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Effectiveness and structural implications of electrochemical chloride extraction from reinforced concrete beams ☆

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Abstract

The overall aim of this paper is to examine the effectiveness and structural implications of electrochemical chloride extraction when applied to reinforced concrete beams containing chlorides as well as chlorides and reactive aggregates. Twelve beams, 100 × 165 × 2300 mm, were altogether cast and tested. The main variables in the beams were water/cement ratio, chloride content and reactive aggregates. Four of the beams contained a reactive aggregate in the concrete mix. All the beams were kept dry, after initial curing, to see if the electrochemical process triggered alkali silica reactivity. The chloride extraction process used an anode system of oxide coated platinised titanium mesh and saturated calcium hydroxide solution as electrolyte. The applied cathodic current density was about 1 A/m^2. Chloride analysis of the concrete in the beams was carried out before, during and after the electrochemical treatment, and the pH of the electrolyte and any alkali silica reactivity were regularly monitored. The beams were subsequently tested to failure at an age of about 400 days. It is shown that electrochemical treatment is an effective and efficient process to remove chlorides from the cover concrete and from the zone containing steel reinforcement without any adverse effect on structural strength, bond or shear. There was clear evidence that unlike cast-in chlorides, the chloride extraction process will be far more efficient in real structures where the majority of chlorides that would have penetrated into the concrete will be from the outside aggressive environment and which will remain mostly in the concrete cover zone. However, further research is needed to establish the long term effects of alkali silica reactivity initiated by the CE process, and the structural implications of carrying out CE in the field if the dead and live loads carried by the beams are not relieved.

Keywords: Chloride extraction; Reinforced concrete beams; Chloride analysis; pH; Alkali silica reactivity; Ultimate strength

1. Introduction

Corrosion of steel in concrete is now universally recognized as a major factor leading to the deterioration of reinforced concrete (RC) structures [1]. Corrosion proceeds through two important processes in a RC structure, both arising from the permeability of concrete – carbonation of the cover concrete and chloride ingress. Of the two, chloride pitting corrosion of steel has been found to be the most widespread form of steel corrosion affecting the service life of many concrete structures, particularly since it can proceed rapidly even in the absence of carbonation. Although corrosion of steel may not, per se, immediately affect the integrity and the ultimate load capacity of a RC structural member, it is the most complex, insidious and destructive form of damage. Once it starts, it is almost impossible to stop the process until eventually the safety, stability and design service life are all drastically reduced with time [2,3].

Because of the nature and role of concrete in the creation, rehabilitation and regeneration of the infrastructure system of any country, RC plays a very important part in a nation's economic development. Lack of durability of RC structures has thus not only massive economic implications to a nation's well-being, but it is also one of the greatest threats to sustainable growth of the concrete and construction industries [4,5]. As a result, a number of

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corrective and preventive strategies, which have the potential to extend the durable service life of RC structures, have been developed and applied extensively over the past two decades. These strategies which confer longer service life in aggressive exposure conditions include the use of pozzolanic cement replacement materials, protective systems for concrete and steel and the adoption of new holistic design approaches [6–14]. New RC structures incorporating one or more of these new strategies are likely to have much longer durable service life than those constructed before.

The fact remains, however, that many RC structures built in the latter half of the last century still remain contaminated with chlorides, and continue to deteriorate at a rapid rate due to steel corrosion. Cathodic protection (CP) and electrochemical removal of chlorides offer two possible methods of rehabilitation of such structures. Cathodic protection systems have been extensively applied, but their main disadvantage is the need for regular maintenance, and the system has to remain on the structure for the duration of its service life. Chloride extraction (CE) is a newer technique, and although similar to CP, it is economically more attractive as it does not need ongoing, long term maintenance. Electrochemical CE has been successfully applied in the field, and tests have demonstrated that the residual chlorides can be brought down to levels that will ensure the cessation of reinforcement corrosion [15–17]. However, much of the reported laboratory research on CE has been carried out on small scale test specimens and not on RC beams [18–23], and there are still concerns on the structural implications of CE, particularly on the initiation and possible enhancement of alkali–silica reactivity (ASR) in structural members.

The overall aim of this paper is to provide answers to some of these concerns, and examine the effectiveness and efficiency of electrochemical chloride extraction from chloride contaminated reinforced concrete beams, and the structural implications of CE, particularly on the ultimate strength and design safety of the beams. The RC beams used in this study were normal beams designed to fail in flexure without distress in shear or bond. Weaknesses in shear and/or bond have other structural implications, and are beyond the scope of this paper. The CE was also carried out on the beams in an uncracked state without external loads. In practice this will correspond to beams when the cracks have been sealed, and the beams are relieved of their dead and live load stresses during the CE process. The paper also reports on the vulnerability of RC beams containing reactive aggregates to ASR due to the CE process. To achieve these aims, 12 RC beams were altogether cast and tested. The main variables in the beams were water/cement ratio and chloride content. Four of the beams contained a well-tested reactive aggregate to simulate ASR in the laboratory. The beams were kept dry prior to CE to see if the electrochemical process triggered ASR. The beams were subsequently tested to failure to examine the effects of CE on ultimate strength and the associated structural implications arising from the electrochemical process. The anode system used for the CE process consisted of oxide coated platinised titanium mesh and saturated calcium hydroxide solution. The applied cathodic current density was about 1 A/m^2.

2. Experimental programme

2.1. Details of test beams

Twelve RC beams, as shown in Fig. 1, were altogether tested, the main variables in the beams being water/cement (w/c) ratio, chloride content and the amount of reactive aggregates. Six of the beams had a w/c ratio of 0.60, and six 0.75. Two of the beams contained no chlorides, whilst two contained 2.4% chloride and two 6%, all by mass of cement. The sodium chloride was added during mixing of the concrete. Four of the beams with 6% chlorides also contained fused silica as a reactive aggregate to create ASR – the fused silica was used to replace the fine aggregate by an amount equal to 5% and 15% by mass of the total aggregate. The details of the beams with their chloride and fused silica contents are shown in Table 1.

All the 12 beams were identical in size, 100 × 165 × 2300 mm, and reinforcement details as shown in Fig. 1. The tension reinforcement and the stirrup hanging bars in the compression zone in the beams were high yield ribbed steel bars with a tensile strength of 630 MPa, and yield stress of 510 MPa. The stirrups were of 6 mm square section with a yield stress of 350 MPa; they were obtained by cutting from a piece of square hollow steel section. The concrete in the beams consisted of Type 1 portland cement with a specific surface of 345 m^2/kg and a total equivalent alkali content of 0.82%. The coarse aggregate consisted of a mixture of washed rounded and crushed gravel with 10 mm maximum particle size. The fine aggregate was washed natural sand. A chemical analysis of both the sand and gravel showed that the aggregates contained very little chlorides, less than 0.01%, and the aggregates can thus be considered to be innocuous so far as alkali aggregate reactivity was concerned. The reactive aggregate used to initiate ASR was a synthetic aggregate – an amorphous fused silica (FS) with over 99.5% silica content and no alkali. Extensive tests on the use of fused silica as a realistic reactive aggregate to simulate ASR in concrete have been reported [24–26].

