

LCC SYNCHRONIZATION BY PROPOSED METHODS OF CHLORIDE DISTRIBUTION IN CONCRETE CONSIDERING STAGNATION OF LIQUID WATER FRONT

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ABSTRACT:

Accuracy of service life prediction largely influence life cycle cost of infrastructure. Most past designers considered cover size and diffusion property as the key parameters to perform durability design where as recent research come up with the conclusion that water penetration front is another key issue regarding cover quality which dramatically change the service life as well as LCC of RC structure.

In this study, Cl^- ion profiles, absorption of water and liquid water front position along with its variation for two types of concrete from bank protection structure located in Okinawa and slag concrete with 70% w/c ratio in the laboratory were examined. Methods of analysis were divided in three groups. Each type of concretes were analyzed with each of the methods and compared with the actual data of chloride profile. It was found that methods of analysis are depended on the type of concrete to be analyzed. A classification of concrete type along with their analysis method is proposed that modifies Fick law of diffusion by the stagnation of liquid water front. It is also shown that adoption of proper method based on type of concrete can synchronizes the LCC supposed by engineers with that of desk researchers.

KEYWORDS: liquid water front, standard deviation, LCC

1. INTRODUCTION

It is very difficult to predict the actual performance scenario of real in-situ structure without the inspection being done. In this context the structure situated in Okinawa was inspected and the cores were brought to the laboratory to get actual profile of chloride ingress. The immersion tests were conducted for 3 months. This work was done by Takahashi Yuya [8] with the conclusion that water penetration depth is the governing position beyond which chloride does not move. This statement was proved for two types of fly ash concrete but OPC did not show similar behavior. For this aspect immersion tests under salt and normal water was continued and a new parameter along with front position is found to be important is the variation of

front position.

One case of slag concrete having 70% w/c ratio was casted and tested for Cl^- ion profile, absorption and liquid water front in the laboratory. Verification for Cl^- ion ingress was done with parameters, mean and deviation of liquid water front, for the case of inspected cores from Okinawa.

A classification is proposed using 3 types of concrete (2 types from Okinawa, 1 from laboratory casted) based on Cl^- ion profile. It is also shown here that adoption of suitable methods of analysis for these types of concrete can reduce the gap between practical engineer and desk researcher in respect of Life Cycle Cost Analysis.

2. BACKGROUND AND PURPOSE

Chloride profile of the cores taken from OKINAWA and the immersion test result of the same clearly stated that liquid water front stops the chloride ion penetration shown in fig. [1a] and [1b] [8].

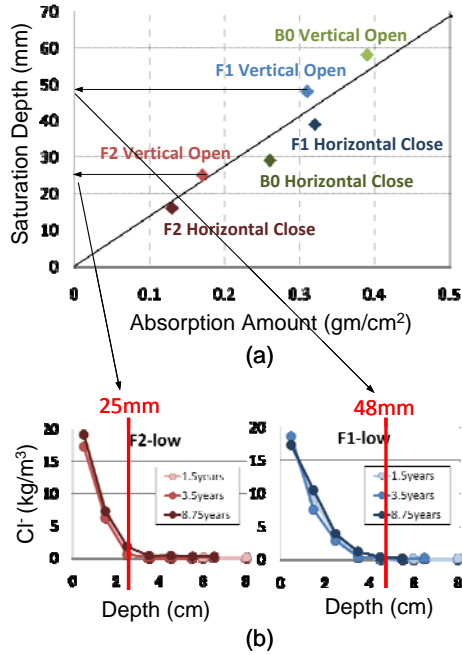


Fig. 1: Comparison between Immersion test and Real Profile

The chloride stopping criterion due to liquid water front (w_f) will change the service life of the structure as explained in fig. [2]. In this figure liquid water front varies from 2 to 10 cm and the concentration of chloride is computed for depth of 6 cm. We can see a large difference in service life that will affect the life cycle cost also for the concrete having different surface quality in respect of liquid water front.

The purpose of this study thus motivated to make a concept that helps to include liquid water front in LCC measurement.

The field engineers suppose that LCC will reduce due to having more durable materials in construction or by improvement of technology where the desk researchers at the design stage

consider only the available options that always overestimates the LCC. To reduce this gap between the image of LCC of two different professionals, a classification of concrete along with the applicable methods of analysis need to be clarified.

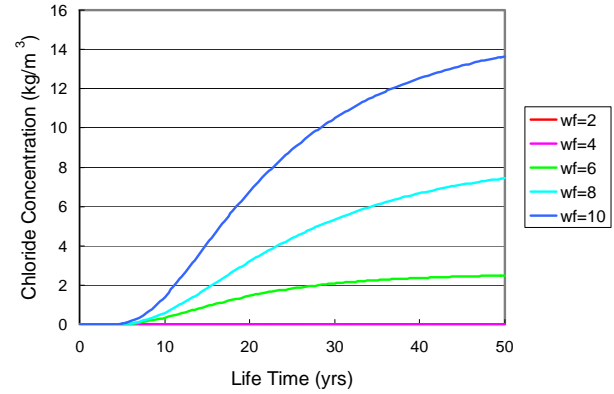


Fig. 2: Effect of liquid water front on service life

3. CONCEPT

The conceptual framework to use liquid water front in LCC is described briefly under sections 3.1, 3.2, 3.3 and 3.4 stepwise.

