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Simulation of chloride penetration into concrete structures subjected to both cyclic flexural loads and tidal effects

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Abstract. Chloride induced corrosion is a concern that governs the durability of concrete structures in marine environments, especially in tidal environments. During the service lives of concrete structures, internal cracks in the concrete cover may appear due to imposed loads, accelerating chloride penetration because of the simultaneous action of environmental and service structural loads. This paper investigated the effects of cyclic flexural loads on chloride diffusion characteristics of plain concretes, and proposed a model to predict the chloride penetration into plain concretes subjected to both tidal environments and different cyclic flexural load levels. Further, a new experiment was performed to verify the model. Results of the model using Finite Difference Method (FDM) showed that the durability of concretes in tidal environments was reduced as cyclic flexural load levels, SR, increased, and the modeling results fitted well with the experimental results.

Keywords: chloride penetration; cyclic flexural load; actual concrete structures; model.

1. Introduction

Chloride induced corrosion, which reduces the cross-section of reinforcement, is a major problem in the control of durability of concrete structures in marine environments. Therefore, resistance to chloride penetration is important to the design and construction of actual concrete structures subjected to simultaneous climate and service loads.

The chloride penetration into concrete is a complex phenomenon that depends on the material characteristics of concrete, environmental parameters, and especially the service loads during the service life time. In addition, in marine environments, the tidal zone is assumed to give rise to the most severe corrosion of concrete structures. In the tidal zone, the chloride penetration involves both diffusion of chloride ions which occurs due to ion concentration and movement of chlorides due to permeation of water into the concrete due to capillary suction, and consequently, the tidal zone conveys a greater quantity of chloride ions than the pure diffusion process of the submerged zone.

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Generally, concrete has low permeability and thus, the concrete cover governs the penetration of chloride ions (Kato and Uomoto 2005). Unfortunately, actual concrete structures such as bridges and highway pavements, which carry in millions of cycles of loads during their service lives, develop internal cracks, inside the concrete cover. These cracks widen or become connected as the number of the cyclic load increases (Gontar *et al.* 2000). As a result, in the marine environment, the chloride penetration of actual concrete structures is accelerated due to the simultaneous action of environmental and service structural loads, particularly, in the case where there is both tidal aggression and cyclic loads. However, the chloride penetration characteristics as well as a model to predict chloride penetration coupled with tidal environment and cyclic loads are still not well established. In this paper, an experimental program was conducted to investigate the effects of cyclic flexural loads on chloride diffusion characteristics in plain concretes. Following these experimental results, a model using FDM, which describes the chloride penetration into plain concretes incorporating a tidal environment and different cyclic flexural load levels, is proposed. The model is verified with a new experiment, in which plain concretes were exposed to simultaneous tidal environment and structural loads.

2. Background of chloride penetration into actual concrete structures

2.1. Chloride binding in concrete structures

Chloride in concrete structures is commonly classified into free and bound chloride. The free chloride is dissolved in the concrete pore solution and moves according to the concentration gradients, and the remainder is bound chloride which includes physically bound chloride and chemically bound chloride.

Chemically bound chloride has reacted chemically with hydration compounds of the cement, particularly with C_3A to form calcium chloroaluminate (Friedel's salt), as a result, chemically bound chloride is present in the solid structure of concrete. The physically bound chloride is attracted to pore surfaces by weak Van der Waals forces. It is generally believed that the free chloride content is proportional to the total chloride content, and that the chemically bound chloride amount is small compared to that of the physically bound chloride. Further, the chloride binding capacity of cements is different for different cement types due to the differences in kinds and contents of compounds present in the cements. Major hydrates of cement are C-S-H, AFm, Aft, and $Ca(OH)_2$. Of these, C-S-H and AFm are responsible for binding chloride ions (Hirao *et al.* 2005). Moreover, with a given amount of hydrate phases, the amount of chloride ions bound into C-S-H is smaller than the amount of chloride ions bound into the alumina phase. There is a relation between chloride binding capacity and concentrations of other ions present on the pore solution, this relation shows that the chloride binding capacity decreases with increasing hydroxide and sulfate concentration in the pore solution. One reason for this phenomenon is that, when the concentration of hydroxide ions increases, there is competition between Cl^- and OH^- to occupy the adsorption sites on the surface of C-S-H (Johannesson *et al.* 2007).

The chloride binding isotherm is a complicated phenomenon, depending on the Cl^- concentration in the pore solution, the presence and concentrations of other ions as well as the composition of the cements. The chloride binding isotherm has been proposed as a linear or nonlinear isotherm (Ishida *et al.* 2009) as shown in Fig. 1.

