Construction and Building Materials 234 (2020) 117371

Contents lists available at ScienceDirect

Construction and Building Materials

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

Experimental fast-assessment of post-fire residual strength of reinforced concrete frame buildings based on non-destructive tests

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HIGHLIGHTS

- High correlation between decrease strength of RC frames and exposure fire time.
- Correlation between decrease strength of RC frames and variation of NDT parameters.
- Fast-assessment of post-fire residual strength of RC frame building using NDT.

ARTICLE INFO

Article history:
Received 25 June 2019
Received in revised form 18 October 2019
Accepted 23 October 2019

Keywords:
Post-fire residual strength
Non-destructive tests
Reinforced concrete building
Structural assessment

ABSTRACT

Assessment of the residual strength of reinforced concrete buildings subjected to fire is a problem that requires fast and sufficiently reliable resolution, necessary for the action of firefighters, forensic fire investigation, and structural assessment of post-fire condition of the building to take place. In all cases safety and integrity of firefighters and researchers can be at risk, and it is necessary to have rapidly and sufficiently reliable information in order to choose whether to enter freely, to enter with caution, or simply do not enter to the burned structure. This required prompt assessment gives no time or background to develop mathematical models of the structure and damage propagation. This work presents an experimental methodology for a fast assessment of post-fire residual strength of reinforced concrete frame buildings based on the high correlation between the loss of strength and non-destructive test results of frame concrete elements subjected to fire action.

1. Introduction

A reliable and fast estimation of the post-fire residual strength of reinforced concrete buildings is required in order to determine the damage state of the structure before firefighters and paramedics can go inside, as well as, a forensic fire investigation or an accurate structural assessment of post-fire condition can be address. In all cases safety and integrity of these professionals can be at risk [1], since there is no time to extract specimens of materials and perform laboratory tests to diagnose the state of the structure no accurately nor in an approximate manner. Additionally, an economic evaluation of the building damage is critical for the decision of repairing or demolishing the public or private infrastructure affected. Considering the high costs of replacing the burned nonstructural elements, equipment and the great expenses of sophisticated studies to determine the level of decrement of the structure strength, to have a simple and fast methodology based on prompt and low-cost testing procedure is convenient: in cases where the repair of the affected structure is not feasible it is possible to avoid unnecessary expenses in evaluation studies. Moreover, in cases where the repair is achievable, it allows not to dismiss the reparation of the structure due to uncertainty of performing more accurate, expensive, and sophisticated studies, since there will be preliminary information available on the level of damage caused by fire on the observed structure. As result of the extensive damage produced by fire in Valparaiso during April 2014, the Chilean Government will spend close to

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https://doi.org/10.1016/j.conbuildmat.2019.117371
0950-0618/

USD102,000,000 within 2014 and 2021 granting housing solution to the 3110 affected families, of which only 20 cases considered the repair of the burned building. Additionally, other USD408,000,000 will be spent on replacing burned public infrastructure by a new one [2]. In this context, it is of great important to have a simple methodology for structural assessment or evaluation of post-fire strength of buildings. Previous research have reported considerable information about forensic assessment of study cases regarding destructive and non-destructive material testing [3–8], and advanced mathematical-computational models, which results have not been completely generalized [9–12] and only some of them consider their applicability on real situations [13]. The objective of this paper is to present an experimental methodology for the fast-cheap initial assessment of post-fire residual strength of reinforced concrete frame buildings based on non-destructive testing.

2. Materials and methods

Eighteen reinforced concrete columns and beams (structural specimens) have been designed and built according to the Chilean and international current design codes [14–16] with the purpose of obtaining structural elements representative of a building reality. The concrete mixture used is shown in Table 1. The dimensions of specimens and the geometric characteristics of steel bars used are shown in Table 2. Steel reinforcement bars have a nominal yield and tensile strength of 280 MPa and 440 MPa, respectively.

Eighteen concrete cylinders with 150 mm diameter and 300 mm length were tested as reference. Three of them were subjected to compression test according to Chilean and international standards [17,18], where an average compressive strength at 28 days of 15.1 MPa, with a standard deviation of 6.2 MPa, was obtained.

The cylinders, columns, and beams were grouped into six identical sets of three elements each. The columns and beams were subjected to the following non-destructive tests (NDT) before and after fire tests: Rebound number or Esclerometric index (EI) using standard Schmidt test hammer and standard procedure [19,20]; and Ultrasonic pulse velocity [21,22] using a Pundit Lab® equipment, considering three different procedures: (a) direct (UPV-D), with the pulse emitter and pulse receiver located on opposite faces of the specimen; (b) indirect (UPV-I), with the pulse emitter and pulse receiver located on the same face of the specimen; and semidirect (UPV-S), with the pulse emitter and pulse receiver located on adjacent faces of the specimen.

An electric oven (Fig. 1a) was used to perform the fire tests of cylinders, columns, and beams. A controlled temperature between 700 and 1000 °C was used to represent the fire according to the literature [13] and current standard codes [23,24] (Fig. 1b). The fire tests of the six sets of elements had a duration of 0 (reference set with no fire), 30, 60, 90, 120 and 180 min according to the tested set [13,23,24]. The columns were exposed to the action of fire on the entire lateral surface (four sides of the columns), while the beams were exposed to heat directly on three of its four faces (its upper face was always protected by a concrete slab, Fig. 1c).

After the specimens cooled down slowly to room temperature (without water or other chemical substances), each of them was subjected to the aforementioned non-destructive tests. Later, the strength of cylinders and columns was measured by means of destructive uniaxial compression tests, while the strength of beams was measured by a flexural test (loading with two load points placed at thirds along the beam span, four-point bending tests [25,26]). Each specimen is identified as "Cyl", "Col" or "Beam" for concrete cylinders, RC columns and RC beams, respectively, followed by the duration of exposure to the fire action in minutes, and finally, a correlative number for listing the specimens.

The compression cylinder test was performed according to Chilean and international current codes [17,18] using a 10 mm thick and 160 mm diameter Shore 60 elastomeric bearings at top and bottom faces of the cylinder, attaining a complete contact between the concrete cylinder faces with the reaction frame and the load application system. The compression load was applied using controlled-deformation with a hydraulic cylinder of 1000 kN of capacity and a nominal load velocity of 0.1 mm/s. Additionally, a displacement transducer was installed to measure the compression strain at the 200 mm specimen length, as shown in Fig. 2a and b.