Various field studies [1,5,9] have shown that corrosion of steel arises from two basic weaknesses – high permeability concrete and low concrete cover due to poor quality construction. So in the tests reported here, two w/c ratios were used, namely 0.60 and 0.75, with a low cover of 20 mm. The higher w/c ratio of 0.75 was deliberately included to examine the structural and ASR implications arising from the more effective current flow and the resulting chloride ion movement (as indeed shown later) that will occur in the beams with high chloride contents.

2.2. Testing for chloride extraction

All the beams were cast in steel moulds, and cured in the moulds under wet hessian for two days. The beams were

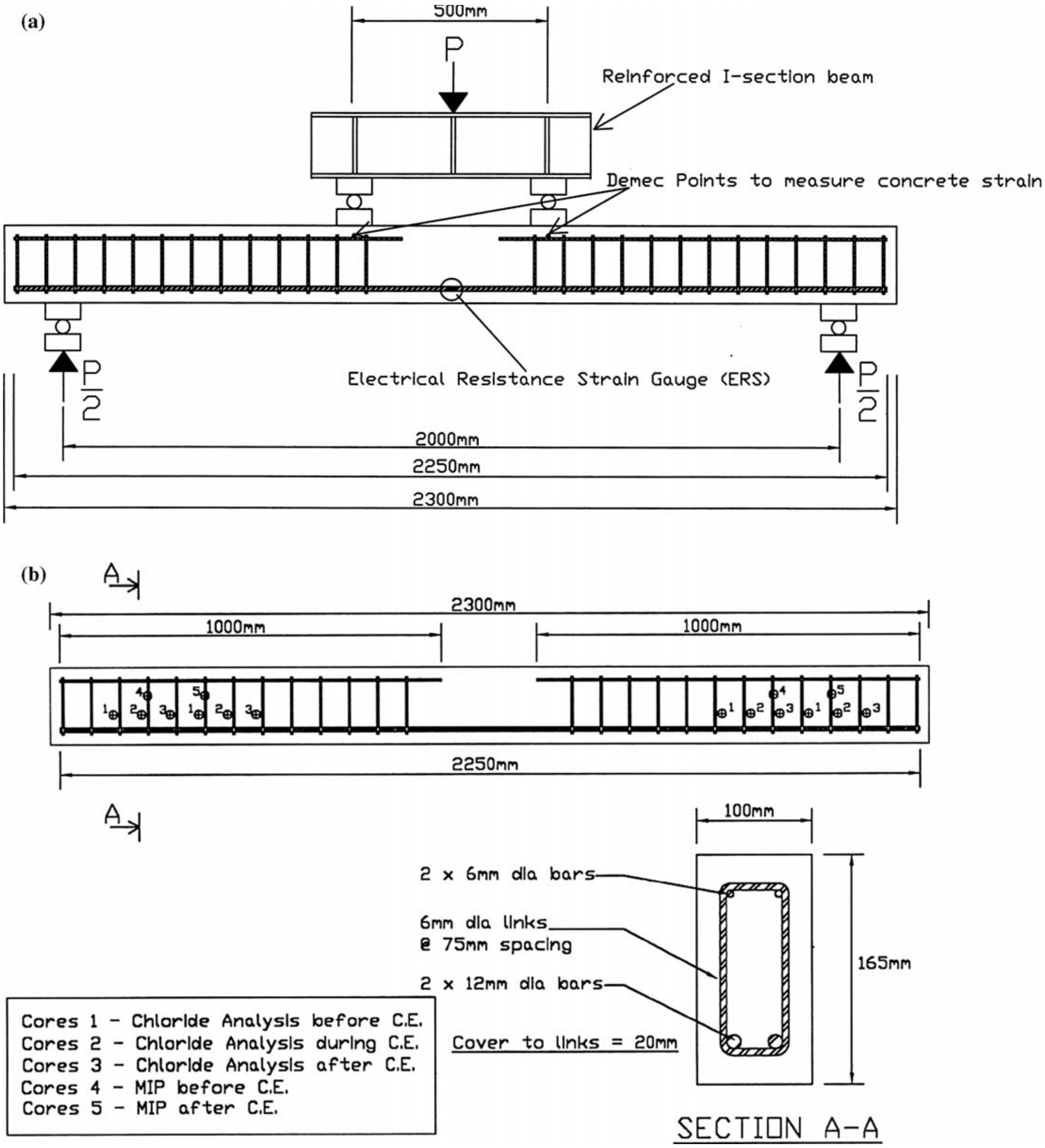

Fig. 1. Details of test beams and locations of cores for chloride and pore structure analysis.

Table 1
Mix proportions, chloride and fused silica contents of test beams (kg/m^3)

Beam	CL (% mass of cement)	w/c	Cement	Water	Fine agg.	Coarse agg.	Chloride	Fused silica
Beam 0	0	0.6	383	230	893	792	–	–
Beam 0	0	0.75	350	263	590	1250	–	–
Beam 2.4	2.4	0.6	383	230	893	792	9.2	–
Beam 2.4	2.4	0.75	350	263	590	1250	8.4	–
Beam 6(1)	6(1)	0.6	383	230	893	792	23	–
Beam 6(1)	6(1)	0.75	350	263	590	1250	21	–
Beam 6(2)[a]	6(2)	0.6	383	230	893	792	23	–
Beam 6(2)[a]	6(2)	0.75	350	263	590	1250	21	–
Beam 6(5)	6(5% FS)	0.6	383	230	809	792	23	84
Beam 6(5)	6(5% FS)	0.75	350	263	498	1250	21	92
Beam 6(15)	6(15% FS)	0.6	383	230	640	792	23	253
Beam 6(15)	6(15% FS)	0.75	350	263	314	1250	21	276

[a] Beams not electrically connected for CE.

then demoulded and cured under wet hessian for a further 10 days, after which they were exposed to ambient dry environment. Six 100 mm cubes and three 100 × 100 × 500 mm prisms were cast with each beam to establish the concrete properties at various ages. The 28 day cube strengths of the concrete without chlorides and fused silica averaged 50.1 MPa and 35.6 MPa respectively for the concretes with 0.6 and 0.75 w/c. The beams were all instrumented to measure steel and concrete strains.