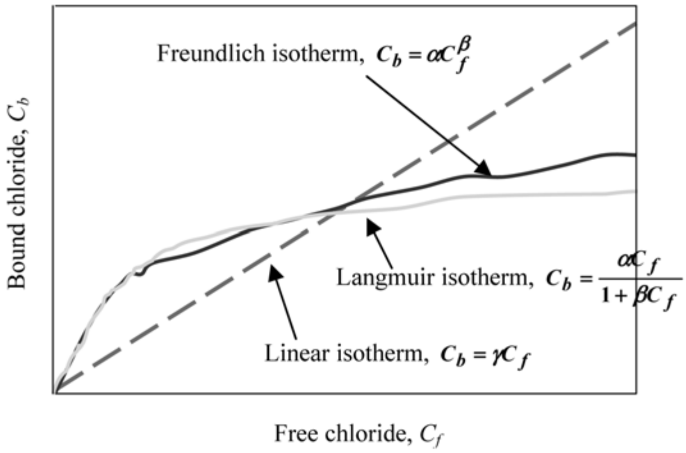

Fig. 1 Proposed chloride binding isotherms

Assuming the Freundlich isotherm simplifies the presence of chemically bound chloride in the product form of the reaction between C_3A and chloride, and the total chloride concentration becomes described as:

$$C_t = C_f + \alpha C_f^{\beta} \tag{1}$$

Where, C_b is the bound chloride, C_f and C_t is the free and total chloride, α and β are constants which depend on the content of C_3A in the cement (Han 2007):

$$\alpha = 0.56 + 0.025C_3A \tag{2}$$

$$\beta = \frac{1}{0.076C_3A + 1.91} \tag{3}$$

Additionally, the relationship between free and total chlorides measured in field-exposed concrete has been shown to be more linear than the corresponding relationship measured in laboratory equilibrium experiments (Mohamed and Hamada 2003). In this case, β =1, and the equation describing the total chloride content becomes:

$$C_t = C_f + \alpha C_f = (1+\alpha)C_f \tag{4}$$

with $\phi = (1+\alpha)$, as the chloride binding capacity of a cement.

$$C_t = \phi C_f \tag{5}$$

2.2. Chloride penetration in drying-wetting environment

Considering chloride transport in concrete one-dimensionally, the mass balance can be expressed as (Oh and Yang 2007):

$$\frac{\partial C_t}{\partial t} + \frac{\partial J_c}{\partial x} = 0 \tag{6}$$

Where, C_t is the total chloride content by the mass of cement content (%), and J_c is the chloride flux (m/s). In drying-wetting conditions, with the assumption of a linear chloride binding isotherm (β =1), the chloride flux that expresses the chloride diffusion due to gradient concentration, J_{c1}, and chloride convection by moisture transport, J_{c2}, can be written as:

$$J_c = J_{c1} + J_{c2} \tag{7}$$

$$J_{c1} = -D_a \nabla C_t \tag{8}$$

$$J_{c2} = C_f J_w = \frac{C_t}{\phi} J_w; \ \frac{\partial J_{c2}}{\partial x} = C_f \frac{\partial J_w}{\partial x} + \frac{\partial C_f}{\partial x} J_w \tag{9}$$

$$-\nabla J_w = \frac{\partial w}{\partial t} \tag{10}$$

with D_a the apparent chloride diffusion coefficient, w the free water content, and J_w the moisture flux (m/s).

From Eqs. (7), (8), (9) and (10), Eq. (6) is reformulated to:

$$\frac{\partial C_t}{\partial t} = D_a \frac{\partial^2 C_t}{\partial x^2} + \frac{C_t}{\phi}\frac{\partial w}{\partial t} + \frac{1}{\phi}\frac{\partial C_t}{\partial x} D_h \frac{\partial w}{\partial x} \tag{11}$$

The correlation between the relative humidity and the free water content is assumed by desorption isotherms as (Saetta *et al.* 1993):

$$w = w_{sat} h \tag{12}$$

Where, h is the relative humidity (%), and w_{sat} represents liquid content (%).

Substituting Eq. (12) into Eq. (11) gives:

$$\frac{\partial C_t}{\partial t} = D_a \frac{\partial^2 C_t}{\partial x^2} + \frac{C_t}{\phi} w_{sat} \frac{\partial h}{\partial t} + \frac{1}{\phi}\frac{\partial C_t}{\partial x} \cdot D_h \cdot w_{sat} \frac{\partial h}{\partial x} \tag{13}$$

Under thermally constant conditions, the humidity, h, and the humidity diffusion coefficient, D_h, are related and expressed by the following equation:

$$\frac{\partial h}{\partial t} = D_h \frac{\partial^2 h}{\partial x^2} \tag{14}$$

Consequently, the governing equation describing the chloride penetration in the drying-wetting condition can be rewritten as:

$$\frac{\partial C_t}{\partial t} = D_a \frac{\partial^2 C_t}{\partial x^2} + \frac{C_t}{\phi} w_{sat} D_h \frac{\partial^2 h}{\partial x^2} + \frac{1}{\phi}\frac{\partial C_t}{\partial x} w_{sat} D_h \frac{\partial h}{\partial x} \tag{15}$$

2.3. Diffusivity of concrete

Many models have been proposed to estimate the chloride diffusion coefficient compatible with parameters such as the material characteristics of concrete, environmental conditions and service load. Yoon (2009) proposed that the chloride diffusion coefficient is a function of tortuosity and hindrance effect. Also, Saetta *et al.* (1993) confirmed that the chloride diffusion coefficient, D_i, can be expressed as a function of temperature, relative humidity and hydration degree through a reference value of the intrinsic diffusion coefficient, $D_{i,ref}$, which was calculated under standard conditions: temperature 23°C, relative humidity (h=100%) and cement hydration degree after 28 days of maturation under standard conditions.

$$D_i = D_{i,ref} \cdot f_1(T) \cdot f_2(t) \cdot f_3(h) \tag{16}$$

Where, $f_1(T)$ is a function that considers the dependence of D_i on temperature T.