The columns were subjected to compression in a similar scheme as the cylinders [17,18]. The compression load was applied using controlled-deformation with a hydraulic cylinder of 1000 kN of capacity and a nominal load velocity of 0.1 mm/s. In both ends of the columns, 10 mm thick Shore 60 elastomeric bearings were used to ensure complete contact between the column and the reaction frame, as well as, between the column and the hydraulic cylinder which applied the compression load. Two displacement transducers were used to measure the compression strain at 400 mm of length, in two opposite sides of the column. The test scheme can be observed in Fig. 3a and b.

Four-points bending tests [25,26] were performed on the beam specimens, applying two point loads placed at thirds of the beam span as depicted in Fig. 4a and b, where the test setup is illustrated. The load was applied using controlled displacement with a hydraulic cylinder with 300 kN of capacity and velocity of approximately 1 mm/s [1,13,25,26]. Two displacement transducers located at each side of the beam midspan section were used to measure the deflection obtained.

Results of the compression tests of cylinders were used as reference for the concrete compressive strength and the compression behavior of burned concrete. In columns and beams, the applied load, the deformation, and the failure modes of the structural frame elements were recorded and analyzed.

Table 1
Proportions of concrete mixture [1,13].

Material	Cement	Sand	Gravel	Water
% (kg/m^3)	18.2 (436)	36.4 (873)	36.4 (873)	9.1 (218)

Table 2
Dimensions and reinforcement bars of specimens [1,13].

Specimen	Length (mm)	Width (mm)	Height (mm)	Upper bars	Lower bars	Stirrups
Columns	600	200	250	2ϕ8mm	2ϕ8mm	ϕ8@50 mm
Beams	650	150	250	2ϕ6mm	2ϕ8mm	ϕ8@100 mm

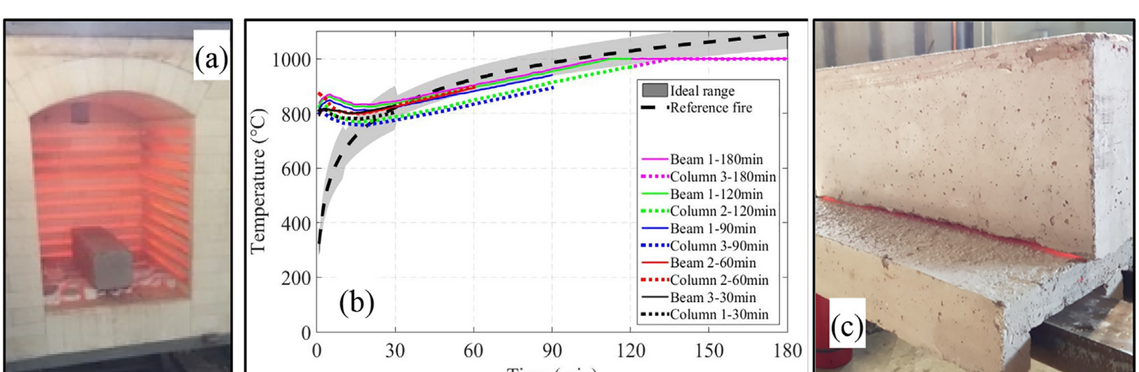

Fig. 1. Fire experimental setup: (a) Used electric oven; (b) Standard fire curve and measure temperature curves; (c) Fire setup for beams.

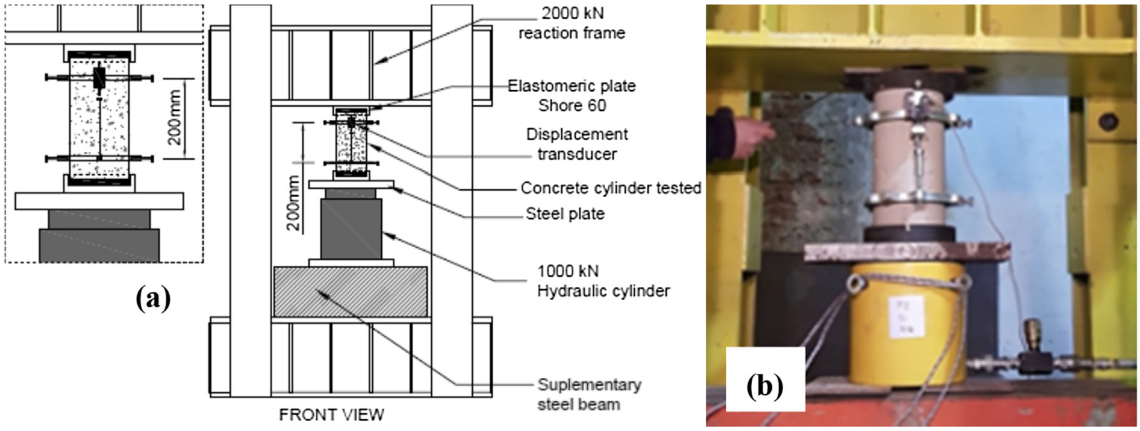

Fig. 2. Compression test setup of cylinders: (a) Scheme of assembly and instrumentation; (b) Photography of assembly of compression test.

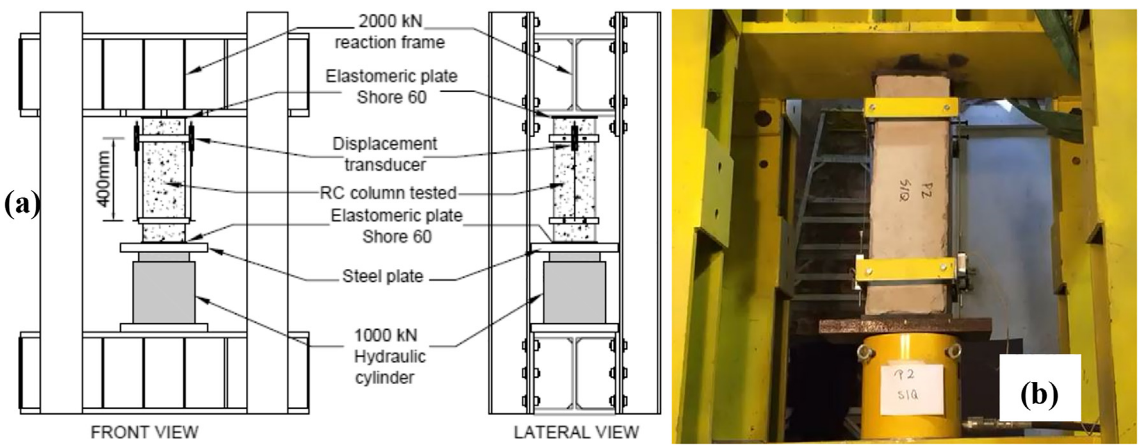

Fig. 3. Compression test setup of columns: (a) Scheme of assembly and instrumentation; (b) Photography of assembly of column test.

