Annual Concrete Seminar 2019

Different Methods to Improve Performance of Concrete under Fire, and to Quantify Residual Properties of Fire Damaged Concrete in Buildings

by

Ir Prof. Y.L. WONG

DipCE, MEng, PhD, FHKIE, FHKCI, RPE, RI

Department of Civil Engineering

Chu Hai College of Higher Education

Email: ylwong@chuhai.edu.hk

Mobile Phone No.: +852 64969170

Garley Building Fire in Hong Kong

20 Nov. 1996: Casualties: 40 deaths

Building after Fire

Buildings after Fire

Temperature	Strength reduction factors
20 °C	1.00
100 °C	1.00
200 °C	0.95
300 °C	0.85
400 °C	0.75
500 °C	0.60
600 °C	0.45
700 °C	0.30
800 °C	0.15
900 °C	0.08
1000 °C	0.04
1100 °C	0.01
1200 °C	0.00

Strength Reduction of Concrete Under Elevated Temperatures

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Causes of damages of concrete under fire

- Thermal mismatch (different expansion coefficients between paste matrix and aggregates)
- Presence of micro-cracks
- Thermal gradient
- Thermal decomposition of hydrates (CSH and CH)
- **Concrete spalling (due to high pore water pressure developed from free water in concrete)**

Decomposition of hydrates

- <105^0C Evaporation of gel/capillary (unsealed) water.
- >105^0C, Dehydration/decomposition of CSH and CH
  CSH into β-C_2S, β-CS and H
  CH into C and H
- 500^0C, 70% of dehydration of CSH completed
- 850^0C 100% of dehydration of CSH completed
- Decomposition of CH starts at 400^0C and completed at 600^0C

Concrete Spalling

High strength concrete (HSC) with granite aggregates fully saturated with water, Heating rate = 2℃/minute, spalled at about 450℃

Possible Solutions to Minimize Concrete Cover Spalling Suggested by BSEN 1992-1-2: 2004

- 1: The provision of protective layers for which it is demonstrated that no spalling of concrete occurs under fire exposure.

- 2: The inclusion of more than 2 kg/m^3 of monofilament propylene fibres in the concrete mix.

- 3: The provision of a reinforcement mesh with a nominal cover of 15 mm, in which the mesh should have wires with a diameter $\geq$ 2 mm with a pitch $\leq$ 50 x 50 mm, and the nominal cover to the main reinforcement should be $\geq$ 40 mm.

- 4: The provision of a type of concrete for which it has been demonstrated that no spalling of concrete occurs under fire exposure.

4.3.1 Prevention of spalling in high strength concrete

4.3.1.1 General requirement

For high strength concrete, the reduction of strength and associated risk of spalling at elevated temperature shall be taken into account. The content of silica fume if used in high strength concrete should not exceed 6% by weight of the total cementitious content. Pfa and ggbs if used in high strength concrete should comply with the requirements given in clause 4.2.5.5; there is no additional requirement or restriction on the use of pfa or ggbs as they are conducive to the prevention of spalling in high strength concrete.

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4.3.1.2 Methods to reduce risk of concrete spalling

At least one of the following methods should be provided.

(a) **Method A:** A reinforcement mesh with a nominal cover of 15mm. This mesh shall have wires with a diameter ≥ 2mm with a pitch ≤ 50 x 50mm. The nominal cover to the main reinforcement shall be ≥ 40mm; or

(b) **Method B:** Include in the concrete mix not less than 1.5 kg/m^3 of monofilament propylene fibres. The fibres shall be 6 – 12 mm long and 18 – 32 μm in diameter, and shall have a melting point less than 180°C; or

(c) **Method C:** Protective layers for which it is demonstrated by local experience or fire testing that no spalling of concrete occurs under fire exposure; or

(d) **Method D:** A design concrete mix for which it has been demonstrated by local experience or fire testing that no spalling of concrete occurs under fire exposure.

For high strength concrete exceeding C80, at least one fire test should be carried out to demonstrate that the main reinforcing bars of a structural member shall not be exposed during the design fire resistance rating. The test specimen should have moisture content not less than the highest moisture content that the structure may attain during its working life.

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So Uk Fire Test: Objectives

To conduct a compartment real fire test to verify/study

1. **Effects of water quenching on new reinforced concrete (normal and high strengths) columns under fire**

2. **Effects of fire and thereafter water quenching on old normal strength concrete (Pump House concrete of 40 years old)**

3. **Effects of spalling of various grades of concrete, with/without propylene fibres and wire mesh, under fire**

4. **Effects of passive protective coatings on enhancing fire resistance of concrete structures**

Pump House (modification for fire test)

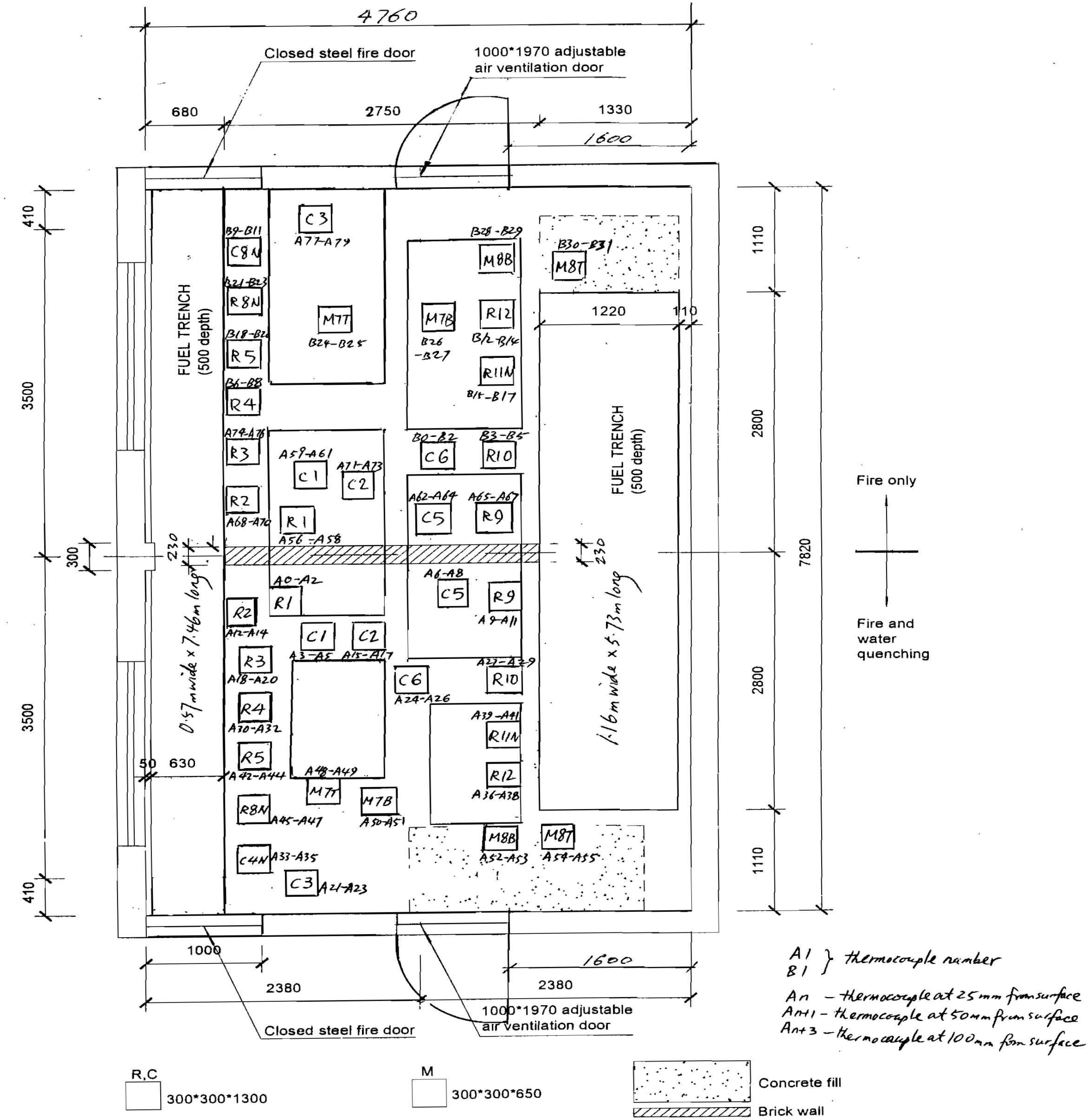
Test columns and Study Parameters

- **32 nos. of 300mm x300mm x 1300mm and 8 nos. of 300mm x 300mm x 650mm reinforced/concrete columns (pre-conditioned to fully moisture-saturated status) to be fire-tested.**
- **Old concrete of Pump House wall, roof slab, and columns to be fire-tested.**
- **Study Parameters**:

  Grades 40, 60, 90 concretes
  Grade 90 concretes with pp fibres, and/or with wire mesh, or with protective coating
  Old concrete (about 40 years old normal strength concrete)
  With and without water quenching

Table 1: Details of 26 Test Columns

Column Mark	Concrete Grade	Measured Cube Strength prior to Fire Test (MPa)	Remarks
R1, R1W	C40	73.3	-
R2, R2W	C60	88.3	-
R3, R3W	C90	107.3	-
R4, R4W	C90a	100.5	With 0.6 kg/m^3 PP fibre
R5, R5W	C90b	107.0	With 1.0 kg/m^3 PP fibre
R8N, R8NW	C60b	79.5	With 1.0 kg/m^3 PP fibre
R9, R9W	C90	107.3	20 mm magnesium silicate board coating attached on surface
R10, R10W	C90	107.3	20 mm vermiculate cement coating applied on surface
R11N, R11NW	C40	73.3	100 mm x 100 mm 2.0 mm dia. Galvanized mild steel mesh embedded in concrete cover
M7B, M7BW	C40A	73.2	-
M7T, M7TW	C40A	73.2	50 mm x 50 mm 2.6 mm dia. galvanized mild steel mesh embedded in concrete cover
M8B, M8BW	C40Ab	73.0	With 1.0 kg/m^3 PP fibre
M8T, M8TW	C40Ab	73.0	With 1.0 kg/m^3 PP fibre, and 50 mm x 50 mm 2.6 mm dia. galvanized mild steel mesh embedded in concrete cover



Note: Test columns shall be in a fully moisture-saturated condition under fire

Figure 6: Layout Plan of 40 Test Columns

Air temperatures near Central Brickwall at 1.6m above Floor

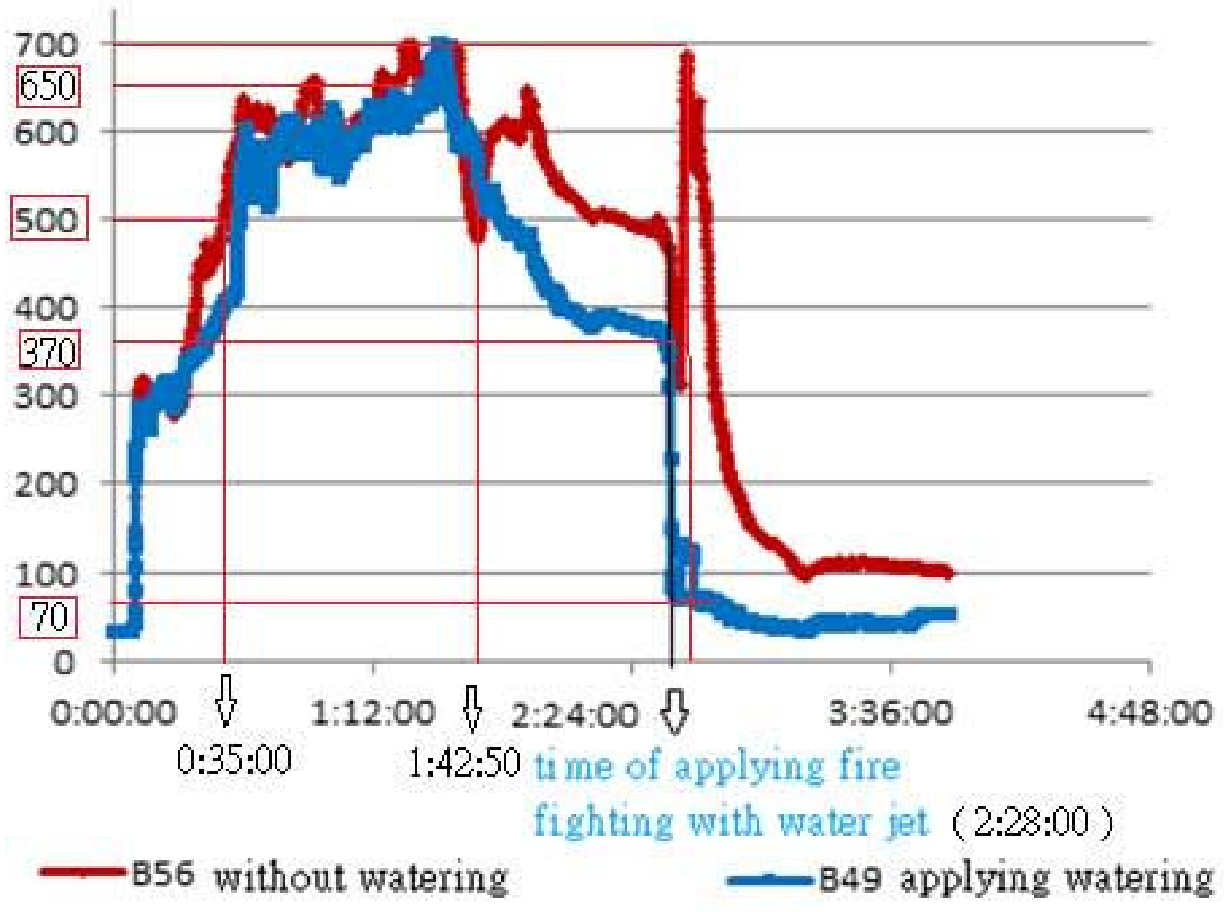


Figure 4: Air Temperatures of Fire Chamber Recorded by Thermocouples B56 and B49

Figure 5: Fire-Damaged Test Columns (with Water Quenching)

Figure 6: Fire-Damaged Test Columns (without Water Quenching)

Grading of Concrete Cover Spalling

- Class 1 was the spalling less than 40 mm deep (longitudinal rebars unexposed).
- Class 2 was the spalling less than 60 mm deep (longitudinal rebars exposed).
- Class 3 was spalling greater than 60 mm deep (spalling penetrated behind longitudinal rebars).

Table 3: Extent of Concrete Cover Spalling of Test Columns without Water Quenching

Column	Remarks	Spalled Area (% of total vertical surface areas of column)			
		Class 1	Class 2	Class 3	∑
R1	C40	13	3	5	21
R11N	C40 with 100 x 100 x 2 steel mesh	7	20	0	27
M7B	C40A	12	5	0	17
M7T	C40A with 50 x 50 x 2.6 steel mesh	5	0	0	5
M8B	C40Ab with 1.0 kg/m^3 pp fibre	0	0	0	0
M8T	C40Ab with 1.0 kg/m^3 pp fibre and 50 x 50 x 2.6 steel mesh	0	0	0	0
R2	C60	8	25	0	43
R8N	C60b with 1.0 kg/m^3 pp fibre	0	0	0	0
R3	C90	35	10	2	47
R9	C90 with 20mm Mg Si board coating	24	0	0	24
R10	C90 with vermiculate cement coating	0	0	0	0
R4	C90a with 0.6 kg/m^3 pp fibre	1	0	0	1
R5	C90b with 1.0 kg/m^3 pp fibre	0	0	0	0

Table 4: Extent of Concrete Cover Spalling of Test Columns with Water Quenching

Column	Remarks	Spalled Area (% of total vertical surface areas of column)			
		Class 1	Class 2	Class 3	∑
R1W	C40	2	29	5	36
R11NW	C40 with 100 x 100 x 2 steel mesh	2	22	11	35
M7BW	C40A	5	0	0	5
M7TW	C40A with 50 x 50 x 2.6 steel mesh	2	0	0	2
M8BW	C40Ab with 1.0 kg/m^3 pp fibre	0	0	0	0
M8TW	C40Ab with 1.0 kg/m^3 pp fibre and 50 x 50 x 2.6 steel mesh	0	0	0	0
R2W	C60	49	2	3	54
R8NW	C60b with 1.0 kg/m^3 pp fibre	1	1	1	3
R3W	C90	2	28	25	55
R9W	C90 with 20mm Mg Si board coating	2	11	15	28
R10W	C90 with vermiculate cement coating	1	5	0	6
R4W	C90a with 0.6 kg/m^3 pp fibre	2	0	0	2
R5W	C90b with 1.0 kg/m^3 pp fibre	0	0	0	0

Key Findings of Concrete Cover Spalling

- (i) Without adopting one of the above-mentioned EN methods to suppress concrete spalling under fire, the severity of concrete cover spalling increased with the cube strength of the test columns. Concrete spalling often occurred at the corners of the columns.

- (ii) Water quenching has an insignificant adverse effect on the concrete cover spalling. This might be due to the fact that most of the concrete cover spalling and macro-crack formation happened in the first hour of fire (air temperature less than 600C). Further sustained fire or increase in fire temperature would lead to more degradation or decomposition of the concrete matrix but the mechanism of concrete spalling basically ceased. As water quenching was applied to the columns at the fire time of 2 hours and 28 minutes, the temperature shock so induced might cause concrete further cracking or crack opening but not spalling.

Key Findings of Concrete Cover Spalling

- (iii) The provision of a mild steel mesh of 2.0 mm diameter at 100 mm x 100 mm centres embedded in the concrete cover could not effectively reduce concrete cover spalling to an acceptable level.
- (iii) The provision of a mild steel mesh of 2.6 mm diameter at 50 mm x 50 mm centres embedded in the concrete cover or 1.0 kg/m3 of propylene fibre in concrete could significantly suppress concrete cover spalling.
- (iv) The provision of an appropriate vermiculate cement coating of 20 mm thick applied on the surface of the concrete column (cube strength about 100MPa) could substantially reduce concrete cover spalling.
- (v) In this study, the selected 20 mm thick magnesium silicate board attached to the surface of the concrete column could not reduce concrete cover spalling to a satisfactory level. Its anti-spalling effect might be improved by a better quality of the board or/and installation method.

Buildings after Fire

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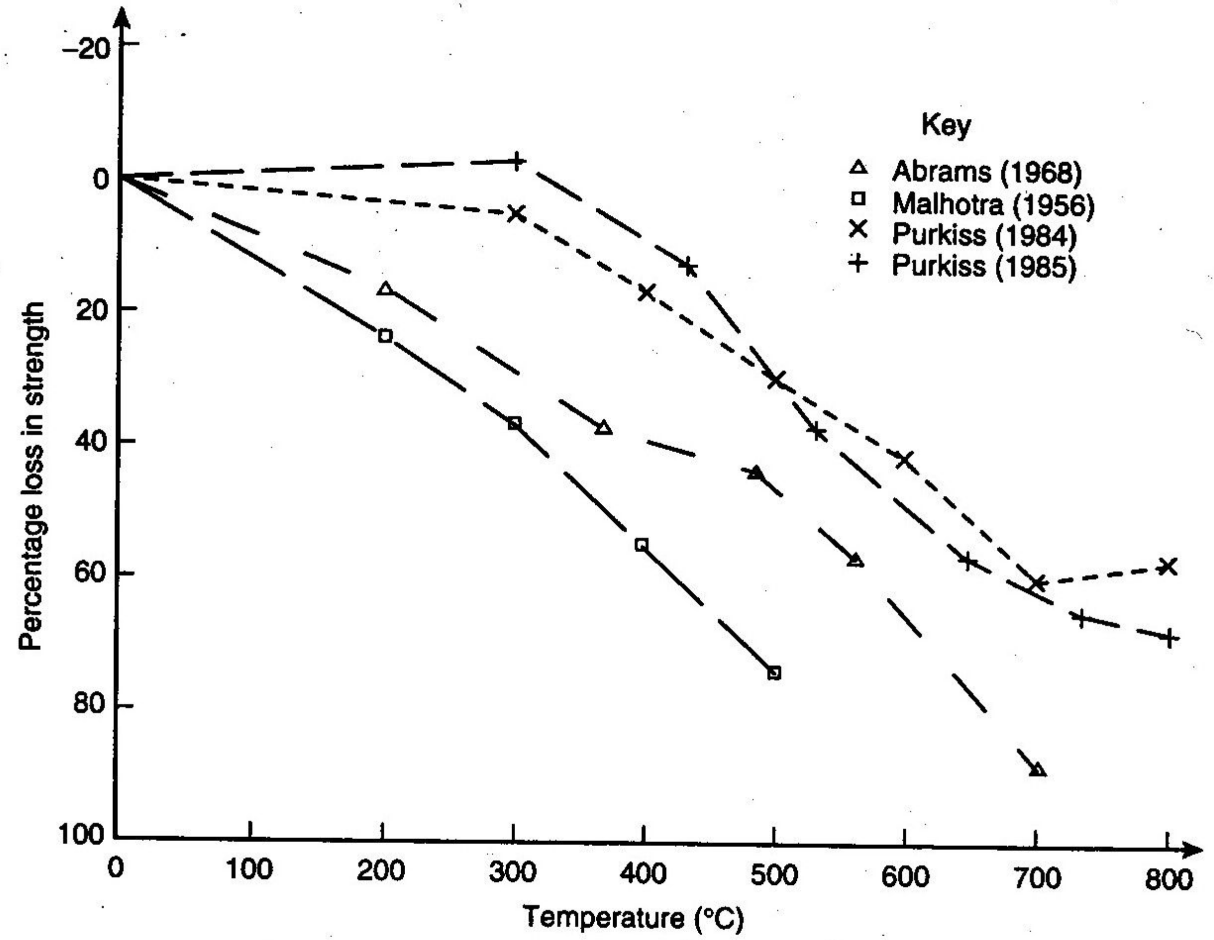
Residual Strengths of Concrete after exposed to Different Temperatures