Chloride analysis of the concrete in each beam was carried out to investigate the effects and efficiency of chloride extraction. Four cores, 25.4 mm in dia × 100 mm long, were taken from the sides of each beam, adjacent to the stirrups as shown in Fig. 1. These cores were taken before CE, during CE, and finally at the end of the CE process, (Fig. 1). The chloride analysis was carried out according to Volhard's method as described in BS 1881, Part 124, 1988. The chloride content was then expressed as acid soluble chloride content by mass of cement. To have reliable and repeatable results, extensive trial tests were carried out on sample preparation, and other details of the test procedure for chloride analysis prior to the actual tests on the beams. These procedures have been reported, and are therefore not repeated here [6,7].

In addition to chloride analysis, the pore structure of the cover concrete of the beams was also determined, before and after CE. Mercury Intrusion Porosimetry (MIP) technique was used to determine the pore size distribution within the depth of the concrete cover of the beams, at the locations shown in Fig. 1. The cores for MIP test were 25.4 mm in dia × 20 mm long, and two cores were taken from each beam before and after CE. Again, extensive trial tests on MIP were carried out to standardize preparation of the samples for MIP, test procedure and interpretation of test data. These test results are, however, not reported.

After coring, the holes were cleaned and filled with mortar of the same w/c ratio as the concrete, and carefully compacted. The mortar was then kept moist for three days to ensure adequate curing. Trial tests showed that the permeability of the mortar fill was similar to that of the concrete, and that the influence of these filled mortar on the transport of chloride and hydroxyl ions was of the same order as the rest of the concrete beam.

2.3. Set-up for chloride extraction

Only 10 of the 12 beams were subjected to CE. Beams marked 6(2) in Table 1 were set-up for CE, but were not electrically connected – they were used as control specimens for all the beams containing 6% chlorides. When all the beams were sufficiently cured, the appropriate electrical connections to the reinforcing bar were made at the ends of each beam. The cores cut to expose the bars and make the connections were filled with a dense mortar mix, and cured. The beams were supported on 25 mm wooden spacers, each beam in a specially made water tight 6 mm thick plastic tank, 200 × 200 × 2500 mm in size. To keep the anode equidistant from the beam, 25 mm wooden spacers were placed between the beam and the anode at quarter points.

In the electrochemical CE process used in this study, the anode system consisted of oxide coated platinised titanium mesh with saturated calcium hydroxide solution as electrolyte. The applied cathodic current density was approximately 1 A/m^2. The titanium mesh anodes, 180 × 2300 mm in size, were used to cover the sides of each beam. For practical reasons, the top and undersides of the beams were not subjected to CE. To add extra rigidity to the anode mesh, it was inter-woven into a plastic geotextile; these formed cartridges which could be placed between the wooden battens and the sides of the tanks. To make appropriate connections, a titanium bar was welded to the mesh at the end of each anode. To help current distribution, the anodes were positioned in each tank so that the connecting bars were not at the same ends. With everything in place, the system was wired up ensuring that all cable lengths were approximately equal. Connections to each anode were made by tying well with wire, covering in silicone, and then wrapped in insulation tape.

A Transformer Rectifier, Martech RD2 unit and multiplexer, was used as the power unit for the CE system. The RD2 had four terminals, each having a cathode and anode connector, clearly marked to ensure that the DC current is passed in the right direction. The current to each terminal was controlled by a Martech software run on a 486 PC, linked to the transformer rectifier unit. In the tests reported here, the beams were connected to the terminals according to the w/c ratio than according to the chloride or fused silica content. The RD2 power unit was provided with built-in data logging facilities of current, resistance and voltage.

In addition to this determination of the chloride profiles before, during and after the CE, the rate of chloride removal during the CE treatment was also monitored by chemical analysis of the $Ca(OH)_2$ anolyte. The pH of the electrolyte was also determined regularly to monitor the chemical reactivity of the beams and to ensure that the formation of chlorine gas did not occur. The total voltage and the current supplied to each beam were also carefully and regularly monitored throughout the CE process, and appropriate steps taken to correct any deviations as and when necessary.

After setting up the beams as described above, the tanks were filled with saturated calcium hydroxide to a level reaching to the top of the beams. The RD2 transformer rectifier unit was then powered up, and connected to the 486 PC which contained the 'Martech' software for controlling and monitoring the current distribution. The CE process was carried out over an 8 week period. Table 2 shows the timetable of events that took place over this period. There were two interruptions to the CE process – from days 21 to 29, and then again from days 33 to 38. This latter stoppage was to allow coring of the beams for chloride analysis followed by filling the holes and curing the filled mortar.

Table 2
Timetable of events during CE

Day	Process Notes
0	Powered up system, fresh electrolyte
1	All electrolytes changed chlorine gas detected
2	Changed electrolytes of 6% chloride, 5% FS, w/c = 0.75; 6% chloride, 15% FS, w/c = 0.75 (as explained)
3	All electrolytes changed
6	First resistor change to balance current distribution. All electrolytes changed
9	All electrolytes changed
15	All electrolytes changed
18	Electrolytes in each tank were agitated
21–29	Pause – Department closure
29	Resumption of CE
33–38	Stopped for chloride sampling
40	Electrolytes in each tank were agitated
42	Electrolytes in each tank were agitated
44	All electrolytes changed
51	All electrolytes changed
59	End of CE

3. Test results and discussion

From the large amount of test data obtained from these tests, only those relevant to this paper are reported here.

3.1. Changes in pH

The pH of the calcium hydroxide electrolyte was regularly monitored to check the rate of chemical reactivity and the formation of chlorine gas. When the CE equipment is powered up, the chlorides in the pore water system in the concrete will migrate away from the beam reinforcement, which acts as a cathode, towards the titanium mesh placed in the $Ca(OH)_2$ electrolyte adjacent to the beam, which acts as the anode. The chemical reactions at the cathode and anode lead to the generation of chlorine gas.

The pH values of the electrolyte in this study were measured from around each tank, and the average values recorded. During the first week, the pH values were found to be not uniform. This was rectified by connecting a second titanium bar at the other end of the anode section, which enabled the pH levels of the electrolyte to remain fairly uniform and consistent throughout the CE process.

Fig. 2 shows an overview of the pH profiles of the electrolytes in which all the beams were kept throughout the CE process. For ease of comparison, all the profiles are shown on the same figure, from a value of 12, the pH of the saturated calcium hydroxide, to zero. The beams in the figure are identified by their w/c ratio, and chloride/FS contents. It needs to be emphasized here that in spite of the excellent nature of the set-up of the electrochemical process, there was considerable fluctuation in the pH values both with time and from location to location around each beam. This is considered to be due to the inherent nature of the RC system – namely the intrinsic heterogeneity both of the permeability of the concrete and the distribution of the chloride concentration along the length of the beam. These heterogeneities are likely to be accentuated in the field in beams of several metres span. The data presented in Fig. 2 should therefore be only considered as a schematic representation of the overall trend of the pH values during the CE process rather than absolute values at any given time and at any given location.