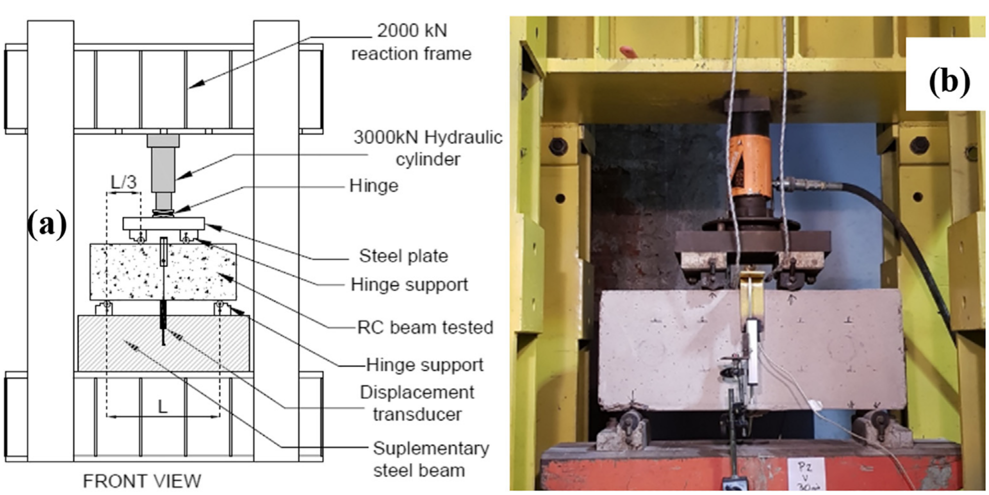

Fig. 4. Third-point loading test setup of beams: (a) Scheme of assembly and instrumentation; (b) Photography of assembly of beam test.

3. Results and discussion

For each concrete cylinder subjected to different duration of exposure to fire action, Table 3 shows the maximum compressive strength (σ_{max}) and the ultimate strain (ε) obtained after cooled down to room temperature. Some values of ultimate strain were missed or not reported because the deformation measurement was distorted in advanced states of the tests (close to maximum strength) due to rotation or falling of the transducer. These cases have been indicated as not measured (NM) in Table 3. The average and the standard deviation (SD) for each group are also presented in Table 3. Fig. 5a shows the maximum measured compression

Table 3
Compressive strength and strain for cylinders.

Specimen	σ_{max} (MPa)	ε (%)
Cyl-00–01	11.35	0.20
Cyl-00–02	11.70	0.21
Cyl-00–03	22.22	NM
Average	**15.09**	**0.21**
SD	**6.18**	**0.00**
Cyl-30-01	11.65	NM
Cyl-30-02	8.08	0.35
Cyl-30-03	7.43	0.51
Average	**9.05**	**0.43**
SD	**2.27**	**0.11**
Cyl-60-01	4.08	0.15
Cyl-60-02	4.87	0.08
Cy-60-03	4.79	0.32
Average	**4.58**	**0.18**
SD	**0.43**	**0.12**
Cyl-90-01	2.28	NM
Cyl-90-02	2.14	0.28
Cy-90-03	2.48	0.49
Average	**2.30**	**0.38**
SD	**0.17**	**0.15**
Cyl-120-01	1.44	0.69
Cyl-120-02	1.91	1.55
Cy-120-03	1.82	1.50
Average	**1.72**	**1.25**
SD	**0.25**	**0.48**
Cyl-180-01	0.81	0.87
Cyl-180-02	0.69	0.83
Cyl-180-03	0.76	1.06
Average	**0.76**	**0.92**
SD	**0.06**	**0.12**

NM: not measured/failed transducer

strength of each concrete cylinder subjected to the fire action, the average and the area enclosed by +/– one standard deviation. Additionally, the most representative stress–strain (σ-ε) relationships of cylinders for different times of fire exposure are shown in Fig. 5b. All the data and the stress–strain curves show an important decrease of the maximum compressive strength, as well as a decrease of the concrete stiffness when the fire exposure time increased.

Table 4 shows for each RC column subjected to different durations of fire exposure the maximum compressive load and ultimate strain (ε) obtained after the specimens cooled down to room temperature. Some values of ultimate strain were missed or not reported because the deformation measurement was distorted due to rotation or falling of one or both deformation transducers at advanced states of the tests. These cases have been indicated

Table 4
Compression strength and strain for RC columns subjected to fire action.

Specimen	Maximum load (kN)	Average (kN)	SD (kN)	ε (%)	Average (%)	SD (%)
Col-00-1	538.80	500.08	33.66	NM	0.04	–
Col-00-2	483.69			0.04		
Col-00-3	477.76			NM		
Col-30-1	379.53	355.93	43.63	1.08	1.30	0.73
Col-30-2	305.58			2.11		
Col-30-3	382.67			0.69		
Col-60-1	336.69	415.73	69.05	0.80	5.38	5.13
Col-60-2	464.38			4.42		
Col-60-3	446.11			10.93		
Col-90-1	303.49	311.28	6.82	11.09	7.69	6.23
Col-90-2	314.19			0.50		
Col-90-3	316.15			11.48		
Col-120-1	217.99	227.54	18.26	3.14	5.34	5.21
Col-120-2	216.04			11.29		
Col-120-3	248.60			1.59		
Col-180-1	186.06	179.80	8.64	11.27	10.82	0.39
Col-180-2	169.95			10.57		
Col-180-3	183.40			10.62		

NM: not measured/failed transducer.

in Table 4 as not measured (NM). Additionally, the average and the standard deviation (SD) of each group are presented. Fig. 6a shows the maximum compression load of each column subjected to the fire action, including the average and the area enclosed by +/– one standard deviation. Furthermore, the most representative load-deformation relationships of column specimens for different durations of exposure are shown in Fig. 6b. From Table 4, as observed with the cylinder tests, a significant decrease of maximum compressive load of columns is obtained when the duration of fire exposure increased. Consistent with the decrease of strength observed in Fig. 6a, Fig. 6b depicts a substantial decrease of the axial stiffness of the columns as the fire exposure increase.

Table 5 shows the NDT measurements for each RC column, before exposed to fire and after the specimens cooled down to room temperature, including the average and standard deviation of each parameter. In all cases, after of the fire exposure of columns a significant decrease of NDT measurements is observed with respect to the NDT results before. This decrease grows as the fire exposure duration increases.