3.1 Model modification

The flow of chloride is modeled for semi infinite column of porous media [1] [2] by the following diffusion convection equation.

$$\frac{\partial C}{\partial t} = D \frac{\partial^2 C}{\partial x^2} - V \frac{\partial C}{\partial x} \quad [1]$$

where V is average linear rate of flow (cm/s) and depends on porosity. Porosity also is dependent on absorption capacity and liquid water front [6] according to the following equation.

$$\varphi = \frac{1000 \times M}{\omega_f} \quad [2]$$

where M is absorption capacity (gm/mm^2) and ω_f is the liquid water front (cm). Diffusion coefficient D is assumed to follow the following rule to make compatible with the stopping criterion

of chloride ion up to the liquid water front. In fig. [3], MFD, MFP and F stand for Modified Fick Deterministic, Modified Fick Probabilistic and Fick respectively.

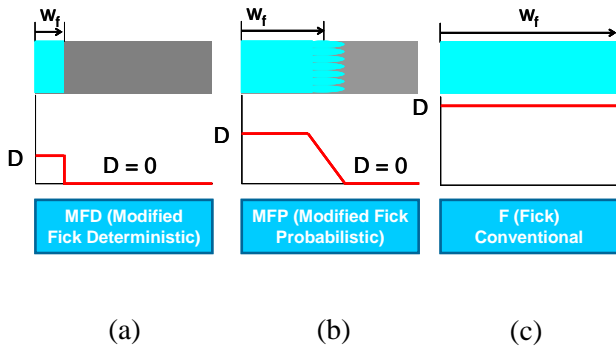


Fig. 3: Coupling between Liquid Water Front and Diffusion coefficient

Fig. [3c] shows the conventional way of thinking that does not depend on liquid water front and the inclusion of the influence of liquid water front on chloride stopping criterion, diffusion coefficient D is set to the value as shown in Fig. [3a] and [3b].

3.2 Parameter quantification

The important parameters governing the flow through concrete can be determined as of the following chart.

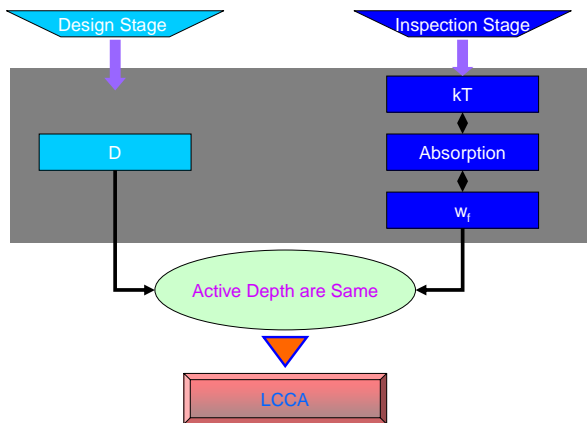


Chart 1: Conceptual Flowchart

Parameter D can be set based on w/c ratio and material used in concrete. Usually the health of structure be judged during the inspection by torrent permeability kT that can be correlated with absorption of water as well as liquid water front.

Finally these parameters can be put as input in LCC computation.

3.3 Reliability based failure

The structural performance function of state 'z' for corrosion initiation of reinforcing steel and crack width are shown below.

$$z = C_{lim} - C_{(x,t)} \quad [3]$$

$$z = \omega_d - \omega_c \quad [4]$$

Equations [3] and [4] can be generalized as load-capacity model shown in Eq. [5].

$$Performance = Strength - Load = A - B \quad [5]$$

where C_{lim} and w_d are the threshold chloride concentration and maximum allowable crack width, $C_{(x,t)}$ is chloride ion concentration (kg/m^3) and is solved using Eq. [1] by finite difference method, w_c is the crack width (mm) as reference [4].

Reliability index can be determined using load-capacity model.

$$\beta_z = \frac{1}{V_z} = \frac{\mu_z}{\sigma_z} = \frac{\frac{\mu_{\ln A} - \mu_{\ln B}}{2}}{\sqrt{\frac{\sigma_{\ln A}^2 + \sigma_{\ln B}^2}{2}}} \quad [6]$$

V_z is the coefficient of variation of performance function z . All random variables are taken as log-normal distribution. Thus $\mu_{\ln A}$, $\mu_{\ln B}$, $\sigma_{\ln A}$ and $\sigma_{\ln B}$ are the mean of strength, load and standard deviation of strength, load respectively.

$$P_f = \varphi(-\beta) \quad [7]$$

The deteriorating structure is characterized by probability of failure $P_f(t)$ or damage over the interval $[0, T]$ as shown in Eq. [7] where φ is the standard normal cumulative distribution function.

