Available online at www.sciencedirect.com
SCIENCE DIRECT®
Cement & Concrete Composites 27 (2005) 449–461
Cement & Concrete Composites
www.elsevier.com/locate/cemconcomp

A study on reinforcement corrosion and related properties of plain and blended cement concretes under different curing conditions

Erhan Güneyisi, Turan Özturan *, Mehmet Gesoğlu

Department of Civil Engineering, Boğaziçi University, 34342, Bebek, Istanbul, Turkey

Received 28 March 2003; accepted 28 May 2004

Abstract

This paper presents the results of an experimental investigation on the steel reinforcement corrosion, electrical resistivity, and compressive strength of concretes. Concretes having two different water–cement ratios (0.65 and 0.45) and two different cement contents (300 and 400 kg/m^3) were produced by using a plain and four different blended portland cements. Concrete specimens were subjected to three different curing procedures (uncontrolled, controlled, and wet curing). The effect of using plain or blended cements on the resistance of concrete against damage caused by corrosion of the embedded reinforcement has been investigated using an accelerated impressed voltage setup. The resistivity of the cover concrete has been measured non-destructively by placing electrodes on concrete surface. The compressive strength, electrical resistivity, and corrosion resistance of the concretes were determined at different ages up to 180 days. The results of the tests indicated that the wet curing was essential to achieve higher strength and durability characteristics for both plain and especially blended cement concretes. The concretes, which received inadequate (uncontrolled) curing, exhibited poor performance in terms of strength and corrosion resistance.

Keywords: Concrete; Corrosion resistance; Curing conditions; Electrical resistance; Plain and blended cements; Strength

1. Introduction

Performance of concrete is generally judged by strength and durability properties. Probably the most important durability issue with reinforced concrete is deterioration due to reinforcement corrosion [1,2]. A detailed description of the corrosion process can be found in the study of Rosenberg et al. [3]. In the alkaline cementitious environment, a stable oxide film is formed on the steel surface which protects the interior steel from corroding. However, corrosion starts due to the carbonation of concrete leading to a reduction in the alkalinity, or the presence of chloride ions causing pitting damage of the protective film on the steel bar. The corrosion product absorbs water and increases in volume. Once the expansion becomes excessive, concrete cracking will occur. Following the approach proposed by Tuutti [4], the corrosion process can be divided into two parts: an initiation (depassivation) stage and a propagation (corrosion) stage. During the initiation stage, corrosion agents such as chloride ions and carbon dioxide penetrate into the concrete cover, but their concentration around the steel reinforcement is not high enough to cause corrosion yet. The end of the initiation stage or the beginning of the propagation stage is the moment when corrosion starts at threshold concentration of aggressive species. Within the propagation stage, steel corrosion is accompanied by the growth of radial cracks from the steel bar, which will eventually lead to spalling of the concrete cover.

* Corresponding author. Tel.: +90 212 3596450; fax: +90 212 2872457.
E-mail address: ozturan@boun.edu.tr (T. Özturan).

0958-9465/$ - see front matter
doi:10.1016/j.cemconcomp.2004.05.006

Performance measurements of reinforced concrete related to the corrosion of embedded steel may be based on the type and condition of steel bar and thickness and quality of concrete cover, properties of cement paste, mortar, and concrete, and electrochemical conditions of the reinforcement in contact with the solution, paste, mortar, or concrete. Issues relating to reinforcement type have been extensively discussed in the literature [5–7], and the use of galvanized steel, stainless steel, or coated reinforcement in preventing chloride-induced steel corrosion in concrete have been investigated [8–10]. Cracking has been noted to increase the likelihood of damage to reinforced concrete [11–13]. Cover to reinforcement is considered of prime importance by some researchers while others believe that concrete type/quality is more critical than the cover thickness [14,15]. Further information on concrete cover, its modification, cracking, and delamination can also be found in the literature [16,17].

Blended (or pozzolanic) cements are being used worldwide to produce dense and impermeable concrete. They contain a blend of portland cement clinker and a variety of natural pozzolans and/or supplementary cementing materials such as blast furnace slag, fly ash, and silica fume. The use of these materials is also environmental friendly because it helps to reduce the CO_2 emission to the atmosphere [18]. The interest in the use of such mineral additives resulted in more detailed specifications in the United States and Europe for blended cements. The range of compositions as specified by the U.S. and European standards are summarized in Table 1. The beneficial effects of incorporating these materials in concrete are widely discussed in the literature [19–23]. Use of pozzolanic materials in concrete considerably reduces its permeability and rate of diffusion of moisture and aggressive species to the steel–concrete interface. Due to the increased density of concrete, resulting from the addition of pozzolanic materials, damage due to sulfate attack, alkali-aggregate reaction, and reinforcement corrosion is greatly reduced. The addition of a wide range of blending materials of differing chemical composition also introduces significant diversity into the cementing system. The wide variation in the performance of the blending materials may be attributed to the variation in their physical, chemical, and mineralogical composition resulting from the industrial processes related to their production and the properties of the raw materials used. Furthermore, the reported longer curing period required for blended cement concretes, as opposed to plain cement concrete, is still a question often debated among concrete technologists. Since pozzolanic reaction is highly dependent on good curing practice, there is often concern as to the effect of curing on the permeability of pozzolanic cement concrete. Many investigators [24–26] believe that a curing period of about 28 to 90 days is required for pozzolanic cement concrete specimens to attain properties superior than that of plain cement concrete. This is attributed to the pozzolanic reaction of these materials, which is often quite slow [24].

This paper is part of a large research project on evaluating the various durability aspects of plain and blended cement concretes. The objective of this study is to investigate the relative performance of a range of portland and blended cement concretes exposed to high chloride concentrations. The performance evaluation has been carried out in terms of corrosion of embedded reinforcement and related properties including compressive strength and electrical resistivity under three different curing conditions. Based on the test results, the effects of the type of cement, water–cement ratio (w/c), age, and curing conditions have been discussed.

Table 1
Blended cements according to American and European specifications

Specifications	Name	Portland cement content	Blended minerals
ASTM C595	Portland blast furnace slag	30–75%	Granulated blast furnace slag
	Slag-modified portland cement	>75%	Granulated blast furnace slag
	Portland pozzolan cement	60–85%	Pozzolan
	Pozzolan modified portland cement	>85%	Pozzolan
	Slag cement	<30%	Granulated blast furnace slag
EN 197	CEM I portland cement	95–100%	Minor addition constituents
	CEM II[a] portland composite cement	65–94%	Blast furnace slag, silica fume, pozzolans (natural or calcined), fly ash, burnt shale, limestone
	CEM III[b] blast furnace cement	5–64%	Blast furnace slag
	CEM IV[c] pozzolanic cement	45–89%	Silica fume, pozzolans, fly ash
	CEM V[d] composite cement	20–64%	Blast furnace slag, pozzolans, fly ash

[a] Includes subclassification depending on type of blended mineral.
[b] Includes subclassification depending on content of slag: 36–65%, 66–80%, 81–95%.
[c] Includes subclassification depending on content of pozzolans (silica fume + pozzolans + fly ash): 11–35%, 36–55%.
[d] Includes subclassification depending on content of blending minerals (blast furnace slag + pozzolans + fly ash): 36–60%, 62–80%.