Table 6 shows the maximum shear load (V), deflection (Δ), the length between supports (L) (see Fig. 4a), the relative deflection (Δ/L), and maximum bending moment (M) for each RC beam tested after cooled down to room temperature. The average and the standard deviation for each group are presented. Fig. 7a shows

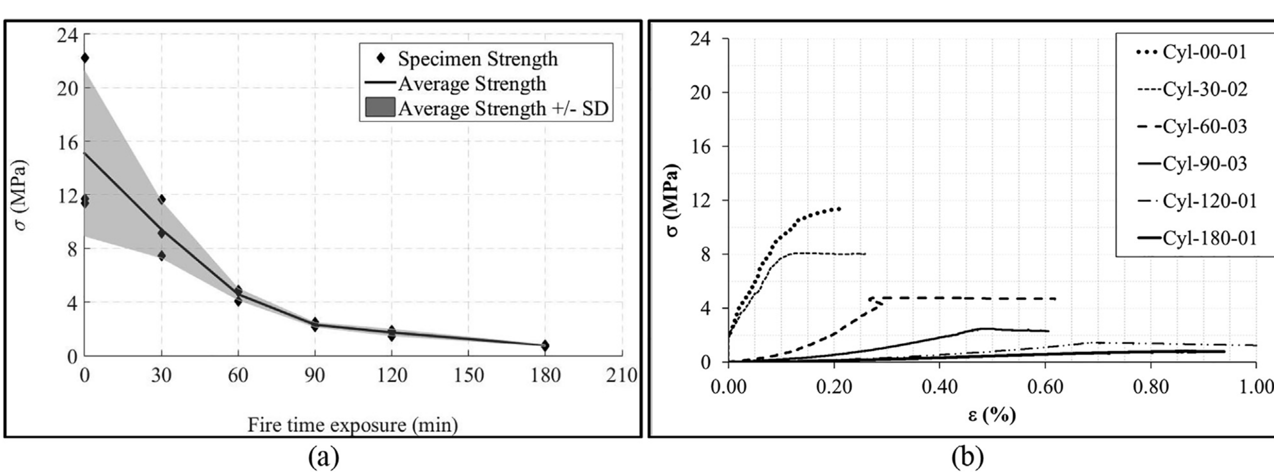

Fig. 5. (a) Relationship between fire time exposure and cylinder compression strength and (b) Observed stress–strain behavior of concrete cylinders subjected to fire action.

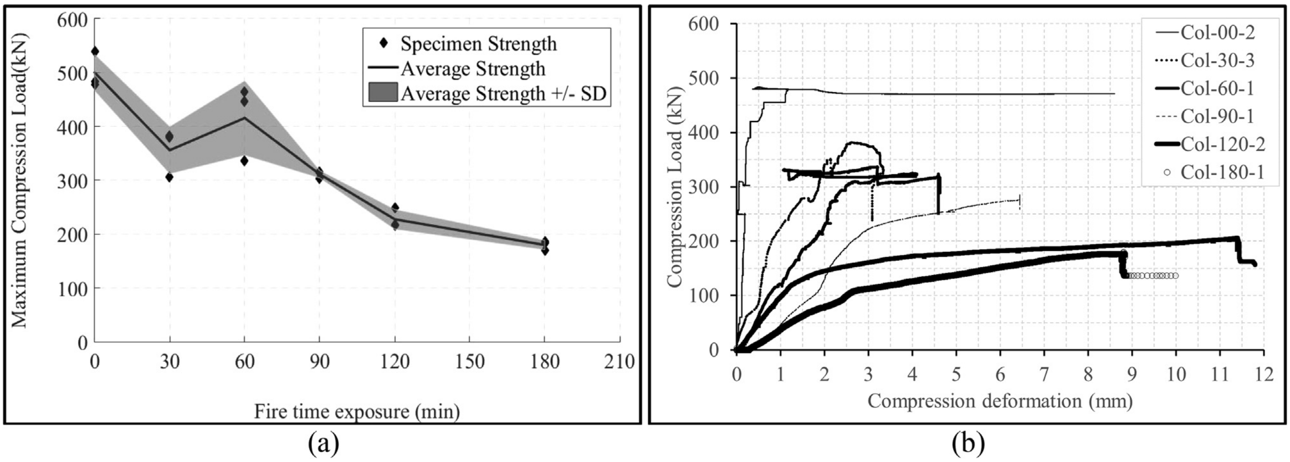

Fig. 6. (a) Maximum compression load for RC columns subjected to fire action and (b) Axial load-deformation measured of RC columns subjected to fire action.

Table 5
NDT measurements RC columns after and before to fire action.

Specimen ID	EI		UPV_D		UPV_S		UPV_I	
	Before	After	Before	After	Before	After	Before	After
Col-00-1	30.4		4043		4248		3221	
Col-00-2	32.0		4205		4291		3582	
Col-00-3	29.1		4112		4076		3686	
Average	**30.5**		**4120**		**4205**		**3496**	
SD	**1.5**		**81**		**114**		**244**	
Col-30-1	30.3	25.4	3947	2428	4113	1589	3700	1640
Col-30-2	30.7	25.0	3845	2033	4171	2191	3844	898
Col-30-3	28.4	24.8	3953	2283	4340	1967	3668	1261
Average	**29.8**	**25.1**	**3915**	**2248**	**4208**	**1916**	**3737**	**1266**
SD	**1.2**	**0.3**	**61**	**200**	**118**	**304**	**94**	**371**
Col-60-1	26.3	20.1	3759	1649	3921	1482	2583	524
Col-60-2	27.6	22.8	3757	1712	4104	1091	3776	501
Col-60-3	28.4	20.9	3851	1776	3842	1941	3016	1175
Average	**27.4**	**21.3**	**3789**	**1713**	**3956**	**1505**	**3125**	**733**
SD	**1.1**	**1.4**	**54**	**63**	**134**	**426**	**604**	**383**
Col-90-1	27.4	19.8	3810	1043	3969	324	3225	215
Col-90-2	25.2	18.8	4063	1681	3936	1075	3861	348
Col-90-3	30.1	19.1	4118	1069	4188	391	2858	67
Average	**27.6**	**19.2**	**3997**	**1264**	**4031**	**597**	**3315**	**210**
SD	**2.5**	**0.5**	**164**	**361**	**137**	**415**	**508**	**141**
Col-120-1	25.3	15.9	3686	1018	3925	334	3045	752
Col-120-2	26.6	17.8	3953	1030	4068	469	2263	204
Col-120-3	28.6	17.2	3798	962	4108	358	3732	591
Average	**26.8**	**17.0**	**3812**	**1003**	**4034**	**387**	**3013**	**516**
SD	**1.6**	**1.0**	**134**	**36**	**96**	**72**	**735**	**282**
Col-180-1	28.4	14.5	3617	577	4055	151	1830	76
Col-180-2	27.1	13.6	3746	468	3718	375	2713	52
Col-180-3	26.5	14.6	3703	785	4064	333	3333	59
Average	**27.3**	**14.2**	**3689**	**610**	**3946**	**286**	**2625**	**62**
SD	**1.0**	**0.6**	**66**	**161**	**197**	**119**	**755**	**12**

the maximum shear load of each beam subjected to the fire action, including the average and the area enclosed by +/− one standard deviation. As indicated for the cylinders and columns, for the beams is possible to appreciate a decrease of maximum shear load and bending moment with the increment of duration of fire exposure. On the other hand, the maximum deflection increases with increase of fire exposure duration.