$$f_1(T) = \exp\left[\frac{U}{R}\left(\frac{1}{T_o} - \frac{1}{T}\right)\right] \tag{17}$$

with T and T_o expressed in deg K (T_o=296K), R is the gas constant [kJ/(mol.K)], and U the activation energy of the diffusion process (KJ/mol);
$f_2(t)$ is a function of the effect of the hydration time on D_i.

$$f_2(t) = \left(\frac{28}{t}\right)^m \tag{18}$$

with t a time (days), m a constant that depends on the properties of the concrete mixtures such as m=0.45 for w/c=0.4 and m=0.35 for w/c=0.5;
and $f_3(h)$ considers the effect of the relative humidity on D_i.

$$f_3(h) = \left[1 + \frac{(1-h)^4}{(1-h_c)^4}\right]^{-1} \tag{19}$$

where h is the relative humidity in the concrete, h_c (h_c=0.75 at 25°C) is the humidity at which the coefficient D_i drops to halfway between the maximum and minimum values.

Practically, concrete structures such as elements, bridges, and highway pavements are subjected to a variety of types of loading, but especially cyclic loads, during their service life. Concrete fatigue arises from the numbers of cyclic loads; suggesting that this accelerates the diffusion process of chloride ions in concrete structures.

With actual concrete structures, consequently, the equation of chloride diffusion coefficient should be rewritten to account for the effects of loadings as:

$$D_i = D_{i,ref} \cdot f_1(T) \cdot f_2(t) \cdot f_3(h) \cdot f_4(SR) \tag{20}$$

Where, $f_4(SR)$ is a function of the dependence of D_i on the load levels, SR.

The effect function, $f_4(SR)$, generally has been formulated based on the load level and the kinds of loads.

To account for the effect of chloride binding phenomena which retard the penetration of the free chloride content, the apparent diffusion coefficient of concrete under coupling load and climate will become:

$$D_a = D_i/\phi \tag{21}$$

$$D_a = D_{i,ref} \cdot f_1(T) \cdot f_2(t) \cdot f_3(h) \cdot f_4(SR)/\phi \tag{22}$$

3. Experimental chloride diffusion in concrete under flexural cyclic loads

Experiments with chloride diffusion under flexural cyclic loads were performed on concrete beams made with Ordinary Portland cement (Type I) and water to cement ratios (w/c) as shown in Table 1.

Concrete beams 500 mm long, 100 mm thick, and 100 mm wide were cast using crushed limestone aggregate with a maximum size of 20 mm and Type I cement with w/c of 0.4, 0.5, and

Table 1 Mix proportions used in the research

Mix No.	w/c ratio	Ordinary Portland Cement kg/m^3	Water kg/m^3	Coarse aggregate kg/m^3	Sand kg/m^3
M1	0.40	512	205	992	636
M2	0.50	410	205	992	738
M3	0.60	342	205	992	806