2. Experimental program

2.1. Details of materials used

Five different cements from various sources, namely portland cement (CEM I), portland composite cements (CEM II/A-M & CEM II/B-M), composite cement (CEM V/A), and blast furnace slag cement (CEM III/A) were used. Physical properties and chemical analysis of these cements as obtained from the manufactures are given in Table 2. Details of the compositions of the cements are presented in Table 3. The coarse aggregate was a crushed limestone with a maximum particle size of 20 mm whereas the fine aggregate was a mix of natural sand and crushed limestone sand. Properties of the aggregates are presented in Table 4. A sulphonated naphthalene formaldehyde-based superplasticizer was used to get a workable fresh concrete. Specific gravity of the superplasticizer was 1.22.

2.2. Mixture details

Two series of concretes were produced in this study. The concretes in the first series had a water–cement ratio of 0.65 and a cement content of 300 kg/m^3. In the second series, the concretes were made with a water–cement ratio of 0.45 and a cement content of 400 kg/m^3. The first and second series concrete mixtures were denoted by (N) and (H), respectively. Five different cements were used in concretes of the both series. Concretes produced with CEM I, CEM II/A-M, CEM II/B-M, CEM V/A, and CEM III/A were designated as B1, B2, B3, B4, and B5, respectively. Grading of the aggregate mixture was kept constant for all concretes. All concrete mixtures were designed to provide a slump of 17 ± 2 cm for the ease of handling, placing, and consolidation. The superplasticizer was added at the time of mixing. Mix proportions for the concretes are given in Table 5.

2.3. Casting and curing of test specimens

All concretes were mixed as per ASTM C192 in a pan mixer by first mixing the dry ingredients for one minute, and then adding the water and mixing for an additional three minutes. Cubes having dimensions of $150 \times 150 \times 150$ mm and 100×200 mm cylinders were cast for the compressive strength and electrical resistivity tests, respectively. The reinforced concrete specimens for the accelerated corrosion tests were 100×200 mm concrete cylinders in which a 16 mm diameter steel bar was centrally embedded. The steel bar was embedded into the concrete cylinder such that its end was at least 30 mm from the bottom of the cylinder, and it was coated with epoxy at the exit from the concrete cylinder in order to eliminate crevice corrosion at these locations. Rebars were cleaned with a wire brush to remove the rust from

Table 2
Physical, chemical, and strength characteristics of cements

Cement type (Turkish & EN 197-1)	PÇ 42.5 & CEM I	PKÇ/A 42.5R & CEM II/A-M	PKÇ/B 42.5 & CEM II/B-M	KZÇ/A 42.5 & CEM V/A	CÇ 42.5 & CEM III/A
Experimental code	B1	B2	B3	B4	B5
SiO_2 (%)	20.64	18.38	28.34	25.63	28.81
Al_2O_3 (%)	5.06	5.05	7.33	5.06	7.2
Fe_2O_3 (%)	3.14	2.89	2.89	3.72	2.31
CaO (%)	63.98	61.78	52.55	48	49.94
MgO (%)	1.2	1.36	2.09	–	4.44
SO_3 (%)	2.38	2.34	2.88	2.3	2.41
Na_2O (%)	0.31	0.28	0.21	–	0.15
K_2O (%)	0.8	0.73	–	–	0.87
Cl^- (%)	0.035	0.036	–	0.01	0.027
Insoluble residue (%)	0.46	0.48	7.8	–	0.64
Loss of ignition (%)	1.72	6.44	1.16	–	2.4
Free lime (%)	1.41	1.44	0.35	–	0.83
Specific gravity	3.15	3.12	3.01	3.05	2.94
Vicat (hour:minute)					
Start	02:28	02:28	02:40	02:32	02:40
Stop	03:02	03:08	03:30	03:22	03:30
Le chatelier (mm)	2	2	1	1	1
Fineness (%)					
45 μm	11.7	18.1	–	–	1.3
90 μm	0.8	3	6.4	0.2	0.0
200 μm	0.0	0.4	0.7	–	–
Specific surface (m^2/kg)	336	334	406	430	464
f_c (at 2 days) (MPa)	27.5	23.7	23.1	20	13.3
f_c (at 7 days) (MPa)	41.3	39	35.9	31	24.6
f_c (at 28 days) (MPa)	51.4	46.2	51.2	45	–

Table 3
Composition of the cements (% by weight)

Composition	Component fraction in cement				
	Cement type & (Turkish EN 197-1)				
	PÇ 42.5 & CEM I	PKÇ/A 42.5R & CEM II/A-M	PKÇ/B 42.5 & CEM II/B-M	KZÇ/A 42.5 & CEM V/A	CÇ 42.5 & CEM III/A
	Experimental code				
	B1	B2	B3	B4	B5
Clinker, K	95.5	78.7	70.5	57.5	46.7
Blast furnace slag, S	0	2.0	13.0	21.8	48.3
Limestone, L	0	11.9	0	3.0	0
Natural pozzolans, P	0	3.2	13.0	12.6	0
Gypsum	4.5	4.2	3.5	5.1	5.0
Total (%)	100	100	100	100	100

Table 4
Sieve analysis and physical properties of aggregates

Sieve size (mm)	Fine aggregate		Coarse aggregate	
	Sand	Crushed sand	No. I	No. II
31.5	100	100	100	100
16.0	100	100	100	76.9
8.0	100	98.7	62.6	1.6
4.0	98.2	89.8	22.8	0.9
2.0	94.8	53.6	3.5	0.7
1.0	91.2	34.6	2.3	0.6
0.50	82.3	22.3	1.8	0.2
0.25	14.3	9.5	1.4	0.2
Fineness Modulus	1.19	2.92	5.06	6.19
Specific Gravity	2.60	2.69	2.70	2.70
Absorption, %	0.50	1.00	0.50	0.40

surface just before casting the reinforced concrete specimens. Twenty-seven cubes, nine cylinders, and twelve reinforced concrete specimens were cast from each concrete mixture. The specimens were cast in three layers and compacted using a vibrating table. After casting, the moulded specimens were covered with a plastic sheet and left in the casting room for 24 h. They were then demoulded and divided into three equal groups and cured under the following conditions:

Uncontrolled curing (UC): specimens were air cured at uncontrolled temperature and relative humidity until the test age.

Controlled curing (CC): specimens were immersed in 20 ± 2 °C water for 7 days and then air cured in a room at 20 ± 1 °C and 50 ± 5% relative humidity until the test age.

Wet curing (WC): specimens were immersed in 20 ± 2 °C water until the test age.