An overall review of these pH profiles shows that four of the beams had much less pH activity than the others. For example, the 6(2)% beams, containing 6% chlorides but not subjected to CE (marked A and B in Fig. 2), had a stable pH of 12 throughout the CE period as would be expected. Similarly, the two beams with no chlorides but subjected to CE (marked C and D in Fig. 2), had also a relatively stable pH throughout the CE period, with only negligible fluctuations of the profiles during this time. These four profiles confirm the reliability and consistency of the pH measurements reported here.

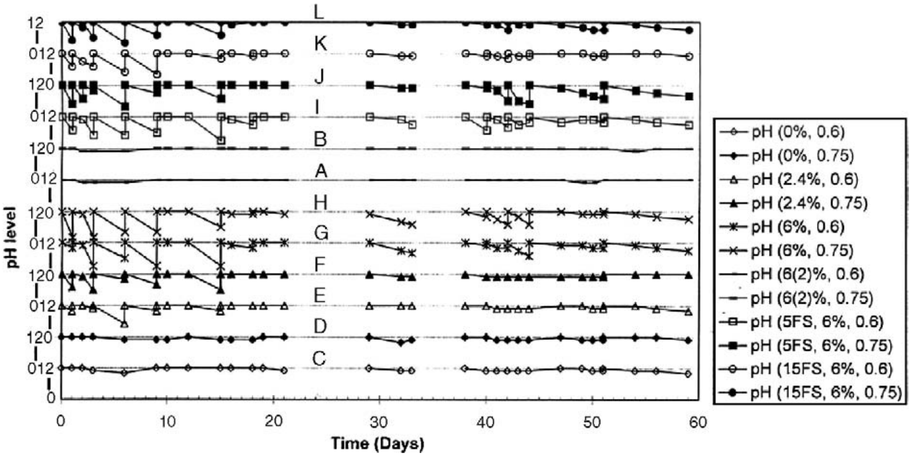

Fig. 2. pH profiles of electrolyte during the CE process.

All the other beams containing chlorides and subjected to CE do show considerable pH activity during the 8 weeks of CE. The profiles shown in Fig. 2 confirm that the changes in pH were highly pronounced for the first 15–20 days in all the eight beams (marked E to H and I to L) containing chlorides and subjected to CE. The beams with 6% chlorides, however, continued the chemical reactivity in various degrees for almost all of the 8 weeks, primarily because of their large chloride content. In beams with 2.4% chloride content (marked E and F in Fig. 2), for example, the lowest pH values measured were about 5 and 6, whereas in beams with 6% chloride content (marked G and H in Fig. 2), pH values as low as 3 and 4 were measured. In beams with chlorides and fused silica (marked I to L in Fig. 2), the lowest pH values measured were of the order of 4 and 5.

To appreciate the problems involved in pH measurements a brief description of what happened in the first two weeks of the CE process is given below. On day 1 all the electrolytes were changed when the pH values of the active beams with chlorides had dropped (Fig. 2, Table 2). And because of the low pH values, chlorine gas was formed, and this was detected by smell in the laboratory. The changes in the pH values in these beams were substantial as shown in Fig. 2. Each time the electrolyte was changed as shown in Table 2, the pH value of the fresh electrolyte returned to a value of 12. On day 2, the pH of the beams containing FS with 0.75 w/c ratio dropped significantly, and so the electrolyte of these beams was changed (Table 2). It is also worth noting from Fig. 2 that before the very first electrolyte change, the pH of the beams with w/c = 0.75 decreased more than that of the corresponding beams with w/c = 0.60. This is as expected, because of the greater permeability of the concrete and the resulting higher rate of release of chlorides and ionic movement, but no such obvious trends were evident thereafter.

The profiles in Fig. 2 also emphasize three other aspects. Firstly, beams with 6% chloride content were active much longer than those with lower salt content. Secondly, throughout the CE process, the pH values in beams with 6% chlorides dropped down more quickly than those in beams with 2.4% chlorides. And finally, beams with reactive aggregates in addition to chlorides were equally vulnerable to changes in pH as beams without such aggregates. Beams with the same w/c ratio had similar drops in pH values irrespective of the FS content in the first 10 days after which the 5% FS beams showed greater changes in pH. On the other hand, after the first two weeks, the 15% FS beams showed very little change in pH. All these happened throughout the CE process, and are again to be expected as the increased release of the chlorides into the electrolyte will increase its reactivity at the anodes, and reduce the pH faster.

A critical evaluation of the actual variations of the pH values summarized in Fig. 2 emphasizes the need for close monitoring of the pH variations in the field where chloride concentrations and concrete permeability can vary from location to location in a long span beam.

3.2. Chloride profiles

The chloride analyses reported in this paper were carried out according to BS 1881 on cores cut from the beams at locations shown in Fig. 1. The cores were then sliced into pre-determined sections of 0–5, 5–35, 35–65, 65–95 and 95–100 mm, and the amount of chloride content by mass of cement in each section determined. In carrying out the chloride tests, the samples prepared from the 0 to 5 mm and 95 to 100 mm were mixed together as they had the same exposure to the electrolyte and CE. The same procedure was adopted for the samples obtained from 5 to 35 mm and 65 to 95 mm depths. The total chloride content was measured before, during (half way through) and after CE, and the numerical results are shown in Figs. 3–14.

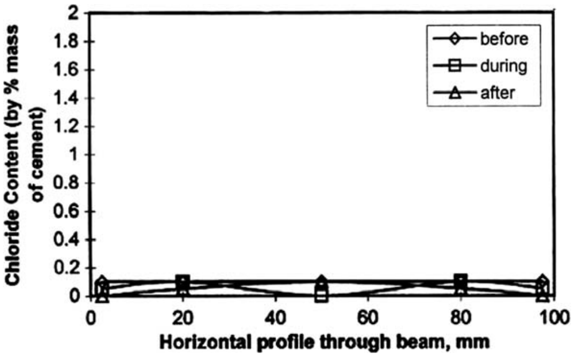

Fig. 3. Chloride profiles for beam with 0% chlorides and w/c = 0.60.