Variation of residual strengths of concrete with temperature (Malhotra, 1956, by permission of the Building Research Establishment: Crown Copyright; Abrams, 1968, by permission; Purkiss, 1984, 1985).

Fire Safety Engineering, J.A. Purkiss

Residual Strengths of Concrete after exposed to Different Temperatures (PolyU Tests)

Concrete mix proportions

Mix	SF (%)	FA (%)	GGBS (%)	W/B	Batched quantities (Kg/m^3)					28 days comp. strength (MPa)
					Water	Cement	Fine agg.	Coarse agg.	SP*	
High strength concrete										
HS-CC**	-	-	-	0.30	150	500	758	927	0.5	85.9
HS-SF5	5	-	-	0.30	150	475	710	1066	0.6	96.5
HS-SF10	10	-	-	0.30	150	450	620	1151	0.8	108.3
HS-FA20	-	20	-	0.30	150	400	618	1147	0.8	82.7
HS-FA30	-	30	-	0.30	150	350	615	1143	0.7	80.2
HS-FA40	-	40	-	0.30	150	300	613	1139	0.7	76.7
HS-SF+FA	10	20	-	0.30	150	350	615	1142	0.8	105.3
HS-BS30	-	-	30	0.30	150	350	616	1145	0.7	83.9
HS-BS40	-	-	40	0.30	150	300	615	1142	0.7	80.9
NS-CC**	-	-	-	0.50	195	390	768	917	--	35.8
NS-FA30	-	30	-	0.50	195	273	626	1133	--	39.3
NS-FA40	-	40	-	0.50	195	234	625	1129	--	36.9
NS-BS30	-	-	30	0.50	195	273	626	1135	--	46.4
NS-BS40	-	-	40	0.50	195	234	625	1132	--	39.8

* Superplasticizer content in percent by weight of binder; ** Control concrete

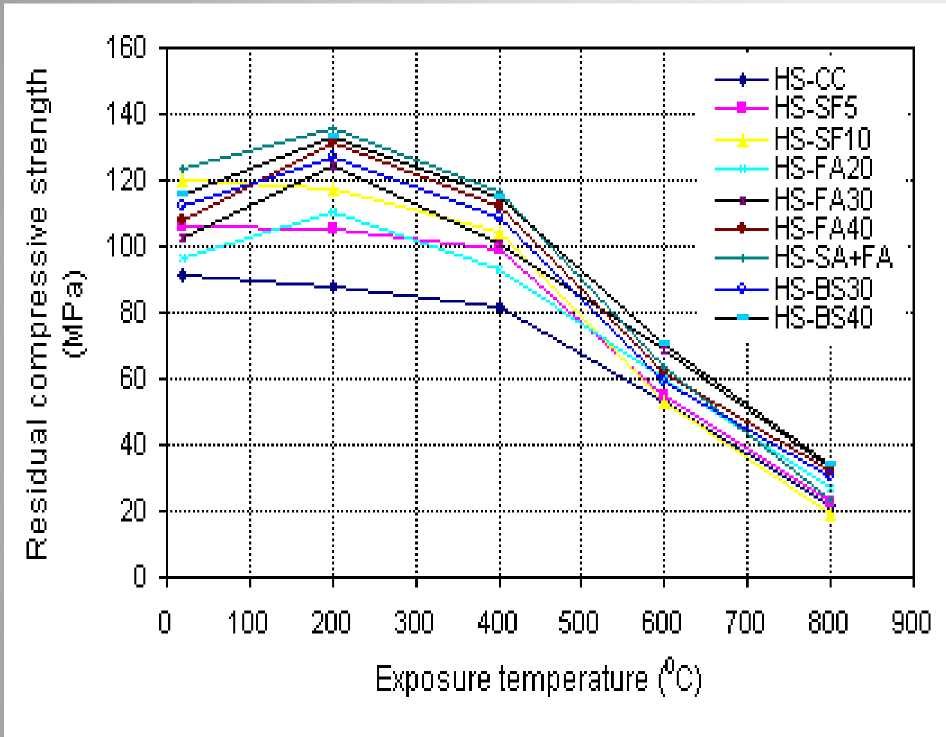
Curing and Heating Regimes, and testing (PolyU Tests)

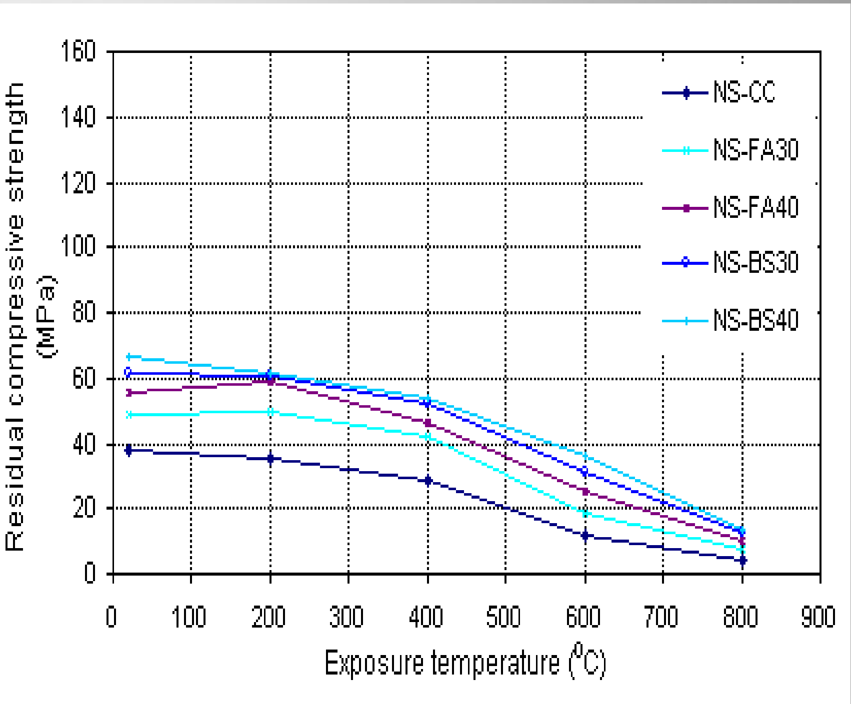
Curing of specimens: water cured for the first 28 days, and thereafter placed in a chamber of 20°C and **75% RH until date of heating (at 60 days)**