2.4. Test details

Compressive strength: The concrete cubes (150 × 150 × 150 mm) were used for the compressive strength tests at 28, 90, and 180 days, where three specimens from each mixture were tested at each testing age. The test was performed on a 2000 kN capacity compression testing machine, and the test procedure followed during the test was in conformity with BS 1881:Part116: 1983 [27].

Table 5
Mix proportioning of concrete in kg/m^3

Concrete series	Code	Mix proportioning (kg/m^3)							
		w/c	Cement	Water	Coarse aggregate		Fine aggregate		SP*
					No. I	No. II	Sand	Crushed sand	
1	N-B1	0.65	308.1	200.3	558.3	616.0	537.6	191.8	0.77
	N-B2	0.65	302.8	196.8	547.8	604.4	527.5	188.2	0.76
	N-B3	0.65	304.7	198.1	548.6	605.4	528.3	188.5	0.76
	N-B4	0.65	306.5	199.2	552.9	610.1	532.4	189.9	0.77
	N-B5	0.65	306.4	199.2	549.7	606.5	529.3	188.8	0.77
2	H-B1	0.45	405.4	182.4	536.2	591.6	516.3	184.2	3.04
	H-B2	0.45	399.6	179.8	527.5	582.0	507.9	181.2	3.00
	H-B3	0.45	399.5	179.8	523.7	577.9	504.3	179.9	3.00
	H-B4	0.45	400.7	180.3	526.0	580.4	506.5	180.7	4.01
	H-B5	0.45	399.8	179.9	521.0	574.9	501.7	179.0	4.00

* SP = Superplasticizer.

Electrical resistivity measurement: The electrical resistivity of three 100 × 200 mm concrete cylinders from each mixture was determined at the ages of 28, 90, and 180 days. For this purpose, an electrical resistivity meter was used, which produced frequency-independent resistivity measurement [28,29]. Two measurements were taken on the side face of each cylinder, and the average of the six readings on three specimens was reported for each concrete mixture. No measurements were made on the top and bottom faces of the cylinders to avoid variations induced by bleeding and repetitive vibration, respectively.

Accelerated corrosion test (ACT): A rapid corrosion testing technique was used to compare the corrosion performance of plain and blended cement concretes. Similar cells with some differences were reported by other researchers [30–34]. In this study, the reinforced concrete specimens were immersed in a 4% sodium chloride solution leveling the top of the concrete cylinder and the steel bar (working electrode) was connected to the positive terminal of a DC power source while the negative terminal was connected to stainless steel plates (counter electrode) placed near the specimen in the solution. In this circuit, the steel bar was the anode, the steel plates were the cathode, and the sodium chloride solution was the electrolyte. The corrosion process was initiated by impressing an anodic potential of 30 V. A high impressed voltage was used to accelerate the corrosion process and to shorten the test period to fit practical laboratory testing conditions. Fig. 1 is a schematic representation of the experimental setup for the accelerated corrosion test. The specimens were monitored periodically to see how long it takes for corrosion cracks to appear on the surface. A data logger was used for recording the current variation with time. The current increased abruptly when the specimen cracked, indicating the occurrence of cracking. The variation of current with time and time to failure of reinforced concrete specimens were determined for all concrete types. Two specimens from each concrete mixture were tested at the ages of 28 and 180 days.

3. Experimental results and discussion

3.1. Compressive strength

The 28, 90, and 180 day compressive strengths for the plain and blended cement concretes subjected to different curing procedures are shown in Fig. 2. Strength values ranged from 32.5 to 67.1 MPa and from 23.3 to 69.7 MPa for the plain and blended cement concrete specimens, respectively, depending upon w/c ratio, curing condition, and age at testing. Variation in compressive strength with concrete type, curing condition, and age at testing is shown in Fig. 2. It was observed that improving the quality of curing was more effective on the later age compressive strength development of the plain cement concretes with larger w/c ratio than the concretes with lower w/c ratio. Test results also indicated that the increase in compressive strength of concretes at later ages made with blended cements (B2–B5) was higher than that of concretes with portland cement (B1), especially under controlled and wet curing procedures. This is most probably due to the secondary pozzolanic reactions taking place in the blended cement

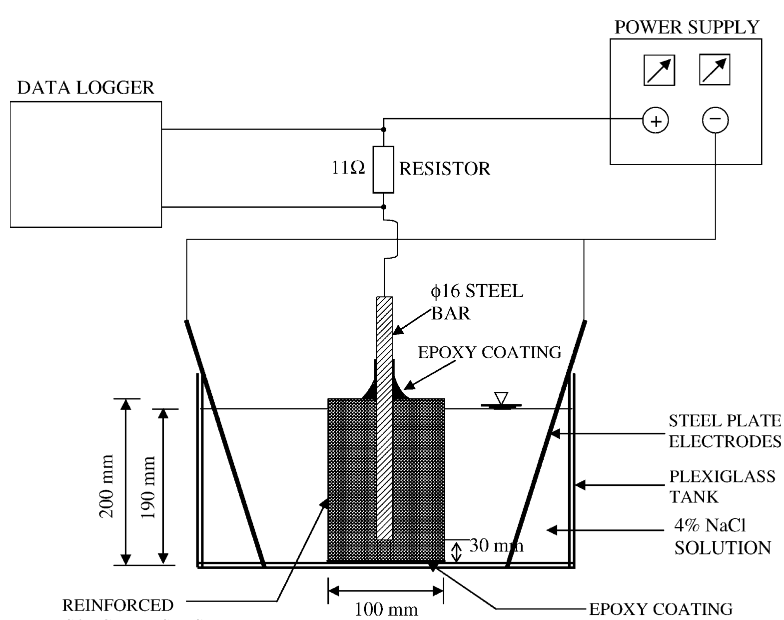

Fig. 1. Schematic representation of the setup for the accelerated corrosion test.

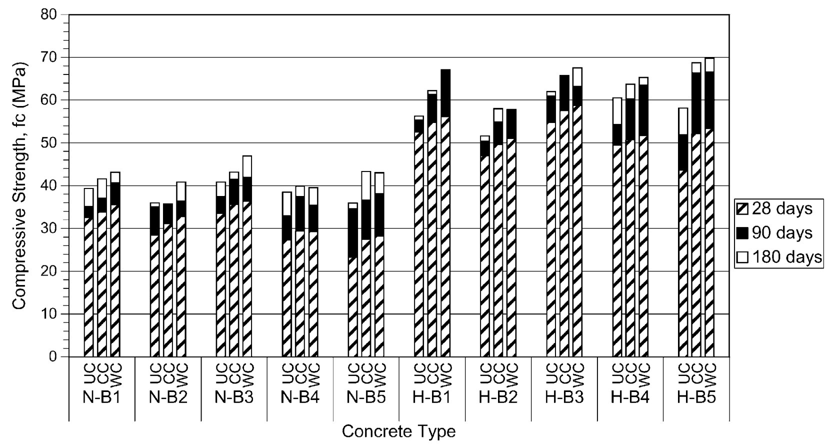

Fig. 2. Variation of compressive strength of plain and blended cement concretes subjected to uncontrolled, controlled, and wet curing conditions.

concretes. For example, the 180 day compressive strength of the concrete with blast furnace slag cement (B5) subjected to wet curing condition showed a 52.5% increase with respect to 28 day strength whereas this increment was only 21.4% for the plain cement concrete (B1). It is seen from Fig. 2 that the compressive strength of the concretes produced with blended cements were mostly lower than those of concretes with portland cement at 28 days. However, later (90 and 180 day) strengths of these concretes mostly exceeded the strengths of the plain portland cement concretes. When the blended cement concretes were compared with each other, for uncontrolled curing condition, concrete made with portland composite cement (B3) had greater strength than the others at all w/c ratios and ages. For controlled and wet curing procedures, portland composite cement (B3) had greater strengths at 28 days; whereas, concrete with blast furnace slag cement (B5) had higher strengths in comparison to the others at later ages, especially at low w/c ratio.

The effect of curing condition on the 28- and 180-day compressive strength of the plain and blended cement concretes is illustrated in Fig. 3. For both concrete types, uncontrolled curing resulted in lower 28- and 180-day strengths as compared to the other curing conditions. It was observed that at 28 days, the ratios of the compressive strength of the specimens cured under controlled curing to those cured under wet curing were generally within 5% of equality for both plain and blended cement concretes. However, the strength of the plain and blended cement concrete specimens cured under uncontrolled curing deviated up to −10% and −20% from those cured under wet curing, respectively, and the ratios always lied below 1.00. For uncontrolled curing condition, the absence of moist curing resulted in lower strengths where the blended cement concrete suffered more at both testing ages (28 and 180 days). This can be attributed to the lack of development of hydration and pozzolanic reactions to produce a dense microstructure as well as the extensive shrinkage cracking, which may have developed due to continuous air curing which has also been indicated by other researchers [35,36].

3.2. Electrical resistivity

The data concerning the variation of electrical resistivity with w/c ratio and concrete age for the plain and blended cement concretes subjected to different curing conditions are plotted in Fig. 4. The electrical resistivity values for the plain and blended cement concretes ranged from 5.2 to 17.8 kΩcm and from 3.6 to 29.6 kΩcm, respectively. The results indicated that cement type, w/c ratio, concrete age, and curing condition had significant effects on the electrical resistivity of concrete. As expected, for a given curing condition, decreasing the w/c ratio of the concrete increased the electrical resistivity, and for a given w/c ratio, improving the curing condition yielded higher electrical resistivity for all concretes. The increase in the electrical resistivity with strength of concrete was mainly due to the denser microstructure of concrete of higher strength resulting from lower w/c ratio. Cao and Sirivivatnon also reported the effect of w/c ratio on concrete resistivity [37]. They concluded that the resistivity of concretes produced with normal portland cement with and without silica fume increased with the increase in the design strength of concrete ranging from 20 to 50 MPa. From Fig. 4, it was noted that the blended cement concrete specimens had greater electrical resistivity than the plain cement concrete specimens for all w/c ratios and testing ages. This was more pronounced for the specimens subjected to controlled and wet curing procedures. Wee et al. [38] explained the lower chloride permeability of concretes containing mineral additives in terms of the lower ionic conductivity (OH^- ions) of the pore fluid and the denser microstructure of the cement paste which may also explain the higher electrical resistivity of these concretes.

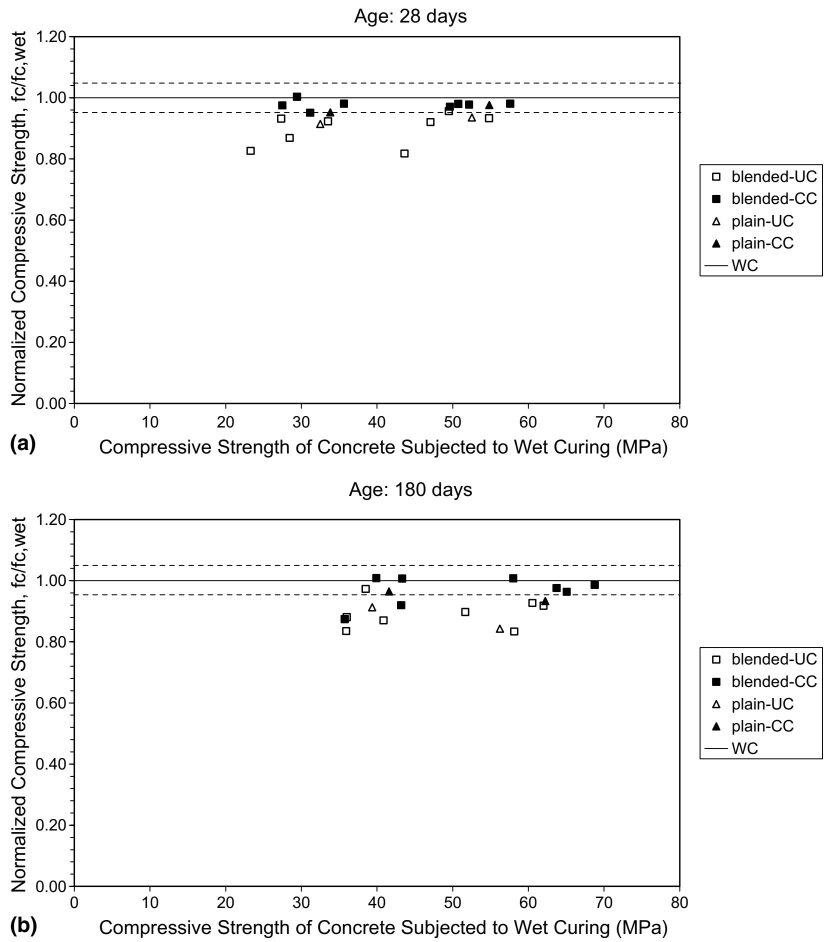

Fig. 3. Effect of curing condition on compressive strength of concretes: (a) at 28 days and (b) at 180 days.

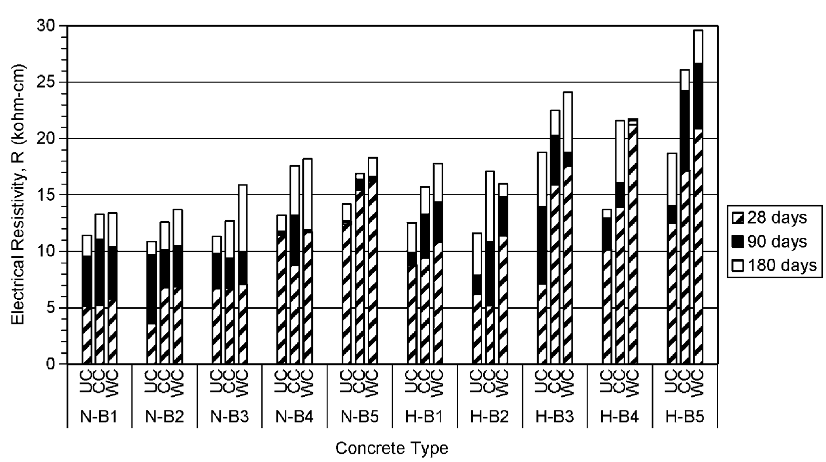

Fig. 4. Variation of electrical resistivity of plain and blended cement concretes subjected to uncontrolled, controlled, and wet curing conditions.

Among the blended cement concretes, blast furnace slag cement concretes (B5) exhibited the highest electrical resistivities whereas the lowest values were obtained from the portland composite cement concretes (B2).

Fig. 5 shows the influence of curing condition on the electrical resistivity of plain and blended cement concretes. From this figure, it was seen that, at 28 days, the ratios for some of the plain and blended cement concrete specimens subjected to controlled curing condition did not fall within 5% of equality and deviated up to −12% and −20% from those cured under wet curing procedure, respectively, as opposed to the compressive

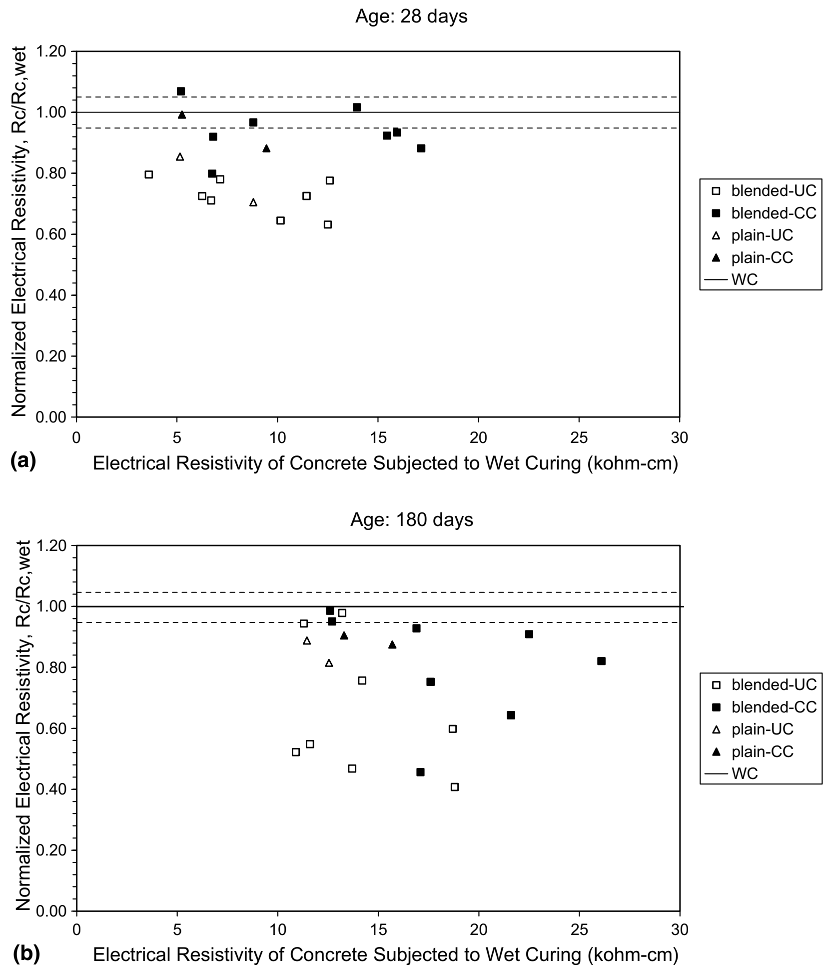

Fig. 5. Effect of curing condition on electrical resistivity of concretes: (a) at 28 days and (b) at 180 days.

strength test results obtained from the similar condition. On the other hand, uncontrolled curing condition caused reductions in the electrical resistivity of both plain and blended cement concretes as compared to controlled and wet curing procedures. Results showed that these ratios for the plain and blended cement concrete specimens under uncontrolled curing condition lied within a range of −15% to −29% and −20% to −37%, respectively. Fig. 5 also showed that the difference between the electrical resistivity of the concretes cured in wet and the other curing procedures, however, was higher at later ages (180 days). This was more pronounced for the concretes subjected to uncontrolled curing procedure.

3.3. Corrosion resistance

The accelerated corrosion behavior of steel bars embedded in plain and blended cement concrete specimens subjected to three different curing conditions were studied by impressing a constant anodic potential. The current required to maintain the fixed potential was plotted against time and the typical curves of corrosion current versus time for the concrete specimens made with portland cement (B1) and portland composite cement (B3) are illustrated in Figs. 6 and 7, respectively. Typical corrosion specimens after the termination of the test are shown in Fig. 8. As seen from Figs. 6 and 7, current–time curve initially descended till a time value after which a steady low rate of increase in current was observed, and after a specific time value a rapid increase in current was detected until failure. Almost a similar variation of the corrosion current with time has also been observed by other researchers [30–34]. The sudden rise of the current intensity coincided with the cracking of the specimen. Thus, this curve was utilized to determine the corrosion time of the specimen when the specimen cracked due to corrosion and the current started to increase sharply. The first visual evidence of corrosion was the appearance of brown stains on the surface of the specimens. Cracking was observed shortly thereafter and it was associated with a sudden rise in the current. Figs. 9–11 present the average corrosion times required to crack the specimens made with plain and blended

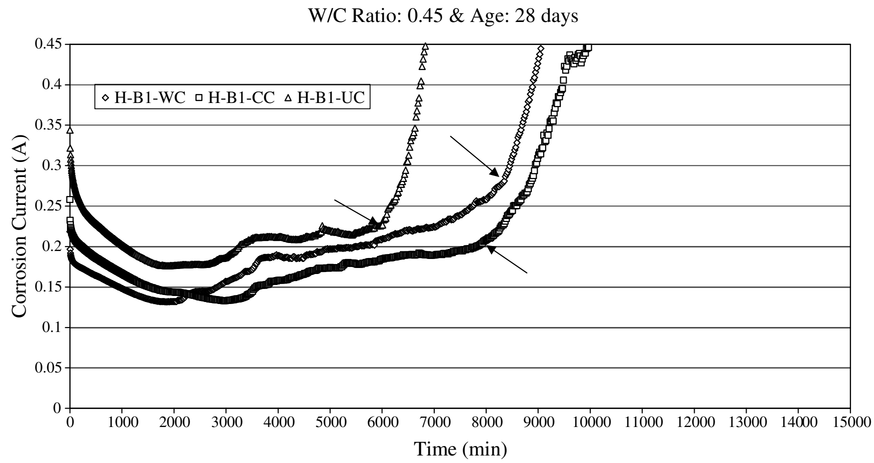

Fig. 6. Typical curve of corrosion current versus time at the test age of 28 days for the concretes made with portland cement (B1) under different curing conditions.