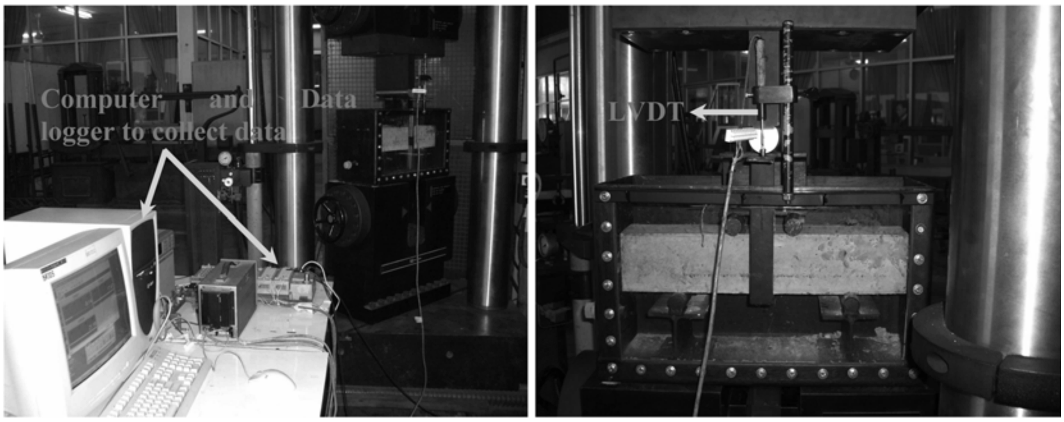

Fig. 2 Set up and equipments used for flexural cyclic loading test

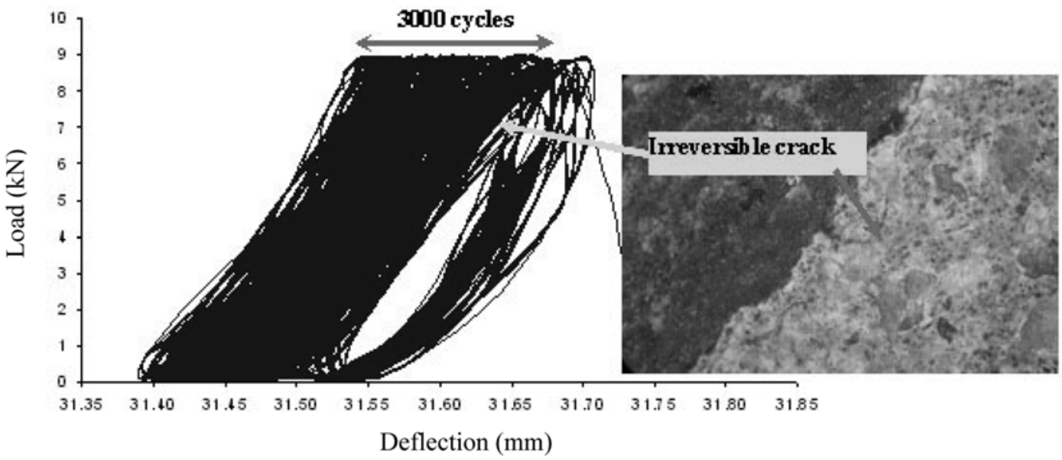

Fig. 3 Typical destructive flexural fatigue results for a load control test, specimen M3

Table 2 Values of N_1 and N_2 for different concrete mixtures

Mix No.	SR*	Fracture load (kN)	Applied load (kN)	Number cycle N_1	Number cycle N_2
M1	0.7	16	11.2	3500	6200
M2	0.7	15	10.5	3200	5000
M3	0.7	13	9.1	3000	4000

*Ratio of applied load to fracture load
N_1 is the number of cycle at which the curve of the load-deflection changes
N_2 is the number of cycle at which concrete beams fracture

0.6. After casting and 24 hours kept in controlled room, concrete beams were demolded and cured in water for 60 days. The flexural strength test of the concrete beams followed ASTM C78. An Instron machine 1200 kN was used to apply static flexural cyclic loads to the beams (Fig. 2). Deflection values of beams were measured by LVDT connected to a Data logger device and a computer. A load control test with low frequency, 0.01 Hz, was used in this research. A ratio of applied load to ultimate load (SR) of 0.7 was adopted as the standard SR to determine the number of cycles N_1 and N_2, at which irreversible cracks and fractures occurred (Fig. 3). The values of N_1 and N_2 which correspond to concrete mixtures are shown in Table 2. All beams were subjected to the flexural cyclic loads using the number of cycles, N_1, with SR 0.5, 0.6, 0.7 and 0.8. Then cored specimens from both the middle bottom and middle top positions were used to identify the effect of the flexural cyclic loads on the diffusion characteristic of plain concrete in the tension as well as in the compression zone. The chloride diffusion coefficients of the cored specimens were determined by a short-term diffusion test which was a modified test combining ASTM C12O2 and NTBUILD 492 (NORDTEST 1999). This test uses cylindrical specimens with a diameter of 100 mm, a thickness of 50 mm and a minimum length of 100 mm sliced from cast cylinders or drilled cores. The specimens were placed in two chambers, a chamber with 3% sodium chloride and a chamber with 0.3N sodium hydroxide, and an external electrical potential of 30V was applied axially across the specimen to force the chloride ions outside to migrate into the specimen. After a certain test duration, the specimen was split axially and a silver nitrate solution sprayed onto one of the freshly split sections. The chloride penetration depth could then be measured from the visible white silver chloride precipitation; next the chloride migration coefficient was calculated from this penetration depth together with equation described in NTBUILD 492. This test procedure was applied to specimens of all the concrete mixtures M1, M2, and M3.

Undergoing flexural cyclic load, the stress of the beam is divided into a tension zone and a compression zone. The effects of the flexural cyclic load level on the diffusion coefficients of plain concrete in the tension and the compression zone are shown in Fig. 4. The chloride diffusion coefficients in the tension zone increased, whereas, it showed a decreasing trend in the compression zone (Fig. 4). Particularly, the effect of the flexural cyclic load on the chloride diffusion coefficient could be expressed as an exponential function in the tension zone and a polynomial function compatible to the compression zone as below:

Tension zone,

$$f_4(SR) = 0.0985e^{7.71SR} \tag{23}$$

Compression zone,

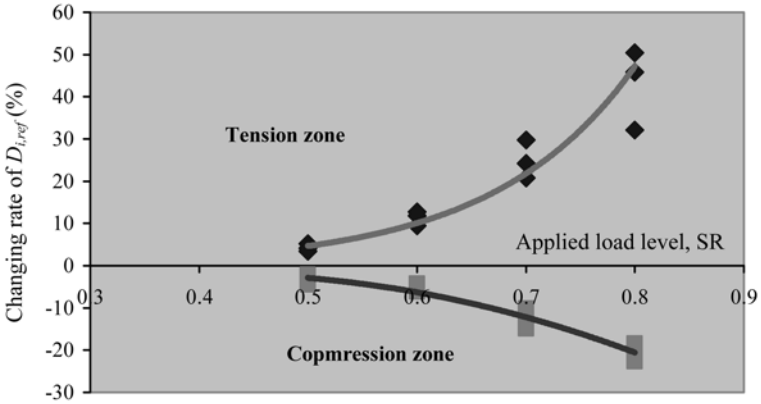

Fig. 4 The effect of applied load levels on chloride diffusion coefficient of concrete both in tension and compression

$$f_4(SR) = -123.33SR^2 + 101.47SR - 22.78 \tag{24}$$

where, $f_4(SR)$ is the function accounting for effects of flexural cyclic load on the diffusion coefficient of the plain concrete, SR is the flexural load level.