The reliability or performance index of structure thus comes to as follow.

$$R_t = 1 - P_f(t) \quad [8]$$

The time to initiation of corrosion is referred as t_i and t_{cr} is named as time to reach allowable crack. Thus, the study reports the failure time as the summation of both the times indicated above.

$$t_f = t_i + t_{cr} \quad [9]$$

It is assumed that when $R(t) < 0.8$ structure needs repair.

3.4 Life Cycle Cost estimation

LCC plays key role in maintaining the infrastructure and provides necessary information to the manager or owner. In this study LCC is computed in the following way.

$$LCC = Initial\ Cost + \sum_{t=0}^T \left(\begin{array}{l} Aging\ Cost + Repair\ Cost \\ + Delay\ Cost \end{array} \right) \quad [10]$$

The three terms in the right hand side were accounted as explained by the following sections.

3.4.1 Initial cost

Initial cost is incorporated in the way below.

$$Initial\ Cost = \left\{ \begin{array}{l} (unit\ cost \times area) \\ \times (1 + \alpha T_{Lifetime}) \end{array} \right\} \quad [11]$$

where α is the cost of durability and is determined by the ration of cover to liquid water front, $T_{Lifetime}$ is the design lifetime of the structure.

3.4.2 Aging cost

This is the cost carried by the owner due to regular maintenance operation. Aging cost is assumed to be proportional to the failure probability, as both of them increase with the increase of age of the infrastructure.

$$Aging\ Cost = \left\{ \begin{array}{l} Initial\ Cost \times \\ 0.05 \times P(f)_{t_i} \end{array} \right\} \quad [12]$$

..... $u = 0$

$$Aging\ Cost = \left\{ \begin{array}{l} Initial\ Cost \times \\ 0.05 \times P(f)_{t-t} \end{array} \right\} \quad [13]$$

..... $u = 1$

It is assumed that 5% of initial construction cost will be expended for maintenance. $P(f)_i$ is the probability of failure at yrs. t , number of repair is subscript i , t_i is the i th repair, u is the decision for repair, $u=0$ means no repair and $u=1$ represents do repair.

3.4.3 Repair cost

This cost is provided by the owner due to repair when the performance goes down below the required. In this study the repair is taken to be happened at performance index of structure goes below 80% of initial. Repair cost is modeled in the way below.

$$Repair\ Cost = 0 \quad \text{.....} \quad u = 0 \quad [14]$$

$$Repair\ Cost = \left[\begin{array}{l} Fixed\ Cost + \\ \left(\begin{array}{l} unit\ Cost \times area \\ \times P(f)_{t_{i-1}} \times \Delta x_{t_i} \end{array} \right) \end{array} \right] \times \frac{t_{RSL}}{t_{Repair}} \quad [15]$$

... $u = 1$

where $unit\ cost$ is the cost of repair for unit area, $P(f)_{t_{i-1}}$ is the failure probability just before repair, Δx_{t_i} is the change of state done by repair i at time t , t_{RSL} is the residual service life in years,

t_{Repair} is the life time of repair material.

It is assumed that the performance will be

improved up to Δx_t^i that will meet initial level of performance. Patching of cover is chosen as repair method with fixed and variable cost \$ 1450 and \$/m² 277 respectively [7]. In the later part of this study the normalized cost refers to the ratio of total cost for 50 years of life time to the initial cost of construction.

3.4.4 Delay cost

This is the part of expenditure carried by the road user for extra fuel consumption and delay due to congestion at the time of repair for partial or full closure of traffic way. It is assumed to be proportional of age as traffic volume is increased with the age and capacity of the road if remains constant.

$$Delay\ Cost = 0 \quad \dots \quad u = 0 \quad [16]$$

$$Delay\ Cost = \left[\begin{array}{l} (\% \text{ traffic Delay}) \times \\ (\text{traffic volume})_t \times \\ \text{repair time} \times \\ \text{average delay} \times \\ \text{unit cost} \end{array} \right] \quad \dots \quad u = 1 \quad [17]$$

where % traffic delay is the number of vehicle delayed at the repair time and is assumed here 10%, traffic volume is the function of time, repair time is the time taken by the repair in days, average delay is the % time delay due to repair by car or truck, unit cost is the time value of delay. Delay cost is calculated from literature stated in reference [5].

After computing the total cost for a particular year, it is discounted to the present time, or the time at which the analysis is being performed. This means the calculated cost at a future time is adjusted

for inflation and prevailing interest rates. After summing costs for the current analysis year and discounting those costs to the present, it returns to the performance model to begin a new analysis year.