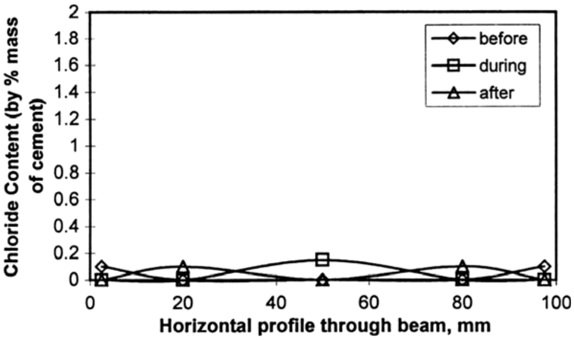

Fig. 4. Chloride profiles for beam with 0% chlorides and w/c = 0.75.

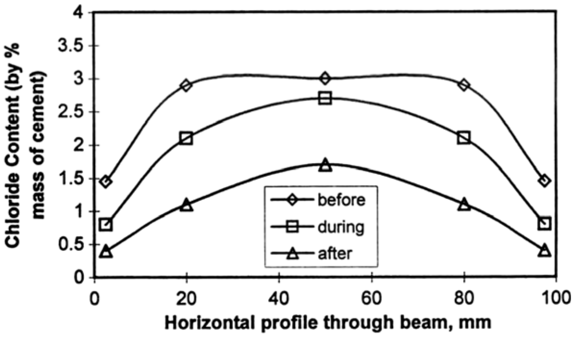

Fig. 5. Chloride profiles for beam with 2.4% chlorides and w/c = 0.60.

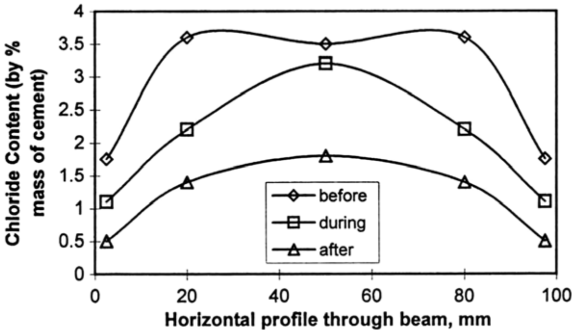

Fig. 6. Chloride profiles for beam with 2.4% chlorides and w/c = 0.75.

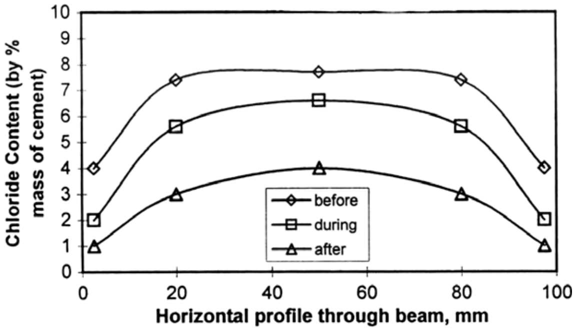

Fig. 7. Chloride profiles for beam with 6% chlorides and w/c = 0.60.

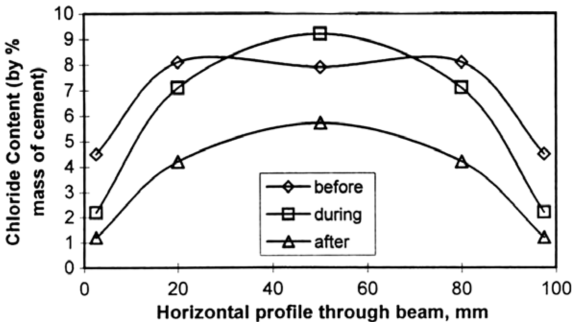

Fig. 8. Chloride profiles for beam with 6% chlorides and w/c = 0.75.

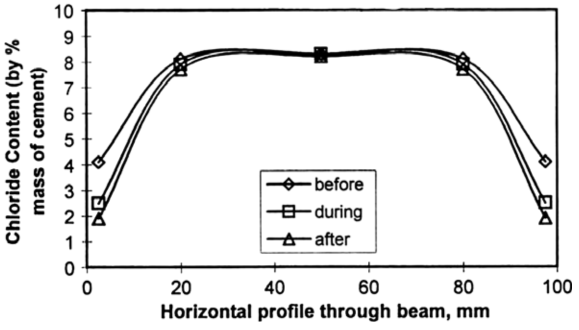

Fig. 9. Chloride profiles for beam (not electrically connected for CE) with 6% chlorides and w/c = 0.60.

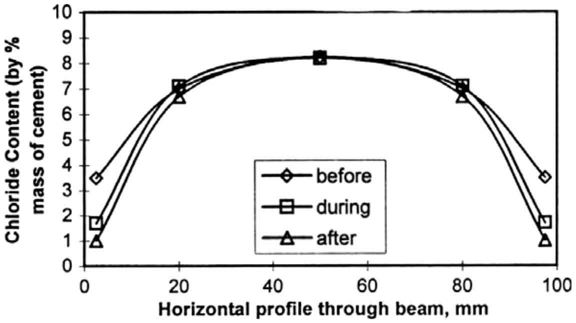

Fig. 10. Chloride profiles for beam (not electrically connected for CE) with 6% chlorides and w/c = 0.75.

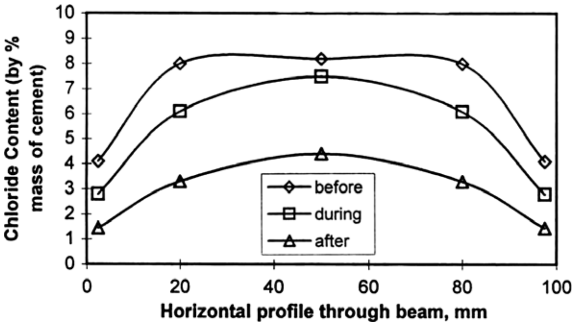

Fig. 11. Chloride profiles for beam with 6% chlorides, 5% FS and w/c = 0.60.

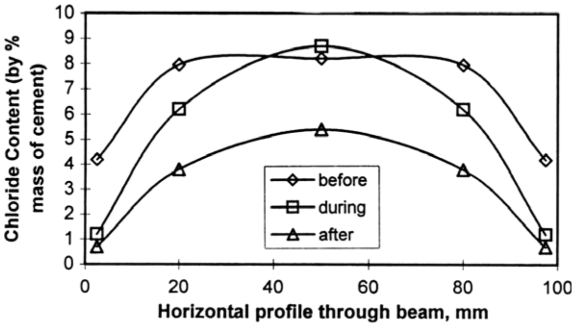

Fig. 12. Chloride profiles for beam with 6% chlorides, 5% FS and w/c = 0.75.