It may be supposed that compression stress, which generated in the compression face of the concrete beam undergoing flexural cyclic loading, makes the concrete microstructure become denser, and accordingly reduce porous connectivity, including less diffusion of chloride. The opposite when microcracks are widened to become even irreversible microcracks at high flexural cyclic load levels that generate tensile stresses beyond the elastic stress of the concrete, the porous volume as well as the connectivity of concrete increases so that the chloride diffusion coefficient increases.

The exponential function is clearly composed of three distinctive portions: The first portion, which has the flattest slope, at SR 0.5 to 0.6, the second portion is from SR of 0.6 to 0.7 generating a steeper slope, and the last portion at SR 0.7 to 0.8 has the steepest slope. This evidence indicated that the flexural cyclic load had a strong effect on the chloride diffusion coefficient of plain concrete at SR from 0.6 to 0.8 (especially at SR 0.8). Also, it was found that the chloride diffusion coefficient of plain concrete subjected to a flexural cyclic load of SR 0.7 was nearly 30% larger than that of plain concrete without load. And, SR 0.8 was impractical in designing experiments, because at this applied load level there was a sudden fatigue fracture.

4. Numerical simulation to predict chloride penetration into plain concretes under coupling flexural cyclic load and tidal conditions

After establishing the function $f_4(SR)$, a numerical simulation was analyzed based on one-dimensional chloride penetration. The apparent chloride diffusion coefficient, D_a, was assumed constant in the time domain. The humidity, h, and surface chloride content, C_s, incorporated in the numerical solution changed in the time domain to simulate tides. Thereafter, verification of the model was conducted through experiments to determine the reliability of the numerical analysis.

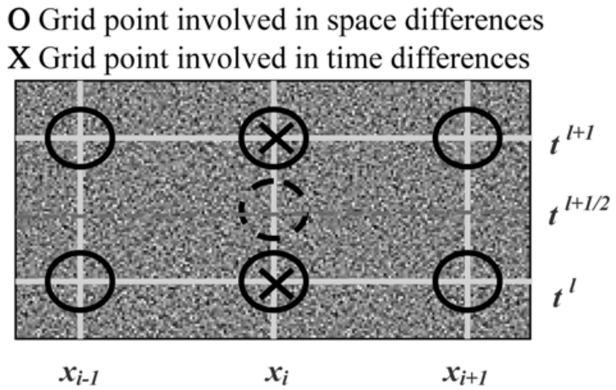

Fig. 5 Schematic discretization in the space and time domain

4.1. Discretization

The finite difference method and Crank-Nicholson algorithm (Chapra and Canale 1998) were used to discretize and solve the one-dimensional problem. A one-dimensional grid in the space domain was used to discretize the section of plain concrete. Also, a one-dimensional grid was applied to discretize the time domain (Fig. 5). The second-order parts of the second-order parabolic partial differential Eq. (15) were approximated with a standard method that has second-order accuracy as shown in Eqs. (25) and (26). And, the first-order part of Eq. (15) was approximated with the first-order accuracy as shown in Eq. (27). This numerical solution is assumed to have good convergence as well as with the chosen

$$\lambda = \frac{D_a \cdot \Delta t}{\Delta x^2} < \frac{1}{2}$$

$$D_a \frac{\partial^2 C}{\partial x^2} = D_a \frac{1}{2}\left[\frac{C_{i+1}^{l+1} - 2C_i^{l+1} + C_{i-1}^{l+1}}{(\Delta x)^2} + \frac{C_{i+1}^{l} - 2C_i^{l} + C_{i-1}^{l}}{(\Delta x)^2}\right] \tag{25}$$

$$D_h \frac{\partial^2 h}{\partial x^2} = D_h \frac{1}{2}\left[\frac{h_{i+1}^{l+1} - 2h_i^{l+1} + h_{i-1}^{l+1}}{(\Delta x)^2} + \frac{h_{i+1}^{l} - 2h_i^{l} + h_{i-1}^{l}}{(\Delta x)^2}\right] \tag{26}$$

$$\frac{\partial C}{\partial t} = \frac{C_i^{l+1} - C_i^l}{\Delta t} \tag{27}$$

$$\frac{\partial C}{\partial x} = \frac{C_{i+1}^{l} - C_i^l}{\Delta x} \tag{28}$$

$$\frac{\partial h}{\partial x} = \frac{h_{i+1}^{l} - h_i^l}{\Delta x} \tag{29}$$

Applying the Crank-Nicholson algorithm, the discretization of Eq. (15) became:

$$\frac{C_i^{l+1}-C_i^l}{\Delta t} = D_a\frac{1}{2}\left[\frac{C_{i+1}^{l+1}-2C_i^{l+1}+C_{i-1}^{l+1}}{(\Delta x)^2}+\frac{C_{i+1}^l-2C_i^l+C_{i-1}^l}{(\Delta x)^2}\right]$$

$$+\frac{w_{sat}}{\phi}C_i^l D_h\frac{1}{2}\left[\frac{h_{i+1}^{l+1}-2h_i^{l+1}+h_{i-1}^{l+1}}{(\Delta x)^2}+\frac{h_{i+1}^l-2h_i^l+h_{i-1}^l}{(\Delta x)^2}\right]+\frac{w_{sat}}{\phi}D_h\frac{C_{i+1}^l-C_i^l}{\Delta x}\frac{h_{i+1}^l-h_i^l}{\Delta x} \quad (30)$$