4. APPLICATION OF VARIATION OF LIQUID WATER FRONT TO SERVICE LIFE PREDICTION (CASE STUDY OF OKINAWA STRUCTURE)

Three types of concrete namely B0, F1 and F2 that stands for Ordinary Portland Cement concrete, fly ash 1 and fly ash 2 concrete, were taken as core that have composition according to table [1].

Table 1- Mix Proportion (Okinawa Structure)

Type	W/(C+F) (%)	Unit Volume(kg/m ³)				
		W	C	F	S	G
B0	56	169	302	0	860	979
F1	56.3	172	245	60	852	955
F2	46.7	175	295	80	688	1052

4.1 Parameters from core data

4.1.1 Profile from inspection data

The cores were collected from Okinawa at the inspection age of 8.75 years. The cores were divided into several parts to utilize in various experiments as shown in fig. [4].

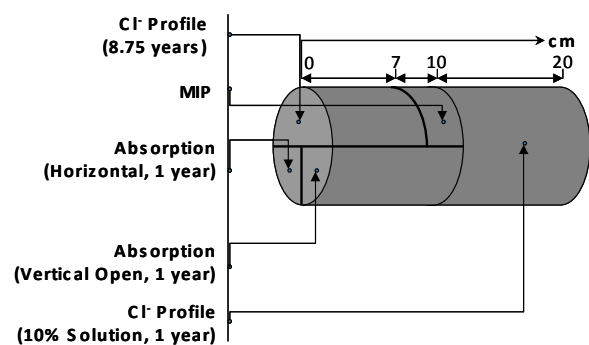


Fig. 4: Division of core specimen

Previous inspection was done by Prof. Yamada (University of the Ryukyus) at 1.5 and 3.5 years of structure's age and the profiles of past data including 8.75 years data at present are plotted in fig. [5], [6], [7].

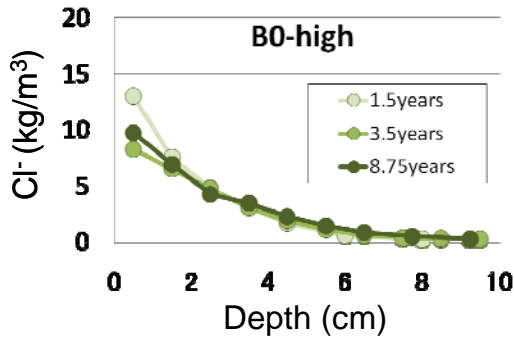


Fig. 5: Chloride Profile for core (B0)

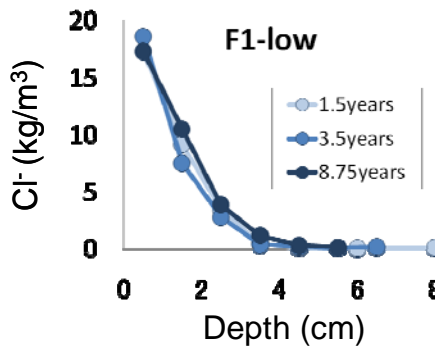


Fig. 6: Chloride Profile for core (F1)

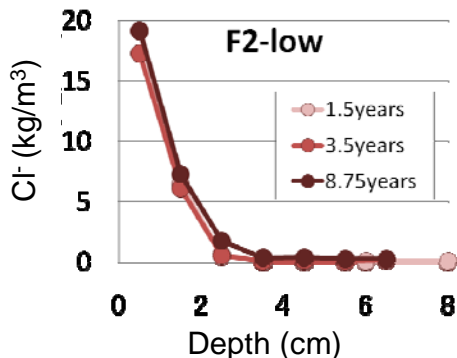


Fig. 7: Chloride Profile for core (F2)

Chloride stopping position changes according to the composition of materials but did not go deeper with the change of age.

4.1.2 Experimental study

To go along with the concept, it is necessary to know diffusion coefficient, absorption and liquid water front. To extract those data, the end part (10 to 20 cm) of the core specimens was coated with primer and epoxy to make it non penetrable. Only the required surfaces are kept open. Specimens are put under 10% NaCl solution. Chloride

concentration profile was measured after 1 year by titration according to JCI SC5 [3] to specify diffusion coefficient. The profile from 1 year of exposure and the real profile show clear stopping position of chloride movement at same distance from surface as it will be described in verification section.

The rest two parts were put in pure water with two different exposure conditions as horizontal and vertical open. The weights were measured continuously to know absorption until 1 year and are shown in fig. [8]. Finally the specimens were broken and water penetration front were measured according to fig. [9].