Heating rate: 2.5°C per minute

Testing: right after natural cooling

Residual Compressive Strength (PolyU Tests)

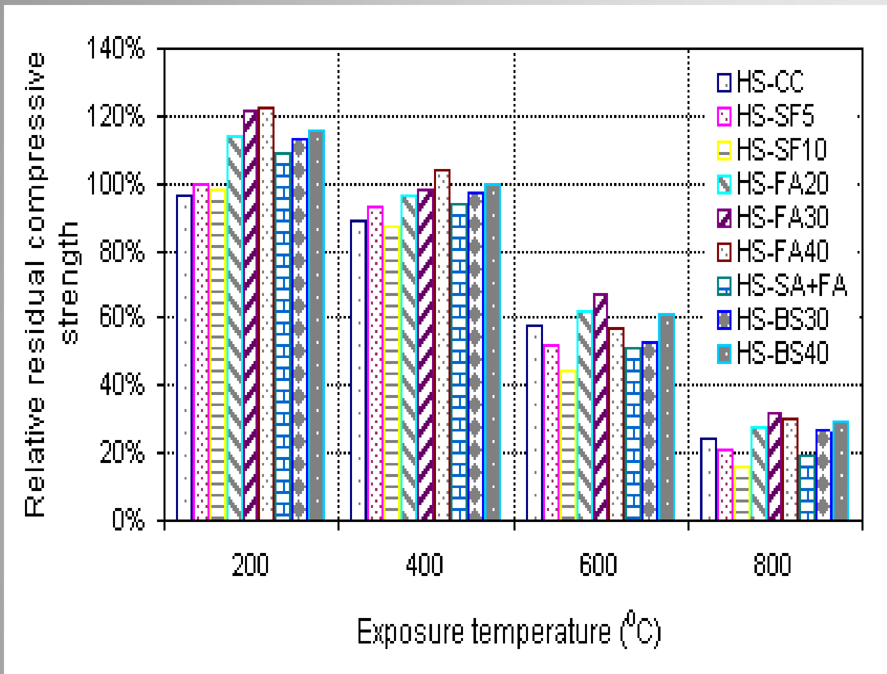




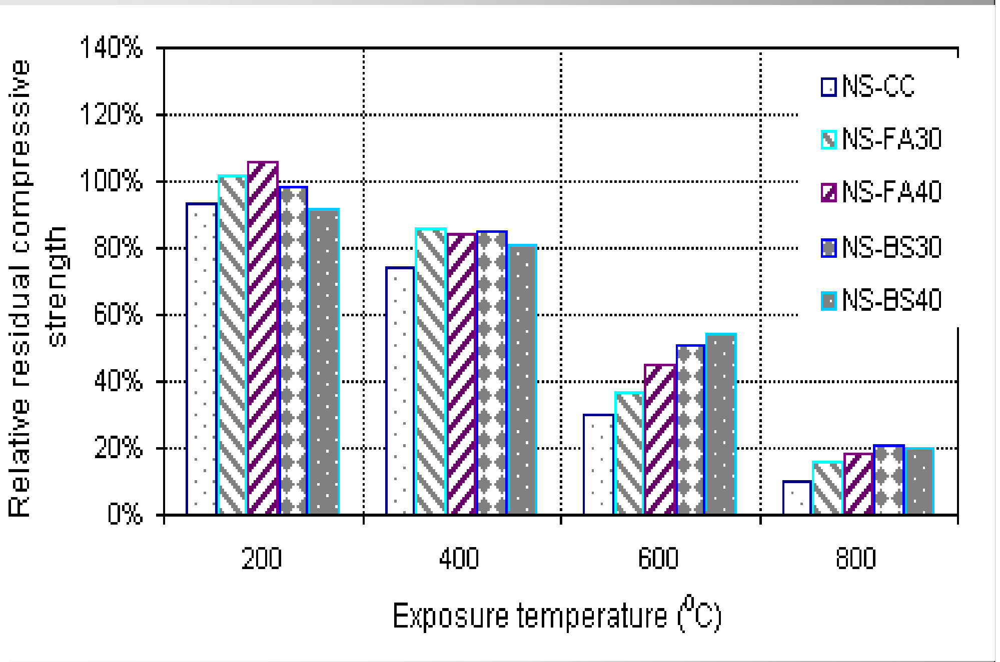
HSC

NSC

Relative Residual Compressive Strength (PolyU Tests)



HSC



NSC

Concrete with mineral admixtures (except CSF) has better performance.
Optimum replacement levels were 30% PFA for HSC,
and 40% GGBS for NSC.
Concretes preconditioned under 75%RH, Spalling was only observed in CSF concretes.

Rapid Chloride Penetration Test (ASTM C1202-97)

Chloride-ions penetrability based on charge passed (C)

Charge passed (coulombs)	Chloride-ions penetrability
> 4,000	high
2,000 – 4,000	moderate
1,000 – 2,000	low
100 – 1,000	very low
< 100	negligible

Impermeability Loss and Recovery (PolyU Tests)

Resistance against chloride-ions penetration before and after re-curing (for 600°C series)

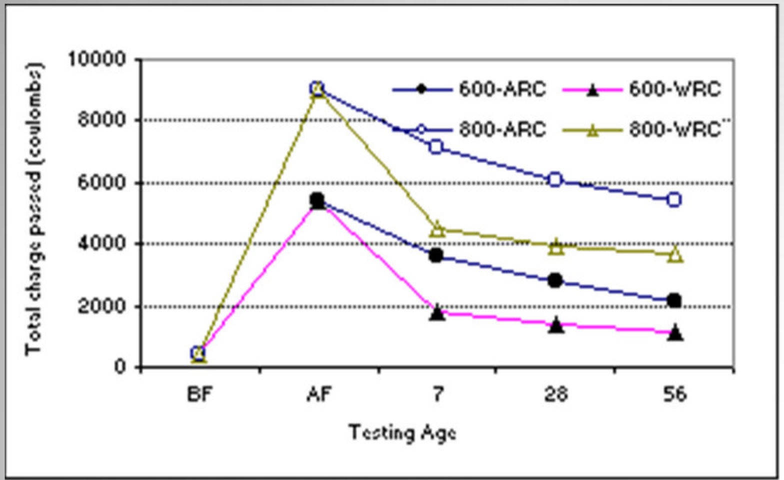
Mix	After cooling		Re-curing after 600°C					
			Air re-curing			Water re-curing		
	20°C	600°C	7 days	28 days	56 days	7 days	28 days	56 days
High strength concrete								
HS-CC	941	12534	8672	6847	6271	4934	4357	4864
HS-SF5	610	8619	6500	5309	4895	3662	3213	2914
HS-SF10	285	10080	7346	6337	5785	4318	4092	3729
HS-FA30	449	5373	3605	2798	2130	1833	1383	1171
HS-BS30	334	6772	4541	3774	3002	2725	2304	1831
Normal strength concrete								
NS-CC	2941	21792	14800	11530	10620	8647	7200	6441
NS-FA40	1454	8550	5605	3847	3533	2964	2591	2375
NS-BS40	1181	6306	5867	4620	4170	3313	3021	2792