4.2. Numerical analysis

Numerical analysis was applied to predict chloride penetration into plain concretes with w/c 0.4 and 0.5, and subjected to both flexural cyclic loads and tidal conditions. During the numerical analysis, the values of D_a and the temperature were assumed constant, however, humidity h and the surface chloride content C_s changed in step rise in the time domain. Using the Crank-Nicholson algorithm, the results of the numerical analysis has good convergence with time steps of 8×10^6s, and very good with time steps of 4×10^6s (Tralla and Silfwerbrand 2002), and 4.32×10^4s (12 hours) (Tran *et al.* 2006). Moreover, taking account for tidal conditions, the time step of the time domain was chosen as 12 hours to include 12 hour wet and 12 hour dry which is representative of the daily tidal cycle periods. Further, space steps of Δx=5 mm were chosen to ensure good convergence of the numerical solution. The values of input parameters used in the numerical analysis are shown in

Table 3 Input parameters used in the numerical analysis of chloride penetration into plain concrete subjected to coupling flexural cyclic loads and tidal cycles

Input parameters	Concrete mixture	
	w/c=0.4	w/c=0.5
Cement type	Ordinary Portland cement	
Concrete age, days	60	
Humidity diffusion		
*Liquid content, w_{sat} (%)	11	13
*Humidity diffusion coefficient, D_h (m²/s)	1.0×10^{-12}	5.0×10^{-12}
Initial humidity, h (%)	100	
Boundary	h_{max}=100, h_{min}=0	
Conditions of time step	Time step, Δt=12 hr	
Chloride diffusion		
**Binding capacity factor, ϕ	1.1597	
Diffusion coefficient, $D_{i,ref}$ (m²/s)	6.25×10^{-12}	13.58×10^{-12}
Initial chloride content (wt % of cement)	$C_{i,0}$ = 0	
Boundary	C_s^{max} = 2.6; C_s^{min} = 0	C_s^{max} = 3.1; C_s^{min} = 0
Conditions of time step	Time step, Δt=12 hr	Time step, Δt=12 hr
Flexural cyclic load		
Cyclic flexural loading level, SR	0; 0.5; 0.6; 0.7 and 0.8	
Temperature, t (°C)	25±2	

*Data from Saetta *et al.* 1993
**Data from Mohamed and Hamada *et al*. 2003

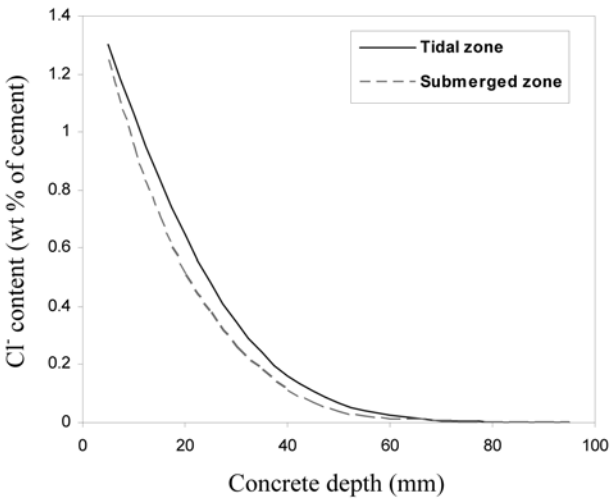

Fig. 6 Prediction of chloride profiles of concretes in tidal zone and submerged zone for 5 year exposure, concrete with w/c=0.5

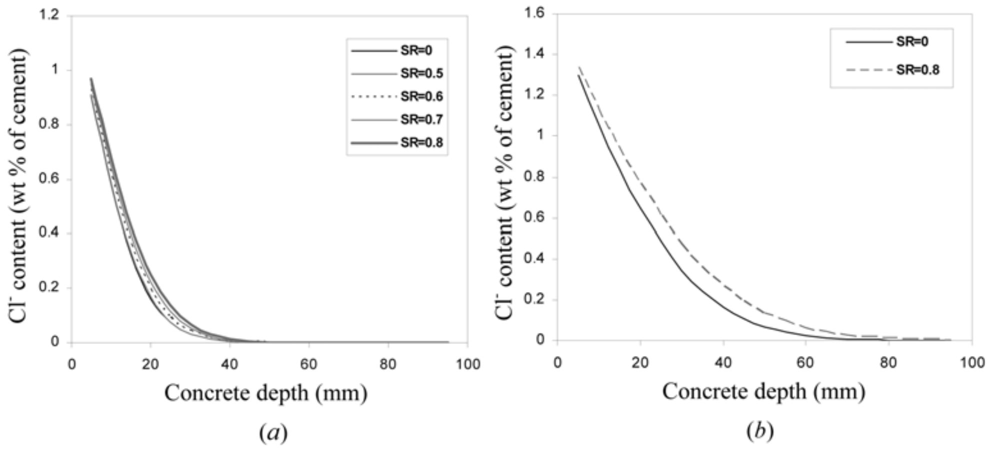

Fig. 7 Prediction of chloride profiles of concretes subjected to coupling cyclic loads and 5 year exposure to tidal environments *(a)* Concrete with w/c=0.4 ; *(b)* Concrete with w/c=0.5

Table 3.