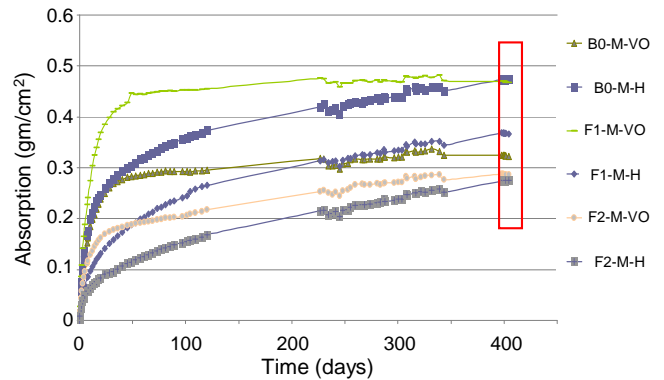


Fig. 8: Water absorption of Okinawa cores



Fig. 9: Measurement of Liquid Water Front

The relation between absorption amount of water and liquid water front is shown in fig. [10].

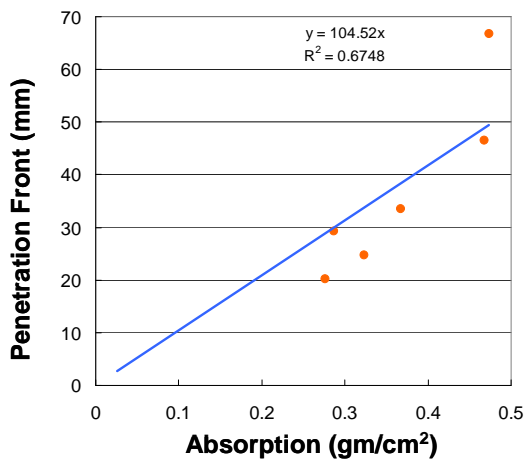


Fig. 10: Correlation between absorption and liquid water front (Okinawa Cores)

Surface chloride C_o , diffusion coefficient D and liquid water front w_f are shown in table [2].

Table 2- Parameters (Okinawa Structure)

Type	C_o (kg/m ³)	D (cm ² /yr)	w_f (mm)
B0	21.74485	2.308418176	47.47577
F1	27.30111	0.12491383	40.04533
F2	30.21536	0.052945642	24.76467

4.2 Verification

Before going to LCCA, it is necessary to compute chloride penetration behavior in perfect. Fick's law is modified taking liquid water front in consideration. Verification is done by matching the 1 year chloride profile for two types of fly ash concrete with the profile derived from modified Fick's law along with liquid water front.

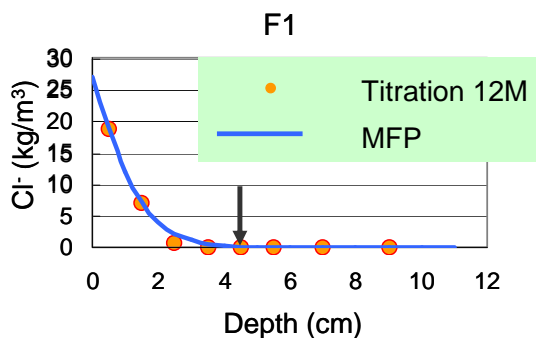


Fig. 11: Verification with MFP (Okinawa F1)

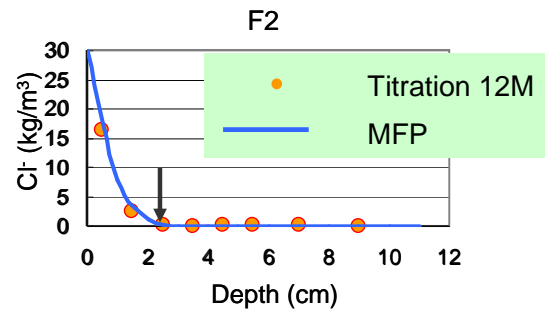


Fig. 12: Verification with MFP (Okinawa F2)

Above fig. [11] and fig. [12] agree well to justify the liquid water front to take into account for chloride profiling. But the exposure conditions explored the risk or variation of liquid water front for different types of concrete as stated in fig. [13].

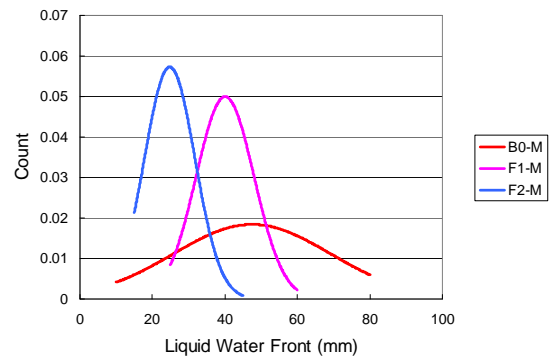


Fig. 13: Variation of Liquid Water Front (Real Case)

The mean value and standard deviation of liquid water front for three different types of concrete are shown in table [3].

Table 3- Variation of Water Penetration Front (Okinawa Structure)

Type	Water Penetration Front, w_f (mm)	
	Mean	Standard Deviation
B0	47.47577	21.67116742
F1	40.04533	7.986315768
F2	24.76467	6.964178162

A comparison graph between standard deviations, assumed as 4.748 and real standard deviation 21.675 from measurement, for B0 concrete is shown in fig. [14].