Impermeability Loss and Recovery (PolyU Tests)

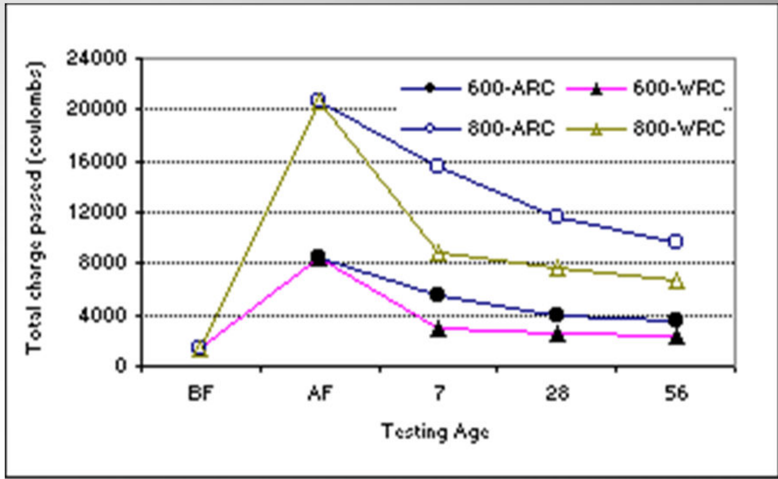
Resistance against chloride-ions penetration before and after re-curing (for 800°C series)

Mix	After cooling		Re-curing after 800°C					
			Air re-curing			Water re-curing		
	20°C	800°C	7 days	28 days	56 days	7 days	28 days	56 days
High strength concrete								
HS-CC	941	23396	18001	14992	14123	12737	10592	12082
HS-SF5	610	18390	13049	12195	11204	10189	9165	7818
HS-SF10	285	25170	17476	16439	15689	12770	11997	10965
HS-FA30	449	9049	7130	6104	5410	4480	3971	3656
HS-BS30	334	10673	7064	6673	6022	5446	4934	4266
Normal strength concrete								
NS-CC	2941	35724	29230	22742	20646	17604	15575	13943
NS-FA40	1454	20666	15525	11600	9640	8892	7596	6752
NS-BS40	1181	15368	13136	11060	10320	7701	7321	6476

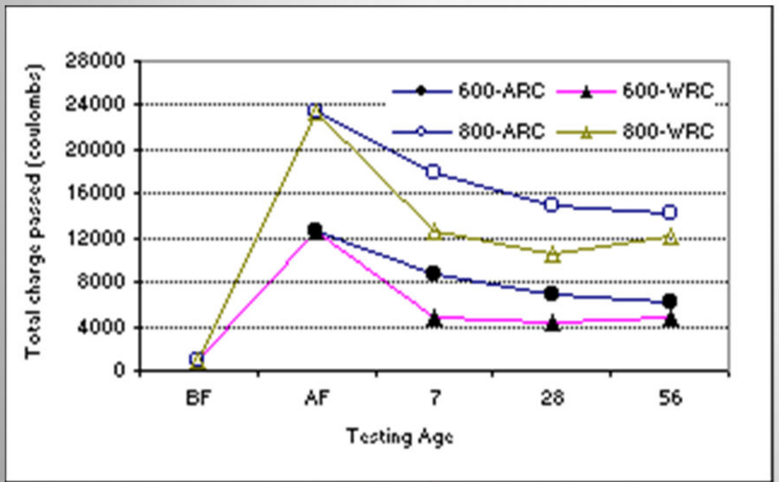
Impermeability Loss and Recovery (PolyU Tests)