The most representative shear load–deflection (V-Δ) relationships of beam specimens for different durations of exposure to fire action are shown in Fig. 7b. Consistent with the decrease of strength observed into Fig. 7a, in Fig. 7b can be appreciated an important decrease of the flexural stiffness of the beams as the duration of fire exposure increases.

Table 7 shows the NDT measurements for each RC beams, before fire exposure and after the specimens cooled down to room temperature, including the average and standard deviation for each parameter. As observed with the NDT measurements of columns, in all cases a substantial decrease of NDT measurements is observed after the exposure of beams to fire. This decrease is more significant as the fire exposure duration increases.

The failure modes in compression of the columns are characterized by greater and faster loss of cover and crumbling of the unconfined concrete as the time of exposure to fire increases (see Fig. 8a). In the beams, the failure mode is characterized by a decrease of both shear and compressive strength near the supports, mainly due to the loss of anchorage of the reinforcement

Table 6
Maximum shear load, deflection and bending moment for RC beams subjected to fire action.

Specimen	V (kN)	Δ (mm)	L (mm)	Δ/L (%)	M (kN·mm)
Beam-00-1	76.74	4.51	560	0.80%	7162
Beam-00-2	85.56	3.10	560	0.55%	7985
Beam-00-3	74.40	4.55	560	0.81%	6944
Average	**78.90**	**4.05**		**0.72%**	**7364**
SD	**5.88**	**0.67**		**0.12%**	**448**
Beam-30-1	59.11	3.80	560	0.68%	5517
Beam-30-2	62.51	5.75	560	1.03%	5834
Beam-30-3	71.19	7.09	510	1.39%	6051
Average	**64.27**	**5.55**		**1.03%**	**5801**
SD	**6.23**	**1.35**		**0.29%**	**219**
Beam-60-1	37.00	4.27	510	0.84%	3145
Beam-60-2	58.16	9.79	510	1.92%	4944
Beam-60-3	56.67	12.31	510	2.41%	4817
Average	**50.61**	**8.79**		**1.72%**	**4302**
SD	**11.81**	**3.36**		**0.66%**	**820**
Beam-90-1	48.10	10.86	510	2.13%	4089
Beam-90-2	43.34	6.51	510	1.28%	3684
Beam-90-3	38.96	6.76	510	1.33%	3312
Average	**43.47**	**8.04**		**1.58%**	**3695**
SD	**4.57**	**1.99**		**0.39%**	**317**
Beam-120-1	21.68	9.75	510	1.91%	1843
Beam-120-2	35.17	11.19	510	2.19%	2989
Beam-120-3	37.06	13.71	510	2.69%	3150
Average	**31.30**	**11.55**		**2.26%**	**2661**
SD	**8.39**	**1.64**		**0.32%**	**582**
Beam-180-1	18.33	18.25	510	3.58%	1558
Beam-180-2	16.33	11.83	510	2.32%	1388
Beam-180-3	16.04	10.50	510	2.06%	1363
Average	**16.90**	**13.52**		**2.65%**	**1437**
SD	**1.25**	**4.14**		**0.01**	**106**

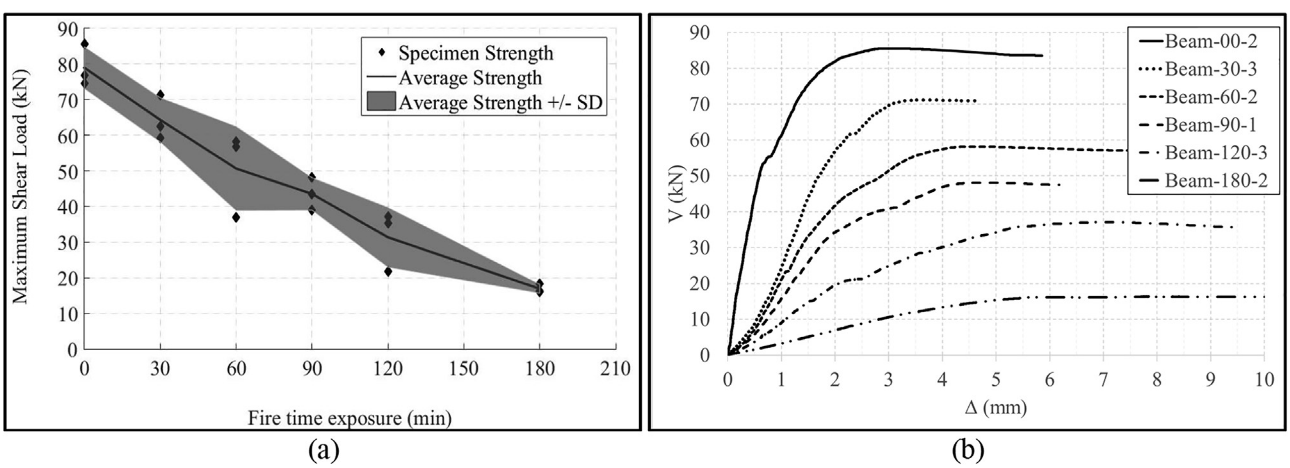

Fig. 7. (a) Maximum load in RC beams subjected to fire action and (b) Response of RC beams subjected to fire action and.

and effectiveness of the shear reinforcement due to the concrete strength loss (see Fig. 8b and c), inducing a brittle failure with a significant uncertainly in the deflection measured due to that the vertical displacement at supports did not measured, as a consequence, the subsequent analyzes only include a maximum strength. This failure is more significant as the time of exposure of fire increases.

As mentioned previously, a decrease of strength in columns and beams with increase of fire exposure time was observed, which allows to infer a correlation between the loss of strength and the time of exposure exists [13]. The residual strength is defined as R/R_0, the ratio between the measured strength of each specimen (R) and the average strength of the corresponding unburned specimen (R_0). Similarly, the variation of NDT results is defined as the ratio between the NDT measurement of each specimen (EI, UPV_D, UPV_I, UPV_S) and the average value of the NDT measurement for the non-fire affected specimen set (EI_0, UPV_D_0, UPV_I_0, UPV_S_0). With these definitions of residual strength and variation of NDT results, it is possible to attenuate the scale effects because each element is compared with itself before and after its exposure to the action of fire. Therefore, the effect of scale has not been considered in this study, however, in future research should be studied to have more generalized results.