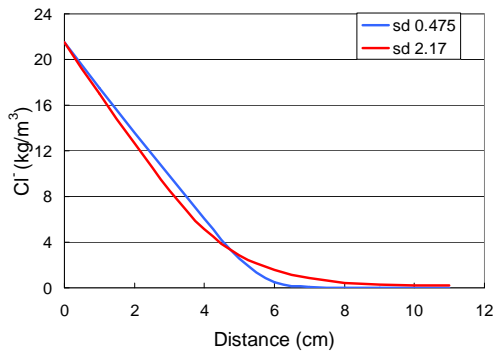


Fig. 14: Effect of Standard deviation on Chloride profile (B0 Concrete)

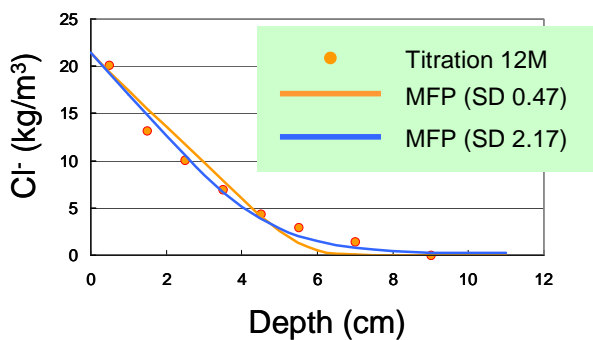


Fig. 15: Verification with MFP (Okinawa B0)

That implies the importance in inclusion of variation of liquid water front in the judgment of RC structural health. Fig. [15] verifies chloride ion profile for B0 concrete taking consideration of liquid water front along with the variation of the front position.

4.3 Experimental study (Laboratory casted slag concrete)

Slag concrete with 50% replacement of cement by slag were cast in the laboratory with w/c ratio of 40%, 55%, 70% respectively. All the specimens were coated by primer and epoxy to make it non penetrable. The required surfaces were kept open. The specimens were put in 10% NaCl solution in three different exposure situations as horizontal, vertical open and vertical coated. Wet-dry cycles were maintained to simulate splash zone. Wet condition was set as submergence in NaCl solution with temperature 20⁰C and 100% RH for 1 day and dry condition was set by keeping specimens inside 40⁰C chamber with 60% RH for 6 days. The weights

for all the specimens were measured twice a week every after each condition. In this paper, concrete having 70% w/c ratio is treated as bad concrete.

After the end of 6 months the specimens were broken and liquid water front were measured. The position of front was measured at a number of points and the risk of variation is shown in fig. [16].

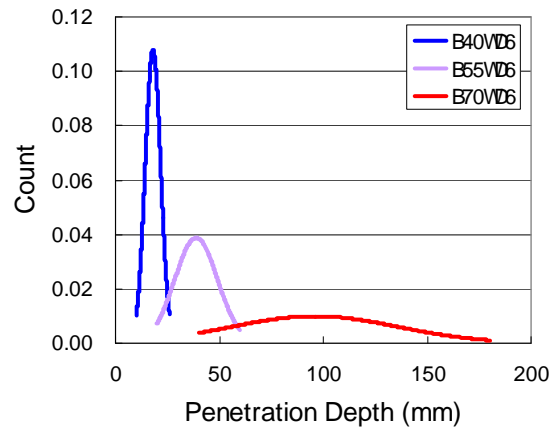


Fig. 16: Variation of Liquid Water Front (Experiment Case)

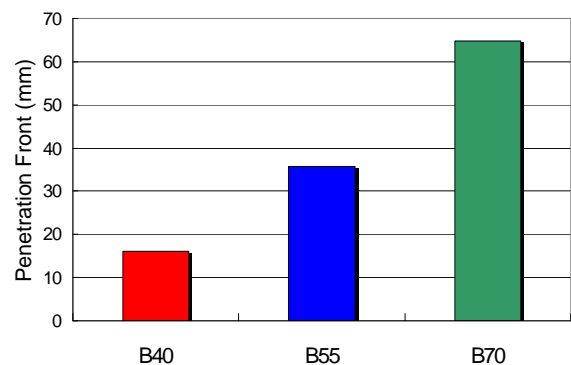


Fig. 17: Mean water front of horizontal series (Slag concrete)

The mean front position of horizontal series is shown in fig. [17], where the depth for 70% w/c ratio will be used in the later section.

5. CLASSIFICATION OF CONCRETE AND ITS APPLICATION TO LCCA

Two types of concrete from Okinawa and one slag concrete casted in laboratory were analyzed in case for classification. Three types of analysis methods are considered namely Modified Fick Deterministic

(MFD), Modified Fick Probabilistic (MFP) and Fick (F). Table [4] shows the difference among all three types of methods. Mean and standard deviation of water front for the analyzed concretes are stated in table [5].

Each type of concrete is analyzed for Cl^- profile by three different methods and compared with the actual profile data as shown in fig. [18]. Root mean square error (RMSE) were determined for all cases and shown in fig. [19].