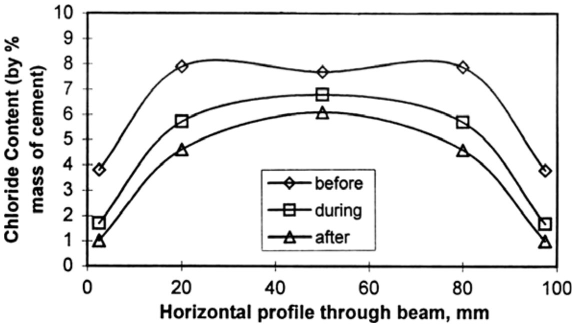

Fig. 13. Chloride profiles for beam with 6% chlorides, 15% FS and w/c = 0.60.

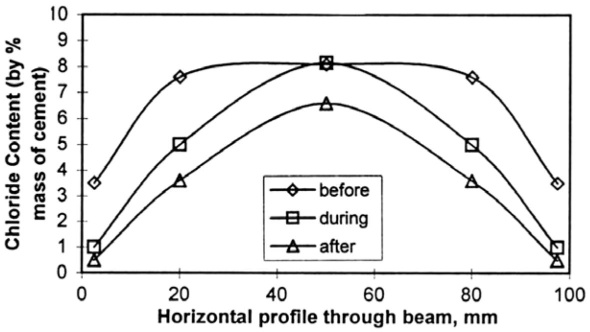

Fig. 14. Chloride profiles for beam with 6% chlorides, 15% FS and w/c = 0.75.

A review of all the data presented in Figs. 3–14 shows two important results. Firstly, the accuracy, consistency and reliability of the test method used in this study for chloride content analysis is clearly demonstrated by Figs. 3, 4, 9 and 10. The chloride amounts obtained from beams with 0% chlorides shown in Figs. 3 and 4 were all very low, as expected, less than 0.15% by mass of cement. On the other hand, the chloride analysis results from the 6(2)% beams which contained 6% chlorides but not subjected to CE (Figs. 9 and 10) changed very little, except at the outer surfaces of 0–5 mm and 95–100 mm of the beams, where the values after the CE period were less than 50% of the original values. This decrease in chloride content at the outer surfaces of the beams is due to leaching and migration of the chlorides after casting and surface precipitation when exposed to the electrolyte. The significance of Figs. 3, 4, 9 and 10 is two fold – they not only serve as a reference data for the rest of the beams, but they also emphasize that it is not the absolute values of the chloride that matter but that it is an accurate representation of chloride content changes showing the effectiveness of CE.

In contrast, all the other beams which contained chlorides and were subjected to CE showed a significant reduction in chloride content throughout the cross-section of the beams, and particularly at the outer surfaces, after the CE process. However, it is important to note that the distribution of the chloride contents in the cross-section varied in each beam as well as from beam to beam not only during the CE process but also at the end of the CE period. The final chloride contents in each beam are obviously a function of its initial conditions, and the fact that, however identical, each beam will behave differently due to the very nature of concrete as cast, and the resulting microstructural characteristics, and chloride penetration.

The chloride analysis data in Figs. 5 and 6 which relate to beams with 2.4% initial chloride content and Figs. 7 and 8 which relate to beams with 6% initial chloride content show conclusively that the CE process has been efficient and effective in removing the chloride from the concrete cover and from around the reinforcement at both 0.6 and 0.75 w/c ratios. In these four beams (Figs. 5–8), the reduction in chloride content in the concrete cover was the highest, varying from some 70% to 75% at the end of the CE process to about 50% to 60% in the zone containing the tension and compression reinforcement. However, about 50–75% of the initial chloride content remained in the mid-section of three of the beams (Figs. 5–8). The beam with 6% chlorides and 0.75 w/c ratio was the least efficient in this respect at both the end as well as during the CE process. There was substantially greater migration of the chloride ions into the middle of this beam (Fig. 8) compared to that in the beam with 0.60 w/c ratio (Fig. 7) [27]. Indeed, during halfway through the extraction process, the chloride content at the mid-section of this beam was higher than the original value prior to the CE process. A similar phenomenon also occurred in two other beams with 6% chloride content and 0.75 w/c ratio, and containing 5% (Fig. 12) and 15% FS (Fig. 14). A possible explanation to this unexpected chloride migration process is the following. During the initial CE process, the chloride ions in the concrete cover are repelled away from the reinforcement, which acts as a cathode, in two opposite directions, into the external electrolyte and into the internal section of the beam. The rate of the chloride migration to the external electrolyte will be faster because of the anode, while the chlorides behind the reinforcement will tend to be pushed towards the middle section of the beam until a substantial amount of chloride is removed from the cover concrete. The ensuing chemical and physical imbalance, and the presence of a more active electrical field will then cause the chloride ions behind the reinforcement to migrate out towards the anode creating a more equilibrium state of chloride ion distribution, without adversely affecting the chloride ion distribution in the

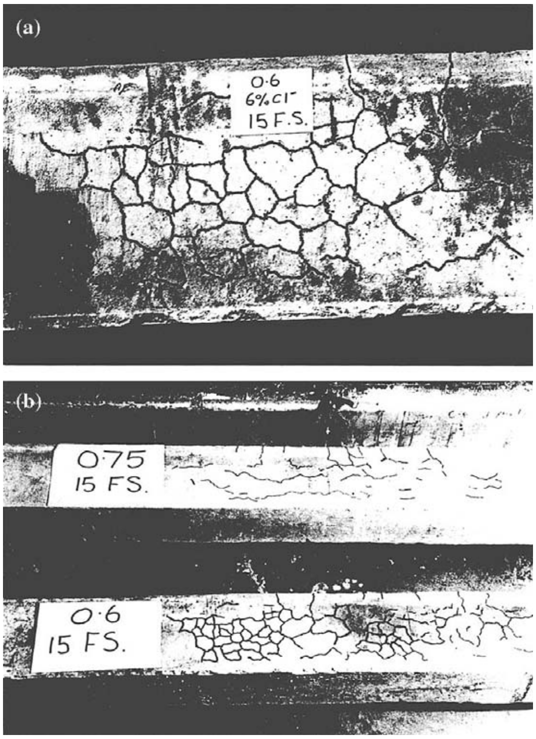

Fig. 15. ASR cracking in beams with 6% chlorides and 15% FS (a) beam with w/c = 0.60 (b) beam with w/c = 0.75.

cover zone. In these beams, prolonged CE treatment may be necessary to drastically reduce the chloride levels in the concrete cover and behind the reinforcement.

The four beams with 6% chlorides and undergoing ASR (Fig. 15) due to the presence of FS reactive aggregates (Figs. 11–14) behaved generally similarly to the beams without ASR (Figs. 7 and 8). The reduction in chloride content at the end of the CE period in the concrete cover zone in these beams (Figs. 11–14) varied between 65% and 85%, and between 40% and 60% in the zone containing the reinforcement bars. Some 55–80% of the initial chlorides, however, remained in the mid-section of these beams after the CE test. The wider range of the residual chloride contents in these beams after the CE test compared to those in the beams without FS must be due to the internal and surface microcracking arising from the alkali silica reactivity, and the changes in the pore structure due to chloride penetration [28].