Fig. 9 shows plots of the variation of NDT results with the fire exposure duration for columns and beams. In both cases, and for all NDT parameters, a high correlation between the NDT variations and the duration of exposure to the fire action exists.

Furthermore, Figs. 10 and 11 show plots of the residual strength (R/R_0) and the variations of NDT results for columns and beams, respectively. The EI and the UPV-D are the NDT parameters that produce the best estimation of the residual strength of column strength; Fig. 10a and b show the best fit of logarithmic functions

Table 7
NDT measurements RC beams after and before to fire action.

Specimen ID	EI		UPV_D		UPV_S		UPV_I	
	Before	After	Before	After	Before	After	Before	After
Beam-00-1	30.0		4025		4345		3706	
Beam-00-2	26.0		3968		4031		3544	
Beam-00-3	25.0		3807		4303		3551	
Average	**27.0**		**3934**		**4226**		**3600**	
SD	**2.6**		**113**		**171**		**91**	
Beam-30-1	29.8	28.0	3850	2897	4271	2158	3371	1002
Beam-30-2	30.8	25.7	3896	2298	4045	2098	3565	529
Beam-30-3	28.6	26.4	3961	2348	4081	2437	3223	694
Average	**29.7**	**26.7**	**3902**	**2515**	**4132**	**2231**	**3386**	**742**
SD	**1.1**	**1.2**	**56**	**332**	**121**	**181**	**171**	**240**
Beam-60-1	29.8	24.4	3886	1934	4010	1235	2950	208
Beam-60-2	30.8	25.7	3954	2147	4353	1398	3514	501
Beam-60-3	28.6	26.2	3846	1911	3924	1721	3562	454
Average	**29.7**	**25.4**	**3895**	**1997**	**4096**	**1452**	**3342**	**387**
SD	**1.1**	**0.9**	**55**	**130**	**227**	**247**	**341**	**157**
Beam-90-1	30.4	24.1	4008	1955	4569	1599	3563	515
Beam-90-2	33.6	20.7	3846	1681	4670	1075	3706	348
Beam-90-3	29.8	21.5	3850	1427	4010	1071	3565	277
Average	**31.3**	**22.1**	**3901**	**1688**	**4416**	**1248**	**3611**	**380**
SD	**2.0**	**1.8**	**92**	**264**	**356**	**304**	**82**	**122**
Beam-120-1	31.3	17.8	3769	1082	4092	413	2862	61
Beam-120-2	32.3	22.9	3643	1413	4819	863	3798	341
Col-120-3	28.6	21.7	4036	1320	4670	910	3692	129
Average	**30.7**	**20.8**	**3816**	**1272**	**4527**	**729**	**3450**	**177**
SD	**1.9**	**2.7**	**201**	**171**	**384**	**274**	**512**	**146**
Beam-180-1	29.7	15.6	3643	838	4670	545	3563	192
Beam-180-2	29.5	15.1	4302	435	4406	472	3371	105
Beam-180-3	31.1	14.9	3986	711	4275	273	3583	59
Average	**30.1**	**15.2**	**3977**	**661**	**4450**	**430**	**3506**	**119**
SD	**0.9**	**0.4**	**330**	**206**	**201**	**141**	**117**	**68**

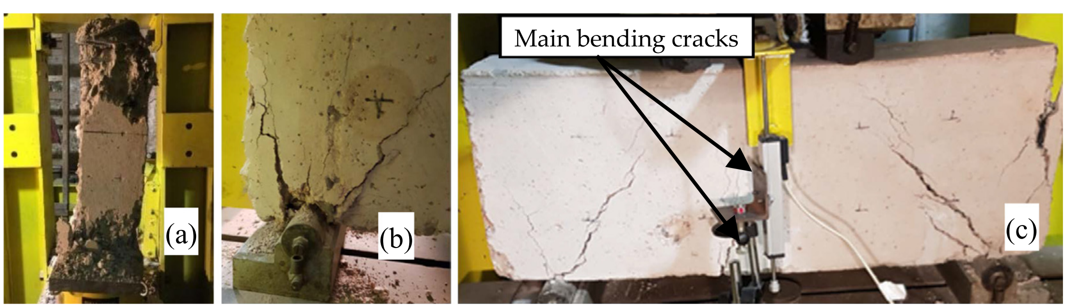

Fig. 8. Observed failure modes: (a) Compression failure of burned reinforce concrete column: loss and crumbling of concrete cover, while the confined concrete core remains relatively undamaged; (b) shear-bending failure of burned concrete beam: the shear and crushing failure at the supports become more significant as the duration of exposure to fire increases. (c) main flexural cracks at the center of the beam are less important in beams with duration of fire exposure increase due to that the crushing and shear failures occur first.

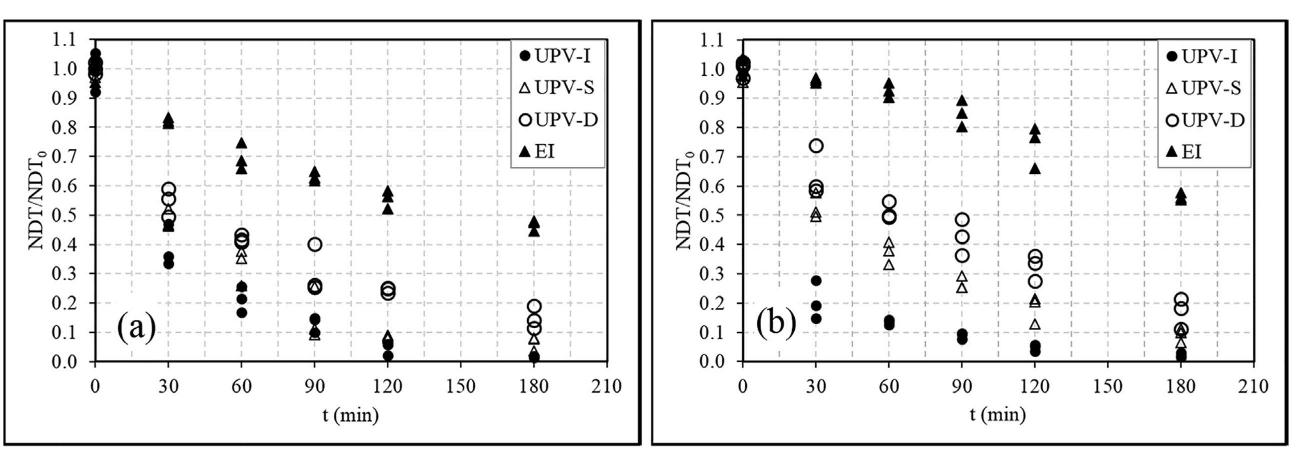

Fig. 9. Variation of NDT parameters: (a) columns; (b) beams.