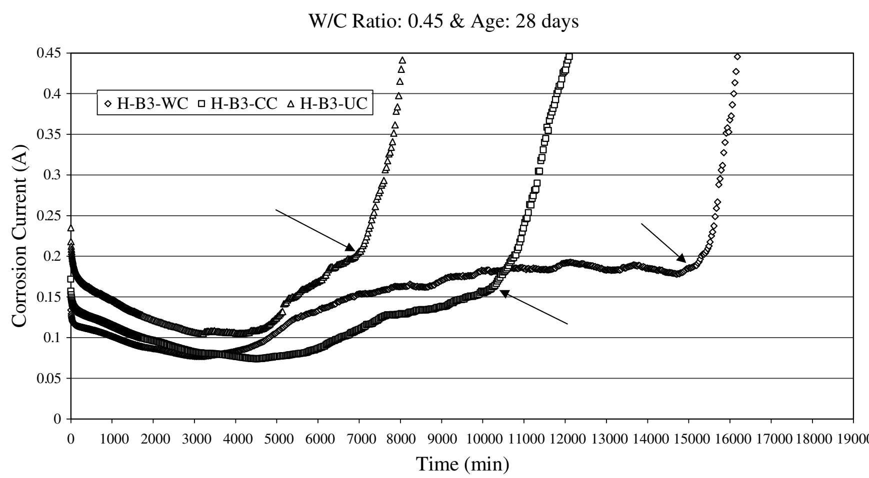

Fig. 7. Typical curve of corrosion current versus time at the test age of 28 days for the concretes made with portland composite cement (B3) under different curing conditions.

Fig. 8. Typical corrosion specimens after the accelerated corrosion test.

cements and subjected to uncontrolled, controlled, and wet curing regimes, respectively. Time to cracking in plain portland cement concrete specimens was in the range of 67–170 h (3–7 days) whereas that in blended cement concrete specimens was in the range of 50–440 h (2–18 days), depending on the cement type, w/c ratio, curing condition, and age at testing. At similar curing condition and testing age, the times of corrosion cracking for the blended cement concrete specimens were longer than the plain cement concrete specimens, which indicated that the former provided better protection to steel reinforcement against corrosion. Results also demonstrated that the increase in time to cracking with test age (from 28 to 180 days) for the plain portland cement concrete specimens were greater for the higher w/c ratio concretes for all curing conditions than mixes with a w/c ratio of 0.45. Moreover, this increment from 28 to 180 days was greater in almost all of the cases for the blended cement concrete specimens in comparison to the plain cement concrete specimens, especially when the specimens were subjected to controlled and wet curing conditions. The effect of using blended cements was more noticeable for the concrete specimens subjected to controlled and wet curing and tested at 180 days. Longer in moist curing for the low w/c ratio blended cement concretes allows the hydration and the pozzolanic reactions to fully develop to form a very dense microstructure with great reduction in porosity, resulting in less corrosion susceptibility of rebars and leading to a significant improvement in the resistance of concrete against cracking due to reinforcement corrosion.

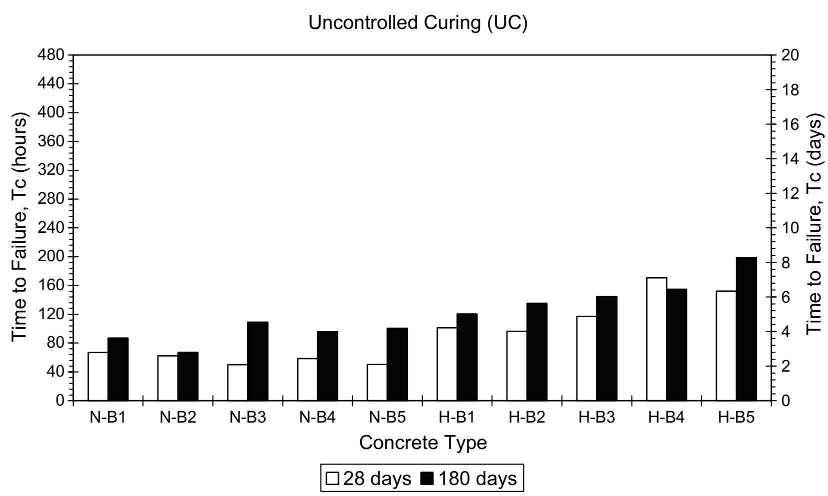

Fig. 9. Comparison of the average corrosion time required to crack the specimens made with plain and blended cements and subjected to uncontrolled curing regime (UC).

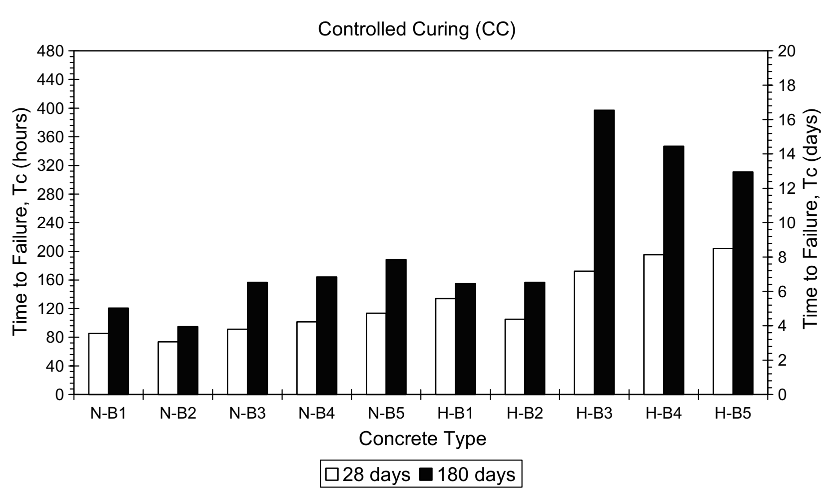

Fig. 10. Comparison of the average corrosion time required to crack the specimens made with plain and blended cements and subjected to controlled curing regime (CC).

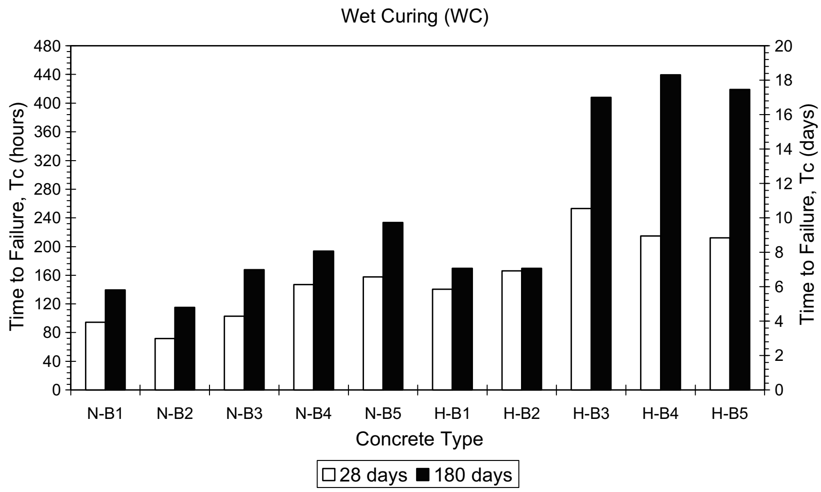

Fig. 11. Comparison of the average corrosion time required to crack the specimens made with plain and blended cements and subjected to wet curing regime (WC).