An overall review of the chloride profiles in Figs. 3–14 shows that the efficiency of removal of chlorides was neither consistent nor uniform as the w/c ratio of the concretes i.e. concrete permeability, or the amount of chlorides, or chlorides + FS varied. In general terms, chloride removal from the cover concrete was most efficient followed by that in the concrete zone containing the reinforcing bars. One striking feature is that in the beams with 0.75 w/c ratio and containing chlorides and chlorides + FS, the chloride content in the mid-section was, halfway through the CE process, higher than the initial amount before CE (Figs. 8, 12 and 14). The implication here appears to be that the high w/c ratio and the high amount of initial chlorides present (6%) were instrumental in creating a more effective current flow in these beams, enabling the shifting of more chlorides with ease. A comparison of the data given in Fig. 2 with those in Figs. 3–14 also shows that there is a general and qualitative correlation between the variations in the pH profiles of the electrolyte and the chloride removal activity in the various beams subjected to CE.

A critical examination of Figs. 3–14 also shows that the amount of chlorides determined from the chloride tests before CE are, in all beams, except at their surfaces, well above the actual amounts added in the concrete mix. This is considered to be due to the methodology of taking dust samples for chemical analysis from the beams by coring rather than by drilling. Both these techniques have their advantages and limitations, but the results obtained from the cores and shown in Figs. 3–14 are consistent with those obtained from a large number of tests carried out with both techniques over many years [6,7]. Whatever technique of extracting the dust samples is used for quantitative chloride analysis, it is emphasized that the amounts of chlorides shown in Figs. 3–14 should not be considered as totally absolute values, but rather as representative of the chlorides added to the concrete mix and the effectiveness and efficiency of CE.

It must be emphasized that in the tests reported here the chlorides were cast in, and the chlorides clustered behind the reinforcement will make CE less efficient. In practice, chlorides penetrating from deicing salts and sea water would largely remain in the concrete cover, and CE would then be a lot easier and more efficient in bringing chloride levels down to safer values near the reinforcement.

3.3. Effect of CE on ASR

Four of the RC beams tested in this project were designed to have ASR during the CE process (Table 1). Based on previous studies [24–26], fused silica was used as reactive aggregate in the concrete mix used to cast these four beams. The beams, together with the control cubes and prisms, were exposed to a dry indoor environment after wet curing for 12 days. At the time of carrying out the CE process on the beams, the cubes and prisms kept in a dry environment showed no visible signs of cracking due to ASR. Subsequent visual inspection up to about 400 days simply confirmed this.

The four RC beams were, however, placed in the electrolyte for 8 weeks of CE. Alkali hydroxides are formed at the steel surface during the CE process, which thus provides both additional alkali and moisture to the beams containing a significant amount of reactive silica. To investigate if the CE process of 8 weeks was long enough to provoke ASR, the beams were visually examined for cracking at the end of CE. Of the two beams with 5% FS, the beam with 0.75 w/c ratio did not show any visible cracking, whilst only very close inspection of the beam with w/c = 0.60 revealed very slight cracking in the middle part of the beam which had no stirrups. In particular, there was no ASR microcracking at the tension steel level. There was clear evidence, however, of the cracks being filled up with ASR gel in this beam.

Both the beams containing 15% FS showed very clear signs of cracking at the end of the CE process. The beam with w/c = 0.6 exhibited extensive map cracking in the mid-section of the beam, as shown in Fig. 15(a) with white exudations of the gel product oozing out of the cracks. The beam with w/c = 0.75, on the other hand, showed only slight ASR cracking; most of the cracks were parallel to the tensile reinforcement interconnected by short microcracks which could eventually develop into a map crack pattern as shown in Fig. 15(b).

A careful examination of all the beams showed that ASR cracking did occur in three of the four beams with FS. The precise role of the hydroxyl ions generated at the reinforcement by the CE process in activating ASR is not clear. However, CE was directly involved in furthering ASR by providing moisture to provoke ASR. There was clear evidence that the reinforcement cage in the two shear spans contributed to a significant restraining effect against ASR cracking, as most of the visible cracking was concentrated in the middle part of the beams where no stirrups had been provided [26].

The control cubes and prisms kept in a dry environment showed no signs of ASR cracking. The 28 day cube strengths of concrete without and with FS were also unaffected, except the concrete with 15% FS and 0.6 w/c ratio which showed a strength loss of about 30%.

3.4. Beam ultimate strength

The aim of this part of the paper is to evaluate the flexural strength and safety implications of CE rather than to provide detailed descriptions of structural behaviour and structural analysis of beams failing in flexure (which are well established) after being subjected to CE without or with ASR.

After the CE process, all the 12 RC beams were tested to failure at an age of about 400 days. The beams were tested over an effective span of 2.0 m, under four point bending with a shear span of 750 mm as shown in Fig. 1. All the beams failed in flexure by crushing of the concrete in the compression zone of the pure bending region, after large deflections and tensile steel yielding. All the beams showed crack patterns typical of beams failing in flexure. The four beams contains FS also showed a crack pattern similar to that of the other beams, and none of the beams showed any bond distress or any major shear cracks at failure.

The major results obtained from the tests, and pertaining to this paper are presented in Table 3. The table includes cube strength at 28 days and at 400 days – bearing in mind that the cubes were kept dry in the laboratory environment after initial curing. The other data in Table 3 relate to the failure load and the corresponding service load. Mu(test) shows the experimental ultimate moment of resistance of the beams at failure. Mu(theory) represents the theoretical ultimate moment of resistance based on conventional RC beam theory and the 400 day cube strength. Mu(design), also based on the 400 day cube strength, is the ultimate moment of resistance calculated with the appropriate factors of safety incorporated in the equations.

The results of the first six beams in Table 3 show that the 400 day cube strength decreased progressively with increase in chloride content, by amounts varying from about 5% to 25%. The maximum strength loss of about 25% occurred in the concrete containing 6% chloride and 0.75 w/c ratio. However, this loss in compressive strength had little influence on the failure loads of the corresponding beams. Beams 6(1)% and 6(2)% were identical in all respects, except that the former was subjected to CE whilst the 6(2)% beams were not subjected to CE. Again, the differences in strength between these two sets of beams are practically insignificant. All these results confirm that the effects of the short term process of CE are of no special significance, and that they have no detrimental effect on the structural strength of the beams.