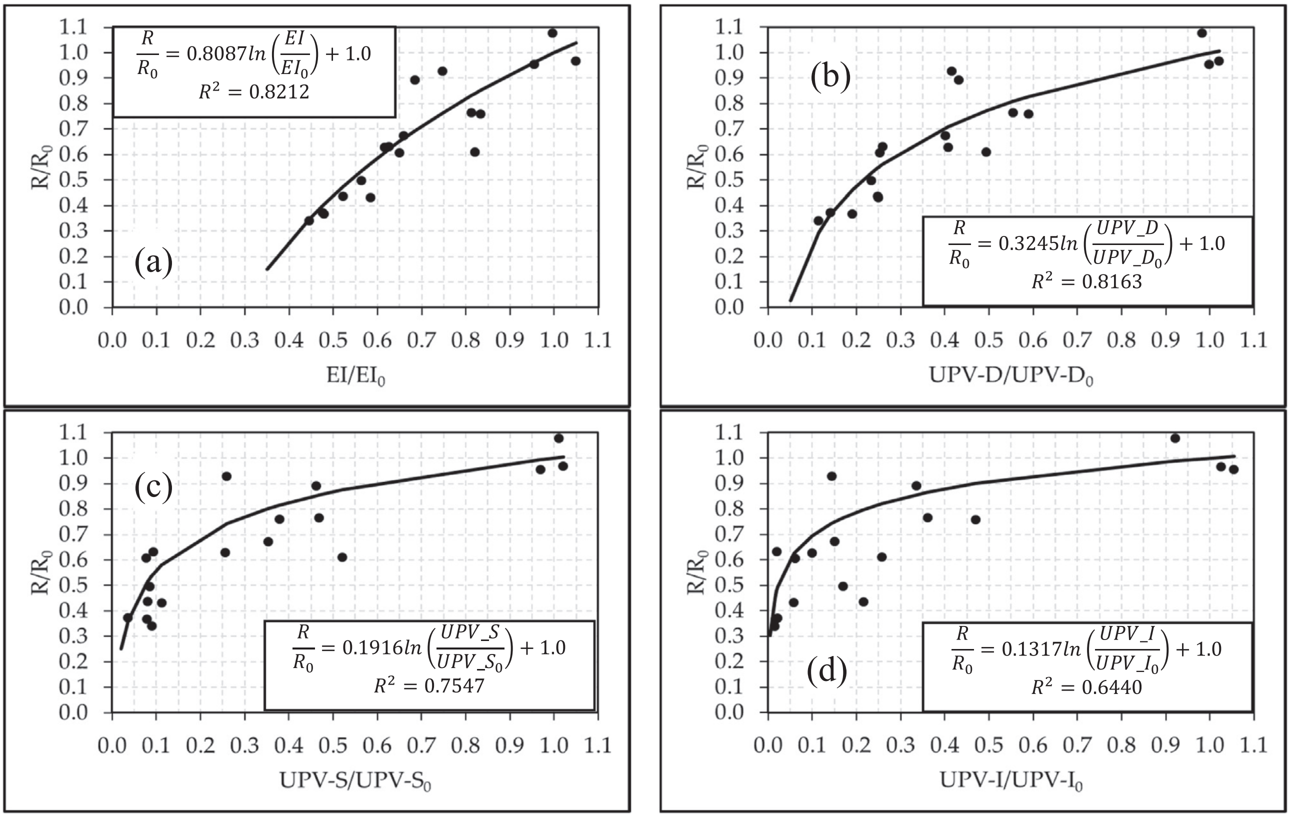

Fig. 10. NDT variation versus strength variation and correlation function in columns according to: (a) EI; (b) UPV–Direct; (c) UPV–Semidirect and (d) UPV–Indirect.