The effect of curing conditions on the corrosion resistance of plain and blended cement concrete specimens is well observed in Fig. 12. Lack of proper curing produced a similar effect on the corrosion time to that on the electrical resistivity of concrete. Uncontrolled curing lead to the shortest corrosion cracking time in all concretes. However, blended cement concrete specimens were affected more negatively by the lack of proper curing than the plain cement concrete specimens due to the change in hydration kinetics of cements and the

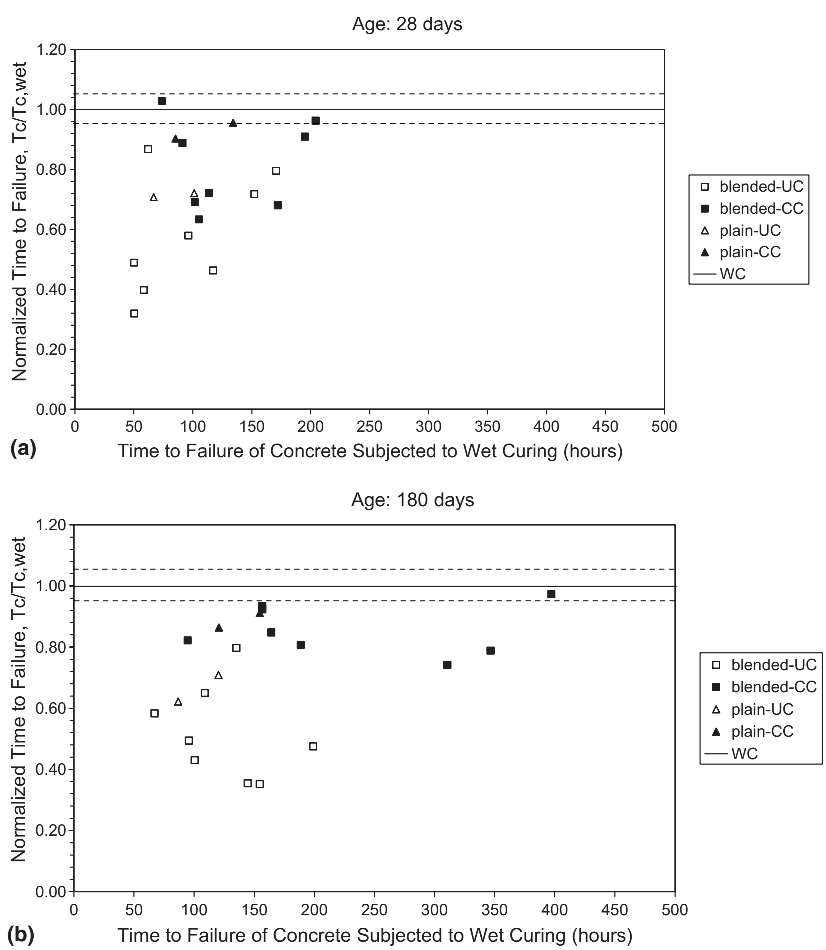

Fig. 12. Effect of curing condition on steel reinforcement corrosion: (a) at 28 days and (b) at 180 days.

hydration development at the curing cessation time [39]. It was also noted from Fig. 12 that, with increasing the testing age from 28 to 180 days, the ratios of time to cracking for the specimens subjected to both uncontrolled and controlled curing, always being below 1.00, did not fall within 5% of equality. Time to cracking of the reinforced concrete specimens cured under controlled and uncontrolled curing procedures deviated up to −25% and −65% from those cured under wet curing procedure, respectively.

4. Conclusions

Based on the results obtained from this study, the following conclusions may be drawn:

1. Cement type, w/c ratio, age, and curing procedure had significant effect on both strength and durability characteristics of concretes. Both plain and blended portland cement concretes subjected to uncontrolled curing in air had lower performance in terms of strength and corrosion resistance compared to the controlled and wet curing procedures.
2. The application of controlled curing gave average compressive strengths within 5% of those obtained at wet curing procedure for both concrete types. However, the strength of the plain and blended cement concrete specimens under uncontrolled curing condition deviated within a range of −10% and −20% from those cured under wet curing, respectively. Both uncontrolled and controlled curing procedures resulted in great differences with respect to wet curing in terms of electrical resistivity and corrosion time of the concretes made with plain and blended cements.
3. The results generally indicated that the strength gain in blended cement concretes was higher than that in plain portland cement concretes, especially under controlled and wet curing conditions. The concretes made with blended cements had mostly lower 28-day compressive strength as compared to the plain portland cement concretes. However, with increasing age, this trend was reversed.
4. For a given curing condition, lowering w/c ratio of the mixes increased the concrete resistivity, and for a given w/c ratio, better curing procedure yielded higher electrical resistivity for all concretes. The blended cement concretes had greater electrical resistivity than the plain portland cement concretes for all w/c ratios and ages.
5. The accelerated corrosion setup used under the present study has been found to be an efficient and simple tool to evaluate the durability performance of concretes, especially in terms of resistance of concrete against reinforcement corrosion.
6. The accelerated corrosion test results indicated that the specimens with blended cements had superior performance and mostly yielded longer time to corrosion cracking at similar curing condition and testing age compared to those with plain portland cements. The corrosion resistance of the blended cement concretes increased significantly with age while that of the conventional concrete had a marginal increase.

Acknowledgments

The author would like to thank to Turkish Cement Manufacturers' Association (TCMA) for providing financial assistance to carry out the present study. Thanks are also expressed to Mr. İlyas Gültekin, Mr. Hasan Şenel, and Mr. Hamdi Ayar for their help during the laboratory phase of the study.