Considering the beams with FS, the beams with 5% FS were generally stronger in flexure than beams with 15% FS, as would be expected. At the w/c = 0.6, the beam with 15% FS had about 15% lower ultimate strength than the beam with 5% FS, although the cube strength of the former was only about 60% of the latter. At the w/c ratio of 0.75, there was little difference in the cube strength and the ultimate strength of the beams with 5% and 15% FS. Comparing similar beams without and with FS, CE had little effect on the ultimate strength of beams at 5% FS content. However, at 15% FS level, CE had a slight adverse effect of about 10% at a w/c = 0.60. It should be emphasized that the four beams with reactive aggregates were tested in this study soon after 8 weeks of CE which is short term in the life of a RC structural member. There is every possibility that the effects of ASR initiated during this period may continue for a considerable time to come, and this may have adverse effects on the serviceability behaviour of the beams as reported earlier [26,27]. Further research is therefore needed to clarify the long term implications of ASR initiated by the CE process in beams containing reactive aggregates.

The last three columns of Table 3 confirm that short term CE and any resulting ASR activity has no adverse effect on the structural strength and structural performance of the 12 RC beams tested in this study. Indeed, in all cases there is an adequate factor of safety such that the application of CE in practice can be carried out in confidence without loss of strength, safety or structural stability. However, as mentioned earlier, the long term effects of CE and the resulting ASR need to be evaluated before the extended

Table 3
Test results related to beam failures

Beam – w/c	Cube st 28 d (MPa)	Cube st 400 d (MPa)	Ult load (kN)	Service load (kN)	Mu test (kNm)	Mu theory (kNm)	Mu design (kNm)	$\frac{\text{Mu(test)}}{\text{Mu(theory)}}\%$	$\frac{\text{Mu(theory)}}{\text{Mu(design)}}\%$	$\frac{\text{Mu(test)}}{\text{Mu(design)}}\%$
0%–0.6	50.1	60.9	39.5	17.5	14.8	12.5	10.5	118	119	141
0%–0.75	35.6	43.6	38.5	16.5	14.4	11.9	9.9	121	120	145
2.4%–0.6	48.1	58.4	41.2	17.4	15.5	12.4	10.4	125	119	149
2.4%–0.75	32.0	38.6	40.8	16.1	15.3	11.7	9.7	131	121	158
6(1)%–0.6	43.2	53.0	40.6	17.2	15.2	12.3	10.3	124	119	148
6(1)%–0.75	26.6	32.7	36.4	15.4	13.7	11.3	9.2	121	123	149
6(2)%–0.6	45.1	56.1	42.2	17.3	15.8	12.4	10.4	127	119	152
6(2)%–0.75	27.9	34.1	35.0	15.6	13.1	11.4	9.3	115	123	141
5% FS–0.6	45.5	48.8	41.2	16.9	15.5	12.1	10.1	128	120	153
5% FS–0.75	28.1	30.4	35.4	15.0	13.3	11.1	9.0	120	123	148
15% FS–0.6	30.1	30.3	35.2	15.0	13.2	11.1	9.0	119	123	147
15% FS–0.75	29.4	29.9	34.1	14.9	12.8	11.1	9.0	115	123	142

design service life of the beams in practice can be reliably assessed.

It also needs to be emphasized that the beams subjected to CE in this study were normally designed beams, safe in shear and bond. Beams which are weak in shear would need structural strengthening as well as CE, and such beams would need specialist evaluation before plans for CE are carried out [29]. As far as the effects of CE on bond are concerned, some initial drop in bond strength due to removal of corrosion products has been reported [21], but these results are based on pull-out tests which have little or no engineering significance or relevance to the state of the bond in RC beams. Further, results obtained from bond tests on cylinders 254 mm long [21] cannot be extrapolated to RC beams.

4. Conclusions

The following major conclusions are derived from the research study reported in this paper:

1. All the beams containing chlorides without or with reactive aggregates showed pronounced changes in the pH value of the electrolyte in the first 15–20 days after commencing the CE process. Beams with 6% chlorides continued this chemical reactivity throughout the 8 weeks of CE. There was a general and consistent inter-relationship between changes in the pH value of the electrolyte and the chloride removal activity.
2. To ensure the prevention of the formation of chlorine gas, and that the pH of the electrolyte does not go down too much, it is essential to monitor the pH of the electrolyte regularly and throughout the CE period.
3. A large amount of chlorides was removed from the concrete into the external electrolyte in the first 15–20 days of the electro-chemical treatment, after which CE remained more or less steady. The peak value for removal of chlorides was day 3, probably due to the time required for a build-up of hydroxyl ions around the reinforcement before effective removal.
4. The electrochemical CE process was effective and efficient, and a CE period of 8 weeks removed a high proportion of the chlorides from the concrete cover and from around the reinforcement at both 0.6 and 0.75 w/c ratios. In the beams without reactive aggregate, the reduction in chloride content in the concrete cover amounted to some 70–75%. In the zone containing the tension and compression reinforcement, the reduction was about 50–60%. In the beams containing reactive aggregates, the range of chloride reduction was slightly higher in these two zones, probably due to internal microcracking and changes in pore structure due to ASR. Even in beams with 6% chlorides stored in the electrolyte without current supply, some 50–70% of the initial chlorides were leached out of the concrete cover by simple diffusion. However, in beams with high initial chloride content and high chloride mobility, the long term effects of CE need further study.
5. The removal of chlorides from the mid-section of all the beams was, however, less efficient as some 50–80% of the initial chlorides still remained in this part at the end of the CE process.
6. The role of the electric current supplied to the reinforcement during CE in initiating ASR needs to be further investigated. In the study reported here, the CE process as a whole did provoke the dormant alkali silica reactivity in the beams containing fused silica reactive aggregate. It is thought that the electrolyte and the concrete pore solution have important roles in the initiation of ASR.
7. The ultimate strengths of the beams in flexure, designed to be safe in shear and bond, were not affected to any significant degree either by the short term CE process or by the amount of chlorides in the beams. All the beams showed conventional flexure failures without any visible distress either in bond or shear. There was, however, some loss in strength of about 10% in the beam with 15% FS due to loss in concrete compressive strength arising from ASR.
8. All the 12 beams exposed to CE showed adequate factors of safety both in terms of structural strength, structural performance and safety.
9. The overall evidence from the tests reported here is that electrochemical CE applied to RC beams is an efficient and economic process without adverse structural implications either on their failure strength or structural performance. Unlike cast-in chlorides which tend to be concentrated behind the reinforcement, the CE process is likely to be more efficient in real structures where the majority of chlorides that have penetrated the concrete would remain in the concrete cover.
10. Further research is needed to establish the long term effects of ASR initiated by the CE process, and the practical and structural implications if the CE is carried out in the field when the structural members carry dead and live loads.

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