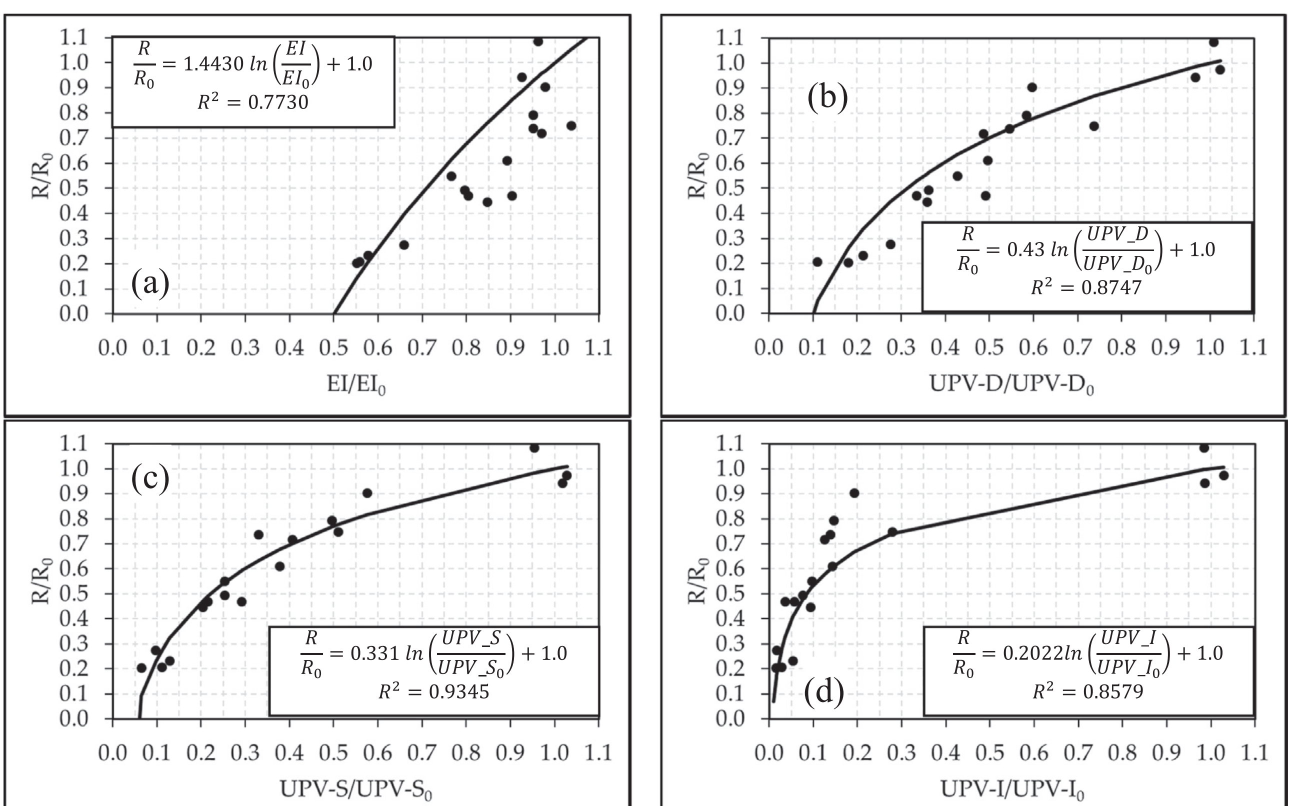

Fig. 11. NDT variation versus strength variation and correlation function in beams according to: (a) EI; (b) UPV–Direct; (c) UPV–Semidirect and (d) UPV–Indirect.

Table 8
Ranges of NDT variations according with safety criterion based on strength variation.

	Columns				Beams			
R/R_0	>0.80	<0.80	>0.60	<0.60	>0.80	<0.80	>0.60	<0.60
Structural condition	Safety	Caution		Danger	Safety	Caution		Danger
EI/EI_0	>0.78	<0.78	>0.61	<0.61	>0.87	<0.87	>0.76	<0.76
UPV_D/UPV_D_0	>0.54	<0.54	>0.29	<0.29	>0.63	<0.63	>0.39	<0.39
UPV_S/UPV_S_0	>0.35	<0.35	>0.12	<0.12	>0.55	<0.55	>0.30	<0.30
UPV_I/UPV_I_0	>0.22	<0.22	>0.05	<0.05	>0.30	<0.37	>0.14	<0.14

with a R^2 coefficients greater 0.8. Moreover, the UPV-S and the UPV-I demonstrated to be good predictors of the residual strength ratio of beams (see Fig. 11c and d). In all cases the logarithmic curve was selected because it was the best fit among low order functions.

According to the safety criterion proposed in [27–29], if the residual resistance ratio is larger than 0.80, the element under study have a low probability of collapse. On the other hand, The European standard EN 1992-1-2 [29] proposes a simplified method to calculate the required strength in fire situation (residual strength), that considering its recommendations and a variety of relationships and combinations of permanent and non-permanent loads, estimates residual strength ratios ranging between 0.55 and 0.70. As simplification, it is proposed a strength ratio of 0.60 to be the minimum accepted strength ratio of a fire-damaged element.

Consequently, considering the above failure criterion based on residual strength for compression [27,29] and for shear-bending [28,29] elements, in addition to the normalized relationship between the results of NDT and residual strength, it is possible to present a simplified experimental procedure to rapidly perform an approximate assessment of the condition of the burned structure. The methodology consists of (a) To measure the NDT parameter selected in areas of RC elements affected by fire and in corresponding RC elements not affected by fire and obtain a ratio of the NDT parameter; (b) To consider the type of frame element (column or beam) and the NDT parameter ratio selected to calculate the strength ratio of the element using the correlation functions proposed; (c) Finally, if the calculated strength ratio is over 0.80, the element under study have a low possibility of collapse, if the strength ratio is between 0.80 and 0.60 the possibility of collapse is moderate, and if the strength ratio is less than 0.60 the possibility of collapse increases. It is important to consider that the limit values of the strength ratio, although based on the literature, are arbitrary and can be selected by the user. Table 8 shows NDT parameter ratios calculated from the correlation functions for the ranges of strength ratios proposed.

An intuitive color code has been used in Table 8 to facilitate the use and interpretation of the method. From here, it is possible to appreciate the simplicity and fast application of the methodology described for a rapid assessment of post-fire residual strength of reinforced concrete frame buildings based on non-destructive tests. However, the proposed method has uncertainty and limitations for its application. The experimental data used for its calibration is limited and does not consider in depth scale effects, shape of the element (relationship between exposed surface and volume of the RC element), amount of steel reinforcement, partial exposition to the fire action, cooldown method, spatial variation of NDT parameters, among others. The definition of the strength ratio (R/R_0) and the NDT parameter ratio (NDT/NDT_0) as the comparison of the parameter under study into the same structural specimen before and after the exposure to the fire action, minimize the influence of the previously mentioned effects in laboratory conditions. For the conditions in an actual building subject to fire, these effects cannot be considered because it is not possible to control, for example, the actual exposition to fire, the cooling down process, the steel distribution. Therefore, the proposed method is only a first approach for a rapid and approximate estimation of post-fire residual strength and structural condition of the RC elements, using low cost and easy to apply non-destructive tests, providing information for short-term safety purposes. Given that the method is simple and that the user can customize the safety criteria, complement the experimental data base, change the correlation functions, and consider other multiple effects (as cooldown method [12], spatial variability of NDT parameters [30–32]) using experimental data and/or numerical models [27], it would be easy to include future improvements of the method.

4. Conclusions

An extensive experimental program has been developed including the design and construction of eighteen RC columns, RC beams, and control concrete cylinder specimens. Fifteen of these eighteen cylinders, columns and beams were subjected to a simulated fire in laboratory during controlled time.

The observed behavior of the structural elements to the fire action is consistent with the observed behavior of cylindrical concrete specimens subjected to the same action. In all cases the strength and stiffness decrease with increase of fire exposure duration.

All RC specimens were subjected to NDT (EI and UPV) before and after the fire action. Finally, strength tests were made for each specimen. The correlation between the variation of the NDT results (or NDT parameter ratios) and the residual strength (or strength ratio) for different time of fire exposition was analyzed and empirical correlation functions with high a determination coefficient (R^2) were proposed. Using this correlation functions a simple and inexpensive methodology for the fast assessment of post-fire residual strength of reinforced concrete frame buildings is presented. This methodology, which only need one or two instruments for NDT and simple measurements, enables the evaluation of structural damage state of the building. This information is critical to establish secure work conditions for firefighters, forensic researchers, and helps define the future actions over the burned reinforced concrete building.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Acknowledgments

Financial support for the construction and testing of specimens was provided by Bomberos de Chile. The equipment for the fire, non-destructive and destructive test was provided for Pontificia Universidad Católica de Valparaíso. The scholarship for the doctoral research of the first author was provided by CONICYT, the national research council of Chile

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