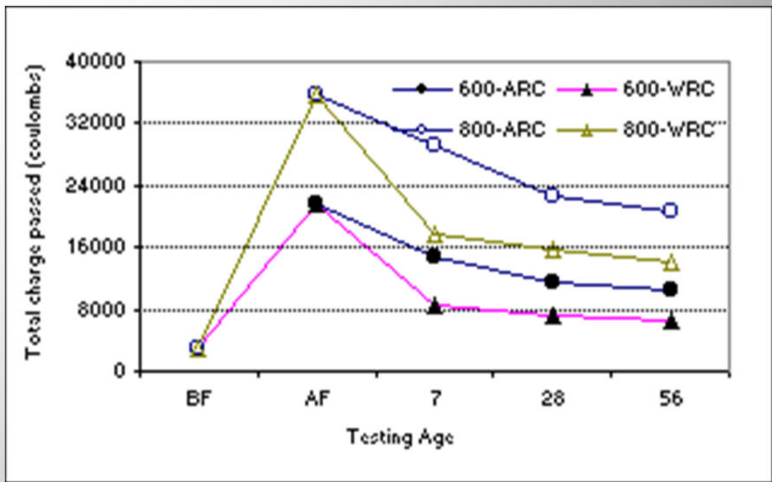
HS-FA30



NS-FA40

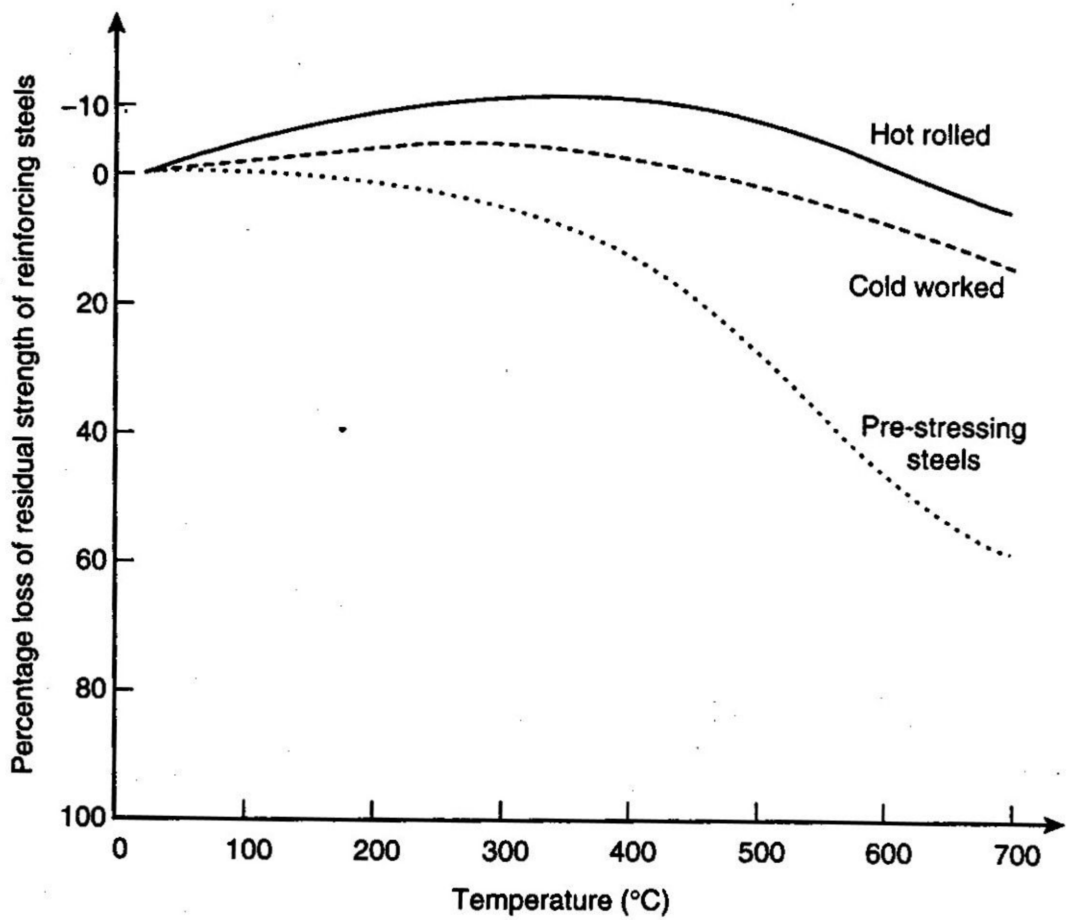


HS-CC



NS-CC

Residual Strengths of Rebars after Exposure to Different Temperatures



Variation of residual strengths of reinforcing and pre-stressing steels with temperature (Holmes *et al.*, 1982).

Fire Safety Engineering, J.A. Purkiss

Residual Strengths of Rebars after Exposure to Different Temperatures (PolyU Tests)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Sample No.	Sample Location (Area) !!	Strength (MPa)		E-Value (kN/mm^2)	Elongation (%)	Yield Strength Loss 1 – (3)/Ym #	Remarks
		Yield	Ultimate				
R16BT (T10)	B	284	459	212	-	40%	##
RS4T (T10)	B	276	451	190	-	42%	
RS1B (T12)	A	405	620	-	35	30%	###
RS4B (T12)	A	398	635	-	42	31%	

Notes:

\# Ym = 0.5(582+573) = 578MPa for T12 rebars
= 0.5(488+461) = 475MPa for T10 rebars
(3) = yield strength value shown in column 3 of this table

\## Un-burned rebars subjected to heating rate of 5^0C per minute up to 900^0C in PolyU Electric Furnace, stayed at 900^0C for one hour, and then air-cooled to ambient temperature.

\### Un-burned rebars subjected to heating rate of 5^0C per minute up to 800^0C in PolyU Electric Furnace, stayed at 800^0C for one hour, and then air-cooled to ambient temperature.

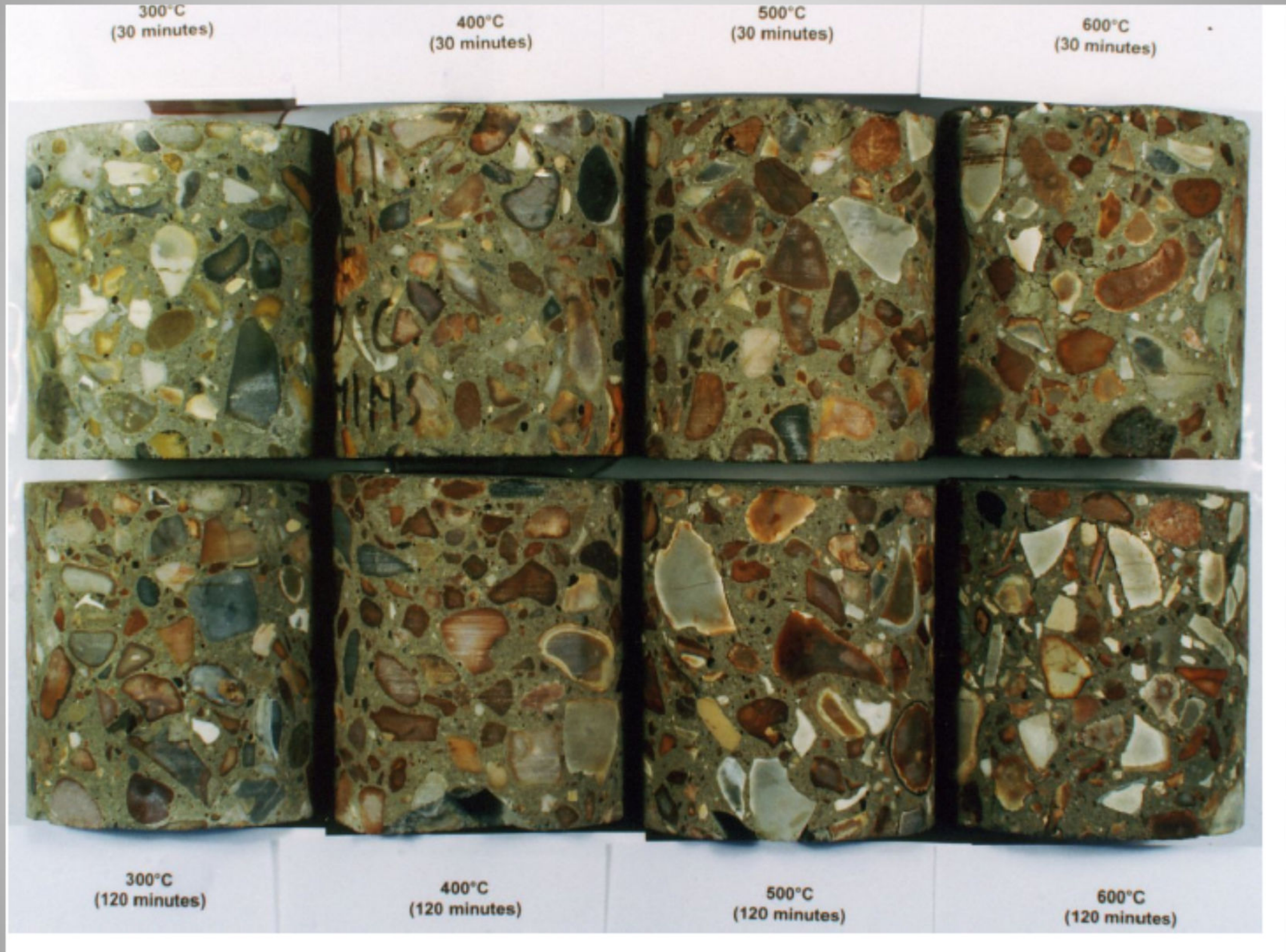
Concrete Colour Change After Fire

Colour changes in heated concrete

Concrete type	Colouration	Temperature (°C)	Condition
Siliceous	Normal	0–300	Normal strength
	Pink	300–600	Loss in strength
	Whitish-grey	600–900	Weak and friable
	Buff	above 900	Weak and friable
Limestone	Grey	0–200	Normal strength
	Light pink	200–400	Loss in strength
	Dull grey	400–600	Poor

Note: Not all siliceous or limestone aggregate concretes will show these changes, as they may be due to impurities in the sand as well as the aggregate. Absence of or a different colour change to those noted above should be treated with care.
Sources: Bessey (1956) Building Research Establishment: Crown Copyright and Ahmed, Al-Shaikh and Arafat (1992) by permission Thomas Telford Publications

Concrete Colour Change After Fire (Overseas Samples)