Generally, in tidal environments, the surface chloride content, C_s, is considered as 13 kg/m^3 concrete, equal to 2.6% and 3.1% of the cement content of the concrete mix at w/c 0.4 and 0.5, respectively. Many investigations confirm that chloride can penetrate deeper in the tidal zone due to combination of diffusion and adsorption of chloride ions. Fig. 6 shows the numerical analysis of chloride profiles of concrete with w/c=0.5 exposed to tidal zone and submerged zone for 5 years. It can see that, at a specific concrete depth, the chloride content of concrete exposed to tidal zone is higher than that of concrete exposed to the submerged zone. Figs. 7(a) and (b) show numerical results of the effect of cyclic flexural load levels on the total chloride content of concrete beams exposed to tidal cycles for 5 years with w/c 0.4 and 0.5. At a specific depth, the chloride content of concrete with w/c 0.4 was lower than that of concrete with w/c 0.5. This is due to the different chloride diffusion coefficients, $D_{i,ref}$=6.25×10^{-12} m^2/s (w/c=0.4) and 13.58×10^{-12} m^2/s for w/c=0.5. Showing that for lower w/c, there is less pore volume and connective paths, consequently making the chloride penetration in concrete more difficult.

Modeling results indicated that the cyclic flexural load accelerated the chloride penetration into concrete. At higher cyclic flexural load levels, the chloride penetration was faster, and the acceleration rates of the chloride penetration into concretes became more distinct with cyclic flexural load levels, SR, of 0.6, 0.7, and, especially, 0.8 (Fig. 7). This is consistent with the results of the experiments which described the effect of flexural cyclic loads on the chloride diffusion coefficient of plain concrete in section 3. Regarding the time till corrosion starts, considering a concrete mix with w/c 0.4, it was assumed that the critical chloride content is 0.4% cement by weight to initiate corrosion, this critical chloride content was obtained at specific concrete depths of about 44 mm, 48 mm, and 50 mm for SR 0, 0.7, and 0.8, respectively (Fig 7a). This made clear that at a specific depth, cyclic loads made the time till corrosion is initiated becomes shorter; the durability of actual concrete structures subjected to service load, particularly cyclic flexural loads, was shorter than assumed in conventional design standards where the effect of service loads on the chloride penetration is not considered.

4.3. Verification of numerical analysis

The reliability of the numerical analysis was verified by the results of the experiments and measured results of other researchers. Firstly, the numerical analysis was verified by measured results of Uji *et al.* (1990) to confirm that the numerical analysis of chloride penetration without loading is reliable or not. The final verification consisted of comparisons between numerical results accounting for flexural cyclic loads and experimental results. The first verification is shown in Fig. 8, in which a concrete specimen has D_a=1.89×10^{-12} m^2/s, D_h=1.0×10^{-12} m^2/s and is exposed to tidal environment in 7.6 years. Fig. 8 shows that results of the numerical analysis fitted quite well with that measured by Uji *et al.* (1990). The numerical analysis mentioned above, thus, was reliable and capable of applying to predict chloride penetration into plain concrete subjected to both flexural

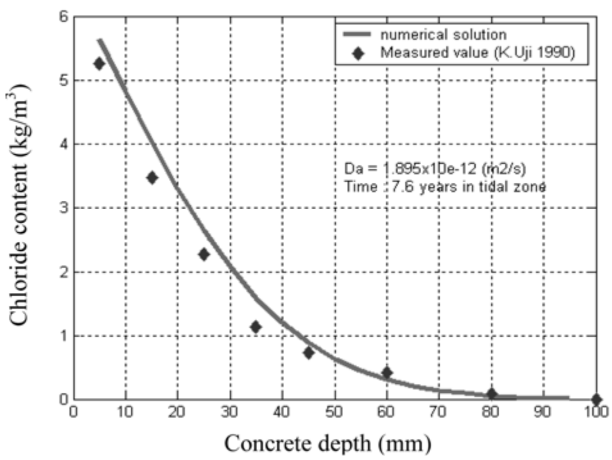

Fig. 8 Comparison between results of numerical solution and measured results of concrete exposed to tidal environment in 7.6 years