Table 4- Difference among methods

Methods	Liquid Water Front (cm)	
	Mean	Standard Deviation
MFD	Actual	No
MFP	Actual	Actual
F	Independent of water front	

Suitable method for each type of concrete is selected based on minimum RMSE. For good concrete case although RMSE of MFP is slightly lower than that of MFD, still MFD is chosen best for its ease of application.

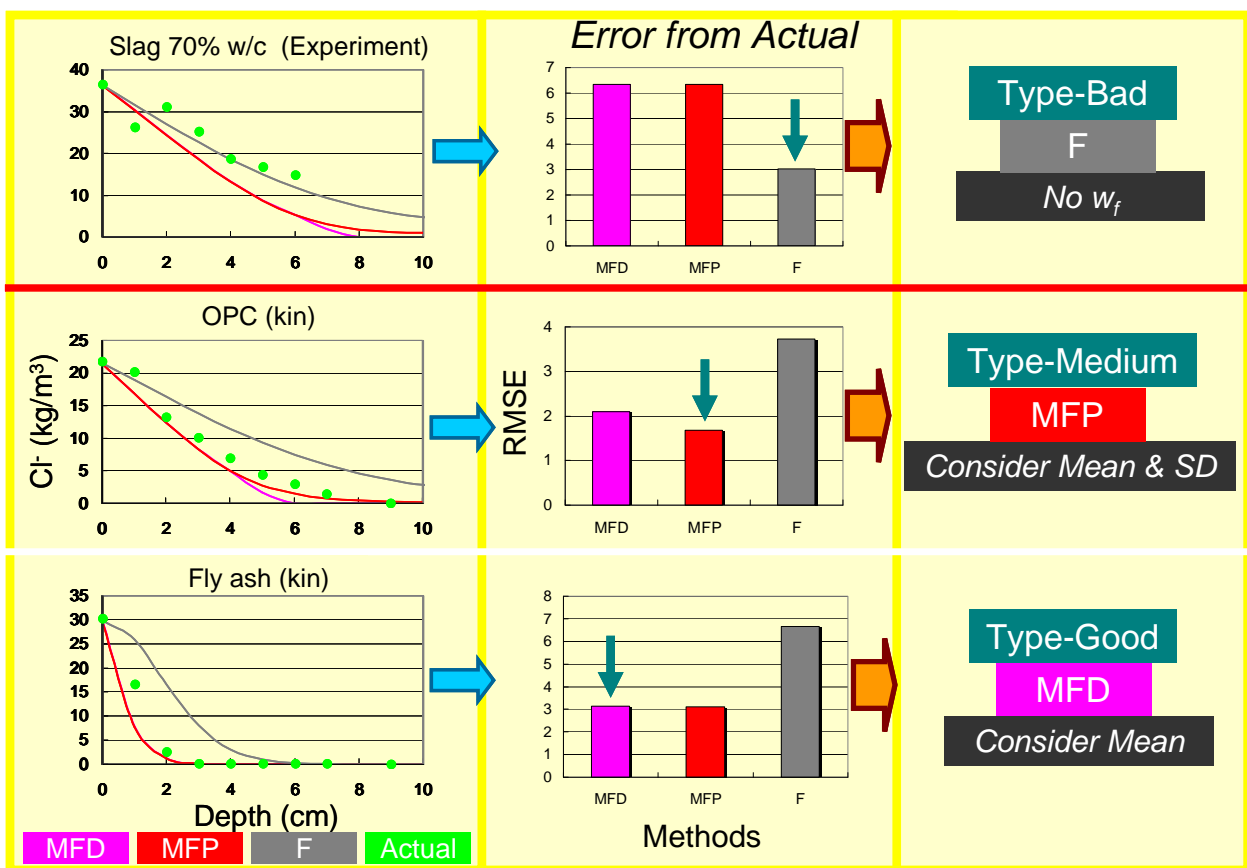


Fig. 18: Cl^- profile by different methods

Fig. 19: RMSE for different methods

Table 5- Water front of analyzed concrete

Type	Liquid Water Front (cm)	
	Mean	Standard Deviation
F2	2.48	0.7
B0	4.75	2.17
B70	6.47	2.42

LCC computed for 50 years of lifetime for each type of concrete using each of three methods is shown in fig. [20]. The base method indicates most close analysis result with actual data. Thus LCC can be compared with the LCC computed using base method.

