1.04	287	1.09	612	0.90
C1	834	189.6	1.01	282	1.05	641	1.02
C2	834	182.1	0.97	282	1.05	638	1.02
C3	823	189	1.01	281	1.04	642	1.03
D1	794	182.4	0.98	368	1.37	657	1.05
D2	794	183.4	0.98	370	1.37	656	1.05
D3	779	206.7	1.11	376	1.40	662	1.06

Test results of the coupons from virgin plates.

t (mm)	θ_{exp} (°C)	Cooling method	E (kN/ mm^2)	E/E_{20}	$f_{0.2}$ (N/ mm^2)	$f_{0.2}/f_{0.2,20}$	f_u (N/ mm^2)	$f_u/f_{u,20}$	ε_u (%)	$\varepsilon_u/\varepsilon_{u,20}$	f_{fr} (N/ mm^2)	$f_{fr}/f_{fr,20}$	ε_{fr} (%)	$\varepsilon_{fr}/\varepsilon_{fr,20}$
8	20	–	186.8 (0.03)	1.00	269.1 (0.02)	1.00	625.7 (0.02)	1.00	59.0 (0.06)	1.00	366.6 (0.02)	1.00	67.2 (0.06)	1.00
8	250	CIW	200.9 (0.02)	1.12	272.1 (0.03)	1.01	632.3 (0.04)	1.01	59.1 (0.05)	1.00	457.7 (0.03)	1.25	66.1 (0.04)	0.98
8	500	CIW	210.4 (0.04)	1.18	269.3 (0.04)	1.00	632.1 (0.05)	1.01	59.6 (0.07)	1.01	441.7 (0.05)	1.20	67.1 (0.06)	1.00
8	750	CIW	184.1 (0.05)	1.03	255.3 (0.07)	0.95	639.3 (0.03)	1.02	55.9 (0.08)	0.95	497.2 (0.06)	1.36	62.6 (0.07)	0.93
8	750	CIA	203.7 (0.05)	1.14	237.7 (0.05)	0.88	613.6 (0.07)	0.98	57.5 (0.07)	0.97	454.4 (0.03)	1.24	64.3 (0.08)	0.96
14	20	–	188.9 (0.03)	1.00	264.3 (0.02)	1.00	679.9 (0.02)	1.00	63.7 (0.05)	1.00	523.9 (0.01)	1.00	72.4 (0.04)	1.00
14	250	CIW	198.9 (0.04)	1.05	287.8 (0.01)	1.09	684.7 (0.06)	1.01	48.9 (0.04)	0.77	532.8 (0.03)	1.02	57.1 (0.06)	0.79
14	500	CIW	188.9 (0.06)	1.00	281.8 (0.04)	1.07	697.0 (0.04)	1.03	63.9 (0.03)	1.00	548.3 (0.04)	1.05	72.8 (0.06)	1.00
14	750	CIW	186.6 (0.05)	0.99	263.8 (0.05)	1.00	705.8 (0.08)	1.04	61.3 (0.08)	0.96	582.1 (0.08)	1.11	69.5 (0.10)	0.96
14	750	CIA	218.2 (0.04)	1.16	257.8 (0.06)	0.98	693.7 (0.05)	1.02	63.2 (0.05)	0.99	542.5 (0.05)	1.04	71.7 (0.08)	0.99

Table(5.2)

III. CONCLUSIONS :

Along with the experimental tests , results & observations , this study includes the conclusions that are derived together from all the studies.

1. Long term exposure of structural components, that isfor passive structural elements, an increase in temperature reduces their technical properties and reduces the capacity of the entire structure.
2. Concrete fillers such as steel fiber reinforced concrete can increase the resistance provided by structural members. [1] Columns filled with steel fiber reinforced concretehave been found to have the highest structural fire resistance compared to concentrically compressedcolumns. This is due to the uniform distribution of steel fibers within the concrete core, increasing the tensile strength of the concrete coreand hence the bending strength .
3. As already mentioned, passive fire protection and hardening depend on the shapeand the cross section of the components. [2] The fire resistance of CFST Column was dependent on the cross-sectional geometry. (For the same stress level (30%), the fire resistanceranged from about 23 (square cylinder) to 43 minutes (elliptical cylinder)).
4. Load factor and eccentricity were two important parameters that greatly affected the fire resistance of columns [4] .

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