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Influence of service loading and the resulting micro-cracks on chloride resistance of concrete

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HIGHLIGHTS

- Effect of microcracks due to sustained loading on chloride transport into concrete was obtained.
- Stress levels above 50% of the ultimate can cause a great increase in chloride ingress.
- Mineral additives perform well against chloride ingress in concrete subjected to sustained loading.
- At high stress levels, removal of the load results in a recovery of the chloride resistance.

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ABSTRACT

Chloride-induced corrosion of steel in reinforced concrete structures is one of the main problems affecting their durability, but most previous research projects and case studies have focused on concretes without cracks or not subjected to any structural load. Although it has been recognised that structural cracks do influence the chloride transport and chloride induced corrosion in reinforced concrete structures, there is little published work on the influence of micro-cracks due to service loads on these properties. Therefore the effect of micro-cracks caused by loading on chloride transport into concrete was studied. Four different stress levels (0%, 25%, 50% and 75% of the stress at ultimate load – f_u) were applied to 100 mm diameter concrete discs and chloride migration was measured using a bespoke test setup based on the NT BUILD 492 test. The effects of replacing Portland cement CEMI by ground granulated blast-furnace slag (GGBS), pulverised fuel ash (PFA) and silica fume (SF) on chloride transport in concrete under sustained loading were studied. The results have indicated that chloride migration coefficients changed little when the stress level was below 50% of the f_u; however, it is desirable to keep concrete stress less than 25% f_u if this is practical. The effect of removing the load on the change of chloride migration coefficient was also studied. A recovery of around 50% of the increased chloride migration coefficient was found in the case of concretes subjected to 75% of the f_u when the load was removed.

1. Introduction

Chloride-induced corrosion of steel in reinforced concrete structures is one of the most serious problems affecting their service life and if unattended it can put the infrastructure at risk and endanger people's lives. An understanding of the processes of chloride transport in concrete is very important for engineers attempting to predict the service life of reinforced concrete structures and numerous studies dealing with this topic can be found in the literature [1–9]. However, most of them have focused on concretes which are neither cracked nor subjected to any structural loading. The work by Gowripalan et al. [10] showed that chloride diffusivity of concrete in the compression zone was much smaller than that in the tension zone when concrete beams were subjected to flexure. Win et al. [11] suggested that chloride can readily penetrate into the concrete through an open crack. Other studies on the influence of crack width on chloride transport [12–15] showed that the influence of micro-cracks on chloride ingress can be significant.

Although it has been recognised that structural cracks do influence the chloride transport and chloride induced corrosion of steel in reinforced concrete structures, there is little published work on the influence of micro-cracks due to service loads on these properties. In some studies [12,16–21] chloride transport

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tests on concrete have been conducted after the load was removed, but the results of concrete under loading and after the load is removed can be different. Based on tests conducted under sustained compressive loading, Antoni et al. [22,23] reported a significant increase of chloride transport into plain concrete when the compressive stress level was above 50% of the ultimate stress – f_u. In contrast, based on a numerical simulation a linear decrease of chloride diffusion in concrete under sustained compressive stress from an unloaded state up to 50% f_u was reported by Wang et al. [24]. It was found that some conflicting views exist in the literature on critical stress level and chloride transport properties of concretes under different loading conditions. Further, most of the work focused on concrete containing only Portland cement CEMI [24,25], and data on concrete with mineral additives are still limited.

The existing service life prediction models, such as Life-365 [26,27], AGEDDCA model [28] and DuraCrete [29], do not consider any effect of loading or consequential cracking (macro-cracking or micro-cracking; micro-cracks are defined below as cracks <0.1 mm wide), hence the predicted service life may not be reliable for most structures in service. The effect of stresses and micro-cracks on chloride transport is not considered in the current Standards. The Eurocode 2 [30] restricts the crack width to 0.3 mm for reinforced concrete in chloride-laden environments but has no restrictions on concretes with small cracks (<0.3 mm) or micro-cracks (<0.1 mm). As a consequence, premature deterioration of structures in service is common. For example, on the basis of a field survey of 57 bridges in Kansas, USA, Lindquist et al. [31] have indicated that the chloride concentration at a depth of 76 mm from the location of a surface crack can exceed the corrosion threshold amount within the first year. Another survey of 219 marine structures along the Norwegian coastline has shown that the signs of corrosion could be seen as early as 5–10 years, which means that the time for corrosion initiation was even earlier [32]. Almost all of these structures required major repair at the age of 20 years, which is significantly earlier than their design life of 100 years.

Therefore, the influence of micro-cracks due to structural loads on the chloride transport in concrete was investigated and thereby recommendations for any change of service life designs of such structures are made. As concrete structures exposed to chloride environments are manufactured with cementitious materials containing mineral additives, the effect of such cementitious materials on micro-cracking under sustained loading and its consequential influence on chloride transport were part of the research reported in this paper.