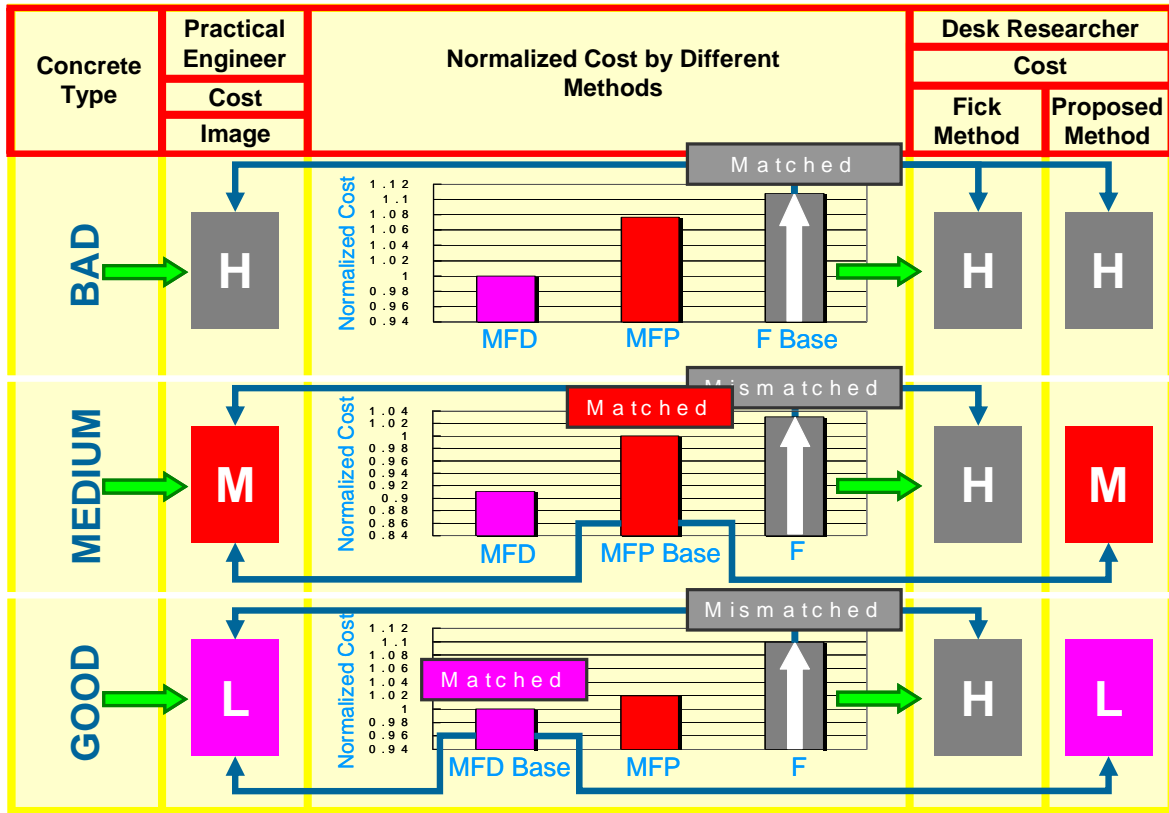


Fig. 20: Gap reduction between two professionals based on correct LCC

where L, M and H represent low, medium and high costs. If the desk researchers consider Fick (F) method for analysis in case of good and medium grades concrete, LCC will be higher than what practical engineer expects. But for the case of bad quality concrete Fick (F) method can be used in LCC computation. That means if desk researchers adopt suitable methods to analyze the different grades of concrete LCC will be same as practical engineer assume. Thus concretes should be analyzed with methods classify in the following chart.

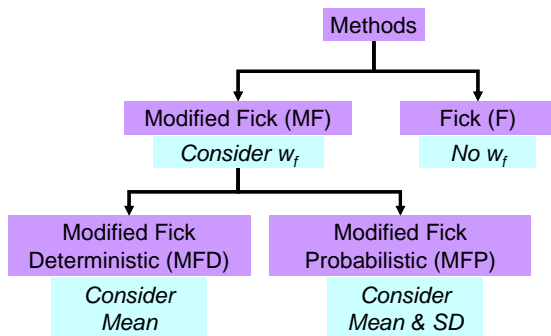


Chart 2: Classification of analysis methods

Thus chloride distribution analysis using suitable methods according to above chart for different grades of concrete and putting the performance as input in LCCA can reduce the gap existed between practical engineer and desk researchers.

6. CONCLUSIONS

A concept is developed to take into account the liquid water front for service life and LCC prediction. By applying this concept to real structure, following remarks can be pointed out.

- ❑ Fick law will overestimate concentration of CI and the LCC for concrete with good surface quality.
- ❑ Liquid water front and its variation play important role in the prediction of CI profile. For the case of very good concrete having smaller pores, the seepage front of water is stopped sharply and CI can not go beyond the water front position. For the case of medium grade of cover quality concrete, water front is

not stopped in sharp but it creates some variation along its mean stopping position due to having small and medium sized pores. Thus Cl^- moves beyond mean position of stopping water front. In the case of bad quality concrete, although saturation drops inside, but water flows deeper with larger pores and creates unsaturated zone through which Cl^- can flow.

- Suitable methods of analysis based on durability classification of concrete can bridge up the gap between practical engineer and desk researchers regarding LCCA.

ACKNOWLEDGEMENT

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