Appearance of flint aggregate concrete cores which have been heated for 1/2 hour (upper row) and 2 hours (lower row), at the temperature indicated

Concrete Colour Change After Fire (Hong Kong Samples)

Photo 7 Colour Change of CB5BI (cut from Beam Core CB5B in Area A)
(Layer 1: Concrete colour change zone, Layers 2 & 3: virgin concrete colour zone)

Table 2: Changes to concrete caused by heating
(compiled from Alarcon-Ruiz et al, 2005; Bessey, 1950; Larbi & Nijland, 2001; Lin, 1996 and Riley, 1991)

Heating temperature	**Changes caused by heating** (blue text denotes a feature that can be used to plot a thermal contour and red text denotes a significant strength threshold)
70-80°C	Dissociation of ettringite, $Ca_6Al_2(SO_4)_3(OH)_{12}.26H_2O$ causing its depletion in the cement matrix
105°C	Loss of physically bound water in aggregate and cement matrix causing an increase in the capillary porosity and sometimes microcracking of the cement paste which can be recognised by fluorescent microscopy
120-163°C	Decomposition of gypsum, $CaSO_4.2H_2O$ causing its depletion in the cement matrix
250-350°C	Pink-red discoloration of aggregate caused by oxidation of iron compounds commences at around 300°C. Explosive spalling likely at 300-400°C. May start to exhibit microcracking. Significant loss of compressive strength commences at 300°C
450-500°C	Dehydroxylation of portlandite, $Ca(OH)_2$ causing its depletion in the cement paste. Red discoloration of aggregate may deepen in color up to 600°C. Flint calcines at 250-450°C. Normally isotropic cement matrix exhibits patchy yellow-beige colour in cross-polarised light, often completely isotropic by 500°C
573°C	Transition of α-to β-quartz , accompanied by an instantaneous increase in volume of quartz of about 5% in a radial cracking pattern around the quartz grains in the aggregate
600-800°C	Decarbonation of carbonates; depending on the content of carbonates in the concrete, e.g. if the aggregate used is calcareous, this may cause a considerable contraction of the concrete due to release of carbon dioxide, CO_2; the volume contraction will cause severe microcracking of the cement paste. Concrete not structurally useful after heating in temperatures in excess of 500-600°C
800°C-1200°C	Complete disintegration of calcareous constituents of the aggregate and cement matrix due to both dissociation and extreme thermal stress, causing a whitish grey coloration of the concrete and severe microcracking. Limestone aggregate particles become white
1200°C	Concrete starts to melt
1300-1400°C	Concrete melted

Materials Testing

Concrete

- Compressive strength test of core samples – average strength of core length
- Schmidt hammer Tests – surface concrete
- Windsor probe and pull out test – surface concrete
- Thermoluminescence test of mortar – changes to silica to determine the maximum temperature, requires specialist equipment
- Petrographic analysis of thin slices from cores – density and type of cracking (Below 300^0C: aggregate-mortar matrix interface cracks, Above 500^0C: cracks in matrix)
- Permeability Tests (such as rapid chloride ion penetration test)

Compressive Strength and Charge Passed of Core Samples

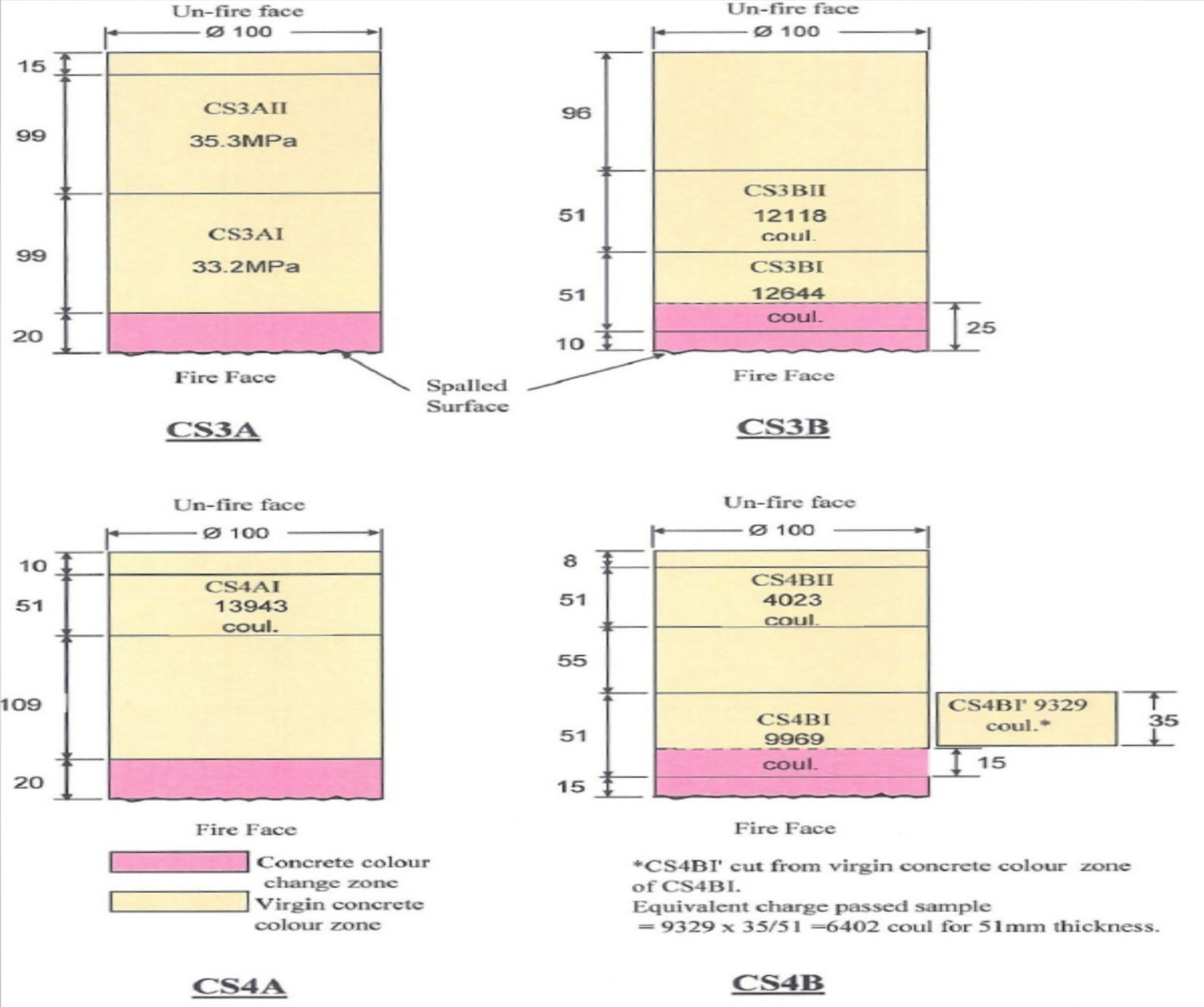


Figure 3 Concrete Colour Change Zone, Compressive Strength, and Charge Passed of Slab Concrete Cores (100 mm diameter) in Area A

End of Presentation

Thank you

Contacts:

Email: ylwong@chuhai.edu.hk

Mobile Phone: +852-64969170