References

[1] Watson R. Spaghetti junction liffs off £400M repairs. New Civil Eng 1990.
[2] Wallbank EJ. The performance of concrete in bridges: a survey of 200 highway bridges. London: HMSO; 1989.
[3] Rosenberg A, Hanson CM, Andrade C. Mechanisms of corrosion of steel in concrete. In: Skalny J, editor. Material Science of Concrete. Westerville, Ohio: American Ceramic Society; 1989. p. 285–313.
[4] Tuutti K. Corrosion of Steel in Concrete. Stochholm: Swedish Cement and Concrete Research Institute, 1982.
[5] Haynes G. Review of laboratory corrosion tests and standards. In: Baobian R, Dean SW, editors. Corrosion Testing and Evaluation: Silver Anniversary Volume, ASTM STP 1000. Philadelphia: American Society for Testing and Materials; 1992. p. 281–8.
[6] McCuen RH, Albrecht P, Cheng J. New approach to power-model regression of corrosion penetration data. In: Chaker V, editor. Corrosion forms and control for infrastructure. ASTM STP 1137. Philadelphia: American Society for Testing and Materials; 1992. p. 46–76.
[7] Fasullo EJ. Infrastructure: the battlefield of corrosion. In: Chaker V, editor. Corrosion forms and control for infrastructure. ASTM STP 1137. Philadelphia: American Society for Testing and Materials; 1992. p. 1–16.
[8] Rasheeduzzafar, Bader AM, Khan MM. Performance of corrosion resisting steel in chloride-bearing concrete. ACI Mater J 1992;89(5):439–48.
[9] Berke NS, Shen DF, Sundberg KM. Comparison of the polarization resistance technique to the macrocell corrosion technique. In: Berke NS, Chaker V, Whiting D, editors. Corrosion rates of steel in concrete. ASTM STP 1065. Philadelphia: American Society for Testing and Materials; 1990. p. 38–51.
[10] Rober H. Reinforcement for concrete structures subject to fatigue. In: Proceedings of Symposium on Fatigue of Steel and Concrete Structures, LABSE, AIPC, IVBH, Lausanne, Switzerland, 1982. p. 239–45.
[11] Dagher HJ, Kulendran S. Finite element modeling of corrosion damage in concrete structures. ACI Struct J 1992;89(6):699–708.

[12] Suzuki K, Ohno Y, Praparntanatorn S, Tamura H. Mechanism of steel corrosion in cracked concrete. In: Proceedings of Third International Symposium on Corrosion of Reinforcement in Concrete Construction, The Chemical Industry Society, UK, 1990. p. 19–28.
[13] Beeby AW. Cracking, cover and corrosion of reinforcement. In: ACI Spring Convention, American Concrete Institute, Dallas, Texas, 1981.
[14] Suzuki K, Ohno Y, Praparntanatorn S, Tamura H. Some phenomena of macrocell corrosion. In: Proceedings of Third International Symposium on Corrosion of Reinforcement in Concrete Construction: Society of the Chemical Industry, UK, 1990. p. 87–95.
[15] Gu P, Beaudoin JJ, Zhang MH, Malhotra VM. Performance of reinforcing steel in concrete containing silica fume and blast-furnace slag ponded with sodium chloride solution. ACI Mater J 2000;97(3):254–62.
[16] Babaei K, Hawkins NM. Performance of rehabilitated/protected concrete bridge decks. In: Chaker V, editor. Corrosion Forms and Concrete for Infrastructure. ASTM STM 1137. 154: American Society for Testing and Materials; 1992. p. 140–54.
[17] Rasheeduzzafar, Al-Saadoun SS, Al-Gahtani AS. Corrosion cracking in relation to bar diameter, cover, and concrete quality. ASCE J Mater Civil Eng 1992;4(4):327–42.
[18] Malhotra VM. Role of Supplementary Cementing Materials in Reducing Greenhouse Gas Emissions. CANMET Report MTL 98-03, OP & J, 1998.
[19] Berke NS. Resistance of microsilica concrete to steel corrosion, erosion, and chemical attack. In: Malhotra VM, editor. Proceedings of the International Conference on Fly Ash, Silica Fume, Slag, and Natural Pozzolans in Concrete, SP-114, American Concrete Institute, Farmington Hills, Mich., 1989. p. 861–86.
[20] Swamy RN. Blended cements in construction. London: Elsevier Applied Science; 1991.
[21] Hussain SE, Rasheeduzzafar. Corrosion resistance of fly ash blended cement concrete. ACI Mater J 1994;91(3):264–72.
[22] Özyıldırım C, Halstead WJ. Improved concrete quality with combinations of fly ash and silica fume. ACI Mater J 1994;91(6):587–94.
[23] Güneyisi E, Özturan T, Gesoğlu M. Laboratory investigation of chloride permeability for high performance concrete containing fly ash and silica fume. In: Dhir RK, Hewlett PC, Csetenyi LJ, editors. Proceedings of the International Conference, Challenges of Concrete Construction, Dundee, Scotland, 2002. p. 295–305.
[24] Manmohan D, Mehta PK. Influence of pozzolanic slag and chemical admixtures on pore-size distribution and permeability of hardened cement pastes. Cement, Concrete Aggr 1981;3(1):63–7.
[25] Nagataki S, Ujike II. Air permeability of concretes mixed with fly ash and condensed silica fume. In: Proceedings 2nd International Conference on the Use of Fly Ash, Blast Furnace Slag, and Silica Fume in Concrete, Madrid, 1986. p. 1049–68.
[26] Marsh BK, Day RL, Bonner DG. Pore structure characteristics affecting the permeability of cement paste containing fly ash. Cement Concrete Res 1985;15:1027–38.
[27] British Standard Institution, Methods for Determination of Compressive Strength of Concrete Cubes. BS 1881: Part 116, London, 1983.
[28] Wenner F. Bull Bureau of Standards 1915;12:469–78.
[29] Gowers KR, Millard SG. Measurement of concrete resistivity for assessment of corrosion severity of steel using Wenner technique. ACI Mater J 1999;96(5):536–41.
[30] Al-Tayyib AJ, Al-Zahrani MM. Corrosion of steel reinforcement in polypropylene fiber reinforced concrete structure. ACI Mater J 1990;87(2):108–13.
[31] Detwiler RJ, Kjellsen KO, Gjorv OE. Resistance to chloride intrusion of concrete cured at different temperature. ACI Mater J 1991;88(1):19–24.
[32] Shaker FA, El-Dieb AS, Reda MM. Durability of styrene butadiene latex modified concrete. Cement Concrete Res 1997;27(5):711–20.
[33] Okba SH, El-Dieb AS, Reda MM. Evaluation of the corrosion resistance of latex modified concrete (LMC). Cement Concrete Res 1997;27(6):861–8.
[34] Al-Zahani MM, Al-Dulaijan SU, Ibrahim M, Saricimen H, Sharif FM. Effect of waterproofing coating on steel reinforcement corrosion and physical properties of concrete. Cement Concrete Compos 2002;24:127–37.
[35] Ramezanianpour AA, Malhotra VM. Effect of curing on the compressive strength, resistance to chloride ion penetration and porosity of concretes incorporating slag, fly ash, or silica fume. Cement Concrete Compos 1995;17:125–33.
[36] Toutanji HA, Bayasi Z. Effect of curing procedures on properties of silica fume concrete. Cement Concrete Res 1999;29:497–501.
[37] Cao HT, Sirivivatnanon V. Corrosion of steel in concrete with and without silica fume. Cement Concrete Res 1991;21(2/3):316–24.
[38] Wee TH, Suryavanshi AK, Tin SS. Evaluation of rapid chloride permeability test (RCPT) results for concrete containing mineral admixtures. ACI Mater J 2000;97(2):221–32.
[39] Bonavetti V, Donza H, Rahhal V, Irassar E. Influence of initial curing on the properties of concrete containing limestone blended cement. Cement Concrete Res 2000;30:703–8.