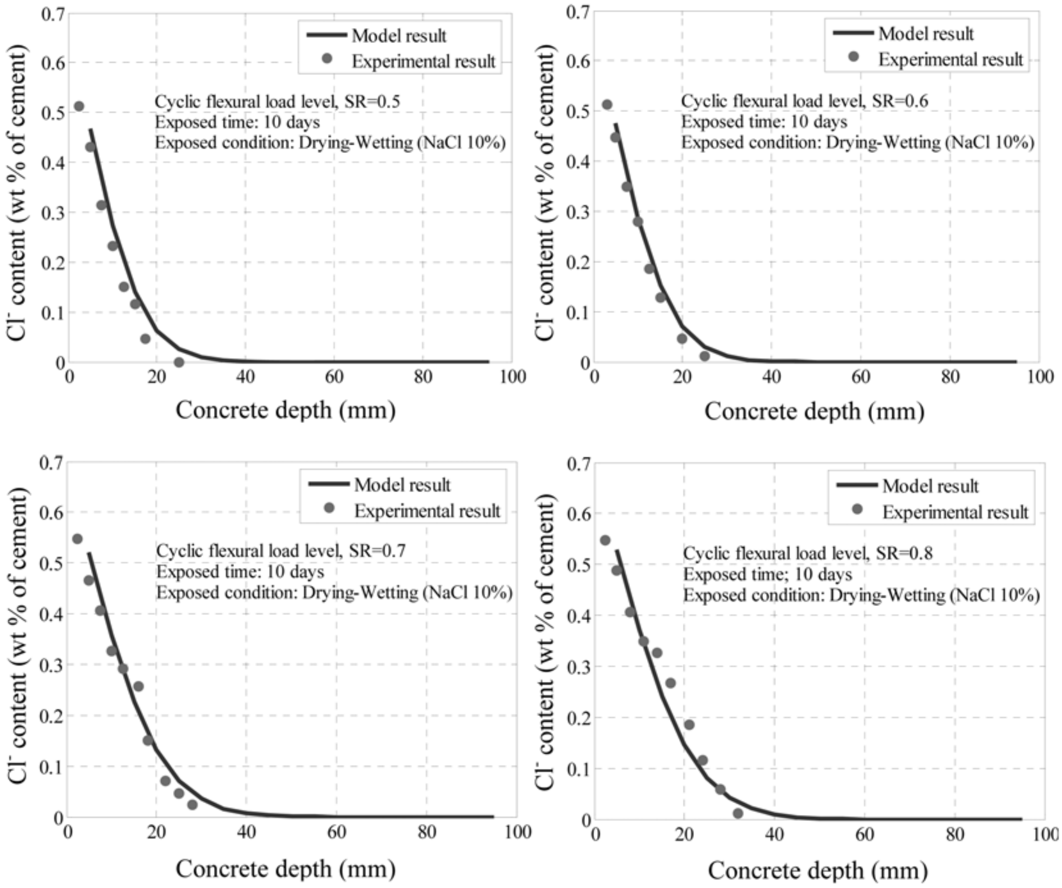

Fig. 9 Verification of chloride penetration into of concrete subjected to both cyclic load and tidal environment

cyclic loads and tidal environments. In experiments of the second verification, 500 mm long, 100 mm thick, and 100 mm wide beams were cast with OPC concrete, w/c = 0.4. After curing for 60 days in water, the beams where all surfaces, except the bottom face, were covered by epoxy subjected to flexural cyclic loads with low frequency, 0.01 Hz, for SR values of 0.5; 0.6; 0.7; and 0.8 up to 3500 cycles. Simultaneously, the simulated tidal conditions, which include 12 hour wetting in NaCl 10% solution and 12 hour drying in atmosphere, were applied to concrete beams so that the concrete beams are subjected to condition resembling the couple of loading and a tidal environment. After finishing 3500 cycles of flexural cyclic load, equivalent to 10 days of cyclic flexural load, cored concrete specimens at the middle bottom of beams were used to analyze total chloride contents at different depths. In the second verification, an experimental equation (Uji *et al.* 1990) was used to determine the surface chloride content, C_s = 5.1% weight of cement. 60 day aged concrete beams have D_a=6.25×10^{-12} m^2/s, D_h=3.0×10^{-12} m^2/s used as input parameters in the numerical analysis accounting for flexural cyclic loads.

The chloride ion profiles of concretes with the couple of different cyclic flexural load levels and simulated tidal environments working for 10 days are shown in Fig. 9. It could be concluded that the experimental results and those of the proposed model fitted well in all cases when applying the cyclic flexural load levels at SR 0.5; 0.6; 0.7 and 0.8. The effect of flexural cyclic loads on the chloride penetration was clear in both the measured and the numerical results. At a given depth, the higher SR value, the higher chloride content was due to widening crack width and appearance of irreversible crack (Fig. 9). These results suggested that the proposed numerical simulation possibly evaluated the chloride penetration into concrete structures subjected to both tidal environment and flexural cyclic load, when the flexural cyclic load levels (SR) changed from 0.5 to 0.8, and the numerical analysis used was FDM combining with Crank-Nicholson algorithm.

5. Conclusions

Experiments of the effect of cyclic flexural loads on the chloride diffusion coefficient in plain concrete were conducted. Then a numerical analysis using the finite difference method was developed to predict chloride penetration into plain concretes subjected to both cyclic flexural loads and tidal environments. The results of the numerical analysis agreed with the experimental results. The present studies suggest the following conclusions:

(1) Cyclic flexural loads have a significant effect on the chloride diffusion in plain concrete at applied load levels, SR, of 0.7 and above. Particularly, the chloride diffusion coefficient of plain concrete in the tension zone applied with SR 0.7 increases nearly 30% over that of plain concrete without load applied.

(2) A new experiment was conducted to describe the simultaneous effect of both cyclic flexural loads and tidal environments. Results from this experiment were used to verify the model.

(3) A model of the chloride penetration into plain concretes coupling cyclic flexural load and tidal environments is proposed. Using a numerical analysis with the finite difference method, results obtained with this model fit well with the experimental data.

(4) The proposed model showed that the cyclic flexural load accelerated chloride penetration into concrete. The higher the cyclic flexural load level, the faster chloride penetration occurred, especially with SR of 0.6, 0.7, and 0.8. This means a reduction in the durability of concrete structures subjected to coupling service loads, particularly cyclic flexural loads, and aggressive environments.

As a result, it will be necessary to modify conventional design procedures of concrete structures in marine environments. And that, it is essential to account for coupling service loads and aggressive environments in modeling to predict the service life of such concrete structures more accurately.

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