2. Materials and test methods

2.1. Materials

The coarse aggregate used was crushed 5–20 mm well graded basalt (size 5–10 mm 33% and size 10–20 mm 67%) and the fine aggregate was natural medium graded sand conforming to BS EN 12620:2002 [33]. Both types of aggregates were oven-dried at 100 ± 5 °C for 24 h to remove the initial moisture content and then allowed to cool down to room temperature (~18 °C) in air-tight containers before being used. Class 42.5 N Portland cement CEMI conforming to BS EN 197-1:2011 [34] was used. Ground granulated blast-furnace slag (GGBS) complying with BS EN 6699: 1992 [35], pulverised fuel ash (PFA) complying with BS EN 450 [36] and silica fume (SF) conforming to BS EN 13263 [37] were used to replace part of CEMI in the mixes studied.

The proportions for each mix, along with their physical properties, are reported in Table 1. As can be seen from Table 1, some of the mixes were added with a polycarboxylate polymer-based superplasticiser conforming to BS EN 934-2 [38]. The solid content of the superplasticiser was 29%. The water content of superplasticiser was considered to be part of the mixing water and hence this quantity was deducted from the water content reported in Table 1. Water from the mains at a temperature of 20 (±1) °C was used to manufacture all mixes.

2.2. Manufacture of test specimens

Four blocks (250 × 250 × 80 mm) and six 100 mm cubes were cast for each mix. The required quantities of the mix constituents were batched by mass and then mixed using a pan mixer in accordance with BS 1881-125: 1986 [39]. The slump was measured immediately after batching the concrete. The concrete was placed in the moulds in three layers and each layer was compacted using a vibrating table until air bubbles appearing on the surface stopped. The cast surface of each concrete specimen was then finished with a trowel and covered by a thick black polythene sheet to prevent any loss of water. The samples were demoulded after 24 h, they were then placed in a water bath at 20 (±1) °C for 3 days, after which they were wrapped in wet hessian and thick black polythene sheet. The specimens were then placed in a constant temperature room at 20 (±1) °C.

2.3. Test methods

2.3.1. Compressive strength test

The cubes were used to carry out the compressive strength test at an age of 28 and 56 days for each mix, the results of which have already been reported in Table 1.

2.3.2. Chloride migration test

Cores of size 100 mm diameter and 80 mm thickness were cut from the blocks at an age of 56 days and the outermost, approximately 15 mm thick layer, was cut off from the two end surfaces using a water-cooled diamond saw. This was done to eliminate the effects of the boundary layer on chloride transport and ensure a smooth test surface of the samples. The middle part of 50 (±1) mm was thus used as the test specimen. The thickness of each specimen was measured with a digital calliper to an accuracy of 0.1 mm. The chloride migration test was carried out using these disks. In order to test them under sustained loading, they were compressed by two diametrically opposite loads by using a special setup that was designed and manufactured, as shown in Figs. 1 and 2. This test setup is novel in the sense that it was not used before and was designed to stimulate micro-cracking of reinforced concrete under the sustained loading condition.

Samples were saturated as per the preconditioning procedure outlined in NT BUILD 492 [40]. The cores were placed in a vacuum container after they were surface-dried, by making sure both end surfaces were exposed. The pressure inside was reduced to 30 (±5) mbar and the vacuum was maintained for three hours. Then the container was filled with $Ca(OH)_2$ solution to immerse all specimens, with the pump still running. The vacuum was maintained for a further hour before allowing air to enter. The cores were kept in the solution for another 18 h, after which they were taken out for placing in the test setup. The cores were then coated on the curved surface with a silicon coating material and clamped in position (Fig. 2). After the silicon had hardened, the specimens were loaded at a constant rate of 1.23 (±0.1) kN/s until failure. The average load of the three failure loads in each mix was taken as the maximum load and the related stress was taken as the ultimate stress – f_u for each mix. The typical crack pattern at the failure of the core specimen is shown in Fig. 3.

Three loading conditions (Control – no loading, under loading and after unloading) were utilised. Fig. 4 shows that cores under different loading conditions were treated separately before testing. The load was removed after 1 min of loading for the After Unloading condition and UPV tests were conducted before the load was applied and after the load was removed. The load was kept constant throughout the test period for the under loading condition, UPV tests were conducted before and after the load was applied. No load applying and UPV tests for the Control condition. Different stress levels (equivalent to 25%, 50%, and 75% of f_u) were applied to the cores by fastening the nuts at the top of the brackets using a torque wrench. Strain gauges placed on bolts were used to determine the forces in the bolts, indicated in Fig. 2. An ultrasonic pulse velocity (UPV) test was conducted before and after the load was applied.

- Control: Specimens were ready for testing and no UPV test was conducted.
- Under loading: Different levels of load were applied and maintained for the tests conducted. An UPV test was conducted before and after the load was applied.
- After unloading: Different levels of load were applied and kept constant for 1 min, and then the load was released. UPV tests were conducted before applying and after releasing the load.

As shown in Fig. 3, cracks along the direction of loading were formed when the cores were under a certain load. The ultrasonic pulse velocity (UPV) in 3 different directions (Fig. 5) was used to determine the damage degree θ, which is the change in UPV [41] as per Eq. (1) caused by the loading. As illustrated in Fig. 5, a direct method and two indirect methods were used to measure the UPV for all mixes. Exactly the same locations were utilised before and after each load was applied and vaseline petroleum jelly was used as a couplant between the transducer and the concrete surface. The transducers with a frequency of 54 kHz were pressed by