Journal of Thailand Concrete Association
วารสารวิชาการสมาคมคอนกรีตแห่งประเทศไทย

# INSPECTION AND SERVICE LIFE EVALUATION OF A CRACKED RC STRUCTURE DUE TO DRYING SHRINKAGE

**Pakawat Sancharoen[1] Tamyot Somyapakdee[2] Raktipong Sahamitmongkol[3] Somnuk Tangtermsirikul[4]**

[1]Research Faculty, Construction and Maintenance Technology Research Center, School of Civil Engineering and Technology, Sirindhorn International Institute of Technology, Thammasat University

[2]Former graduate Student, Faculty of Engineering, Rajamangala University of Technology Thanyaburi

[3]Former research faculty, CONTEC, SIIT, Thammasat University

[4]Professor, School of Civil Engineering and Technology Sirindhorn International Institute of Technology, Thammasat University

**ARTICLE INFO:**

*Received:February 20, 2014*
*Received Revised Form:May 2, 2014*
*Accepted: May 30, 2014*

**Corresponding Author,*
*Email address: pakawat@siit.tu.ac.th, somnuk@siit.tu.ac.th*

**ABSTRACT :**

Damages on RC structures cause awareness to the public on the safety and serviceability of those structures. Moreover, structure durability also decreases when no attention is given to correct the damages. In this study, damages in terms of cracks were observed on the concrete surface. To ensure the safety of this structure, visual inspection, non-destructive tests (NDTs) were conducted to gather necessary structural information. Inspection results were concluded and used to analyze the cause of damage and performances of the structure. Moreover, progress of damage conditions had been monitored before a suitable maintenance planning program was recommended accordingly.

**KEYWORDS:** Maintenance, Drying shrinkage, Corrosion, Durability, Non-destructive tests

## 1. Introduction

Maintenance planning, normally consisting of many procedures including inspection, deterioration prediction, decision making, and remedial actions, is very important to maintain safety and serviceability of RC structures. Service life of the structures can be extended as well as maintenance cost can be reduced if the maintenance has been properly conducted. Normally, RC structures can deteriorate by many mechanisms such as shrinkage, sulfate attack, chloride attack, carbonation, etc. Maintenance has to be planned based on specific service conditions of each structure. In the past, an example of maintenance planning of structure attacked by chloride was proposed [1].

In this study, an RC structure located in the central region of Thailand is adopted as an example for conducting maintenance planning. The structure has been in service since 1998. Its internal appearance is shown in Figure 1. Its environmental condition is not severe as it is located far from the sea and from the city. However, cracks have been observed widely on the bottom surface of slabs as well as the bottom and side surfaces of beams of this structure. To determine the safety and serviceability of this structure, visual inspection, non-destructive tests (NDTs) and partially destructive tests were conducted. Inspection results were concluded and used to analyze the cause of the damage and future performance of the structure. Moreover, progress of damage conditions had been monitored before a suitable remedial action was recommended. Figure 2 shows the general maintenance procedure [2].

## 2. Inspection Program

As explained, cracks have been observed extensively on the concrete surfaces of the structure. Firstly, inspection programs including visual inspection, non-destructive tests (NDTs) were conducted to determine level of the damage, cause of the damage, and future deterioration rate.

*2.1 Visual Inspection*

Visual inspection was firstly conducted mainly to determine the damage pattern by visualization and simple measurement devices such as crack scale and microscope. Approximately 70-80% of the structure area was inspected. Then, crack width, crack tip, crack location and time of inspection were recorded. Locations of testing are shown in Figure 3.

*2.2 Depth of concrete cover*

Depth of concrete cover is necessary for protecting reinforcing steels from corrosion. In this study, selected slabs and beams were inspected by a commercial rebar detector based on the technique of electromagnetic field induction. Approximately, 10-20% of total structure area was inspected and depth of concrete cover was recorded. Locations of testing are shown in Figure 3.

*2.3 Concrete compressive strength*

In order to inspect the variation of concrete quality, concrete compressive strength was estimated by using a rebound hammer. Although there are other

**Figure 1** Structure appearance.

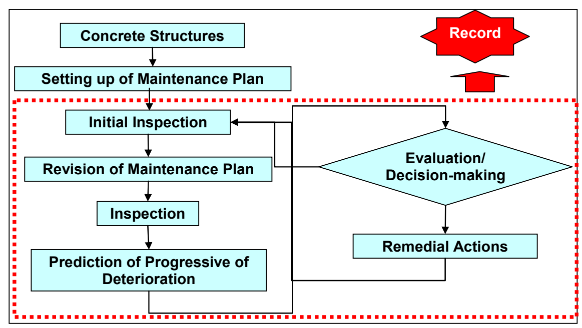


**Figure 2** General maintenance procedure.

NDT methods such as air or water permeability test, rebound hammer is still the most convenient method to be conducted at site. Smooth concrete surface and direction of rebound hammer are carefully controlled during the measurement. For one set of sample, the minimum of 12 points of which spacing is 2.5 cm were measured for the rebound number. Totally 12 sample sets were measured. Locations of testing are shown in Figure 3. Concrete compressive strength can be calculated from the measured rebound number as shown in Equation (1).

$$f_c' = -18 + (1.27 \times RN) \tag{1}$$

Where, $f_c$' is concrete compressive strength (MPa), and RN is rebound number.

*2.4 Half-cell potential*

Corrosion of reinforcing steel was inspected by half-cell potential measurement at the selected locations of the structure. Concrete surface was wet for 2 hours before measuring the half-cell potential to stabilize the measuring value. A $Cu/CuSO_4$ electrode was used as a reference electrode. Locations of testing are shown in Figure 3.

*2.5 Carbonation depth*

Carbonation depth was also tested on the drilled concrete powder based on NDIS3419. Locations of testing are shown in Figure 3.

*2.6 Ultrasonic pulse velocity*

Crack depth was inspected by using ultrasonic pulse velocity. Depth of the crack can be calculated. Locations of testing are shown in Figure 3.

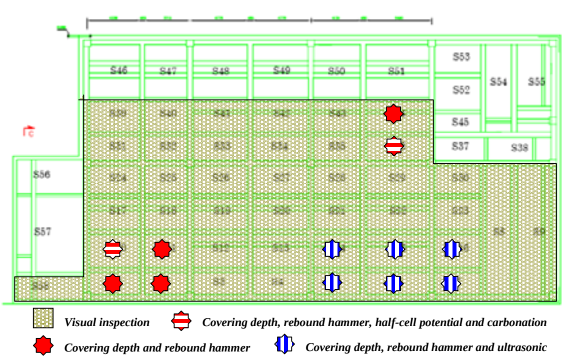


**Figure 3** Locations of inspections.

## 3. Inspection result and discussion

*3.1 Visual inspection and shrinkage analysis*

Figure 4 shows the result of crack map. As shown, most cracks were observed to be perpendicular to the longitudinal axis of the members. Most values of crack width were in the range of 0.1-0.2mm.

According to the observed crack pattern and there is no evidence of over loading, externally restrained shrinkage was expected to be the main reason of the cracking. Based on JSCE Equations for shrinkage estimation [3], shrinkage strain of concrete structure can be estimated. The equation which is suitable for materials and environment in Thailand is being proposed at present.

$$\varepsilon'_{cs}(t,t_0) = \left[1-\exp\left\{-0.108(t-t_0)^{0.56}\right\}\right]\cdot\varepsilon'_{sh} \quad (2)$$

$$\varepsilon'_{sh} = -50+78\left[1-\exp\left(RH/100\right)\right] + 38\log_e W - 5\left[\log_e\left(V/S/10\right)\right]^2 \quad (3)$$

$$t-t_0 = \sum_{i=1}^{n}\Delta t_i \cdot \exp\left[13.65 - \frac{4000}{273+T(\Delta t_i)/T_0}\right] \quad (4)$$

where, $\varepsilon'_{sh}$ is final value of shrinkage strain (x$10^{-5}$) and can be calculated from Equation (3), $\varepsilon'_{cs}$(t, $t_0$) is shrinkage strain of concrete from age of $t_0$ to t (x$10^{-5}$), RH is relative humidity (%) (45%≤RH≤80%), W is unit water content (kg/$m^3$) (130kg/$m^3$≤W≤230kg/$m^3$), V is volume ($mm^3$), S is surface area in contact with outside air ($mm^2$), V/S is volume-surface ratio (mm) (100mm≤V/S≤300mm) and $T_0$ is temperature (°C).

In this study, relative humidity is assumed to be 75%, unit water content is 200kg/$m^3$, average member size is 3160x4630 mm and temperature is 25°C. Cracking strain is the time-dependent strain that causes cracking in concrete and was studied by Tongaroonsri et al. [4]. Figure 5 shows comparison between the calculated shrinkage strain and cracking strain. As shown, cracking was generated 73 days after shrinkage started. Moreover, cumulative crack width for each member, which is 0.71mm at the time of inspection, was calculated by the difference between current shrinkage strain and cracking strain under the assumption that the degree of restraint is 100%. Also strain in concrete is uniform even after cracking. From the actual inspection result as shown in Figure 6, the average of measured cumulative crack width of each member is 0.70mm which is close to the calculated result.

*3.2 Non-destructive testing and load carrying capacity analysis*

Results of NDTs are presented in this section. Figures 7 and 8 show distribution of covering depth and rebound number as well as result of distribution fitting. Average covering depth and calculated compressive strength were 40mm and 39MPa, respectively, which are higher than the design requirements.

Carbonation test and half-cell potential measurement were also conducted. Results show that there was no sign of corrosion observed in the structure, even at the locations of crack. This is because the structure is located in the non-severe environment (no chloride, low carbonation and low relative humidity).

Ultrasonic pulse velocity ws used to measure the crack depth. The measured results show that most of the observed crack depths on slab and beam surfaces are in the range of 30-80 mm. The observed crack depths were used to analyze the remaining load carrying capacity of the members. Although, shear resistance of the cracked members was significantly reduced, analytical results show that the remaining load carrying capacity of the structure is still 2 times greater than the design requirement.

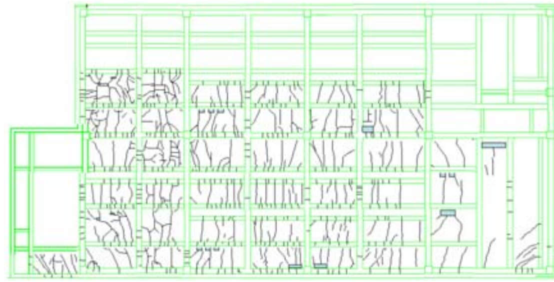

**Figure 4** Results of crack mapping.

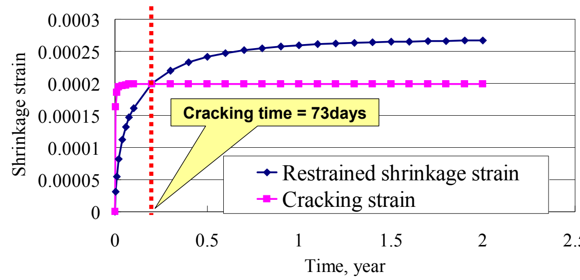


**Figure 5** Comparison between calculated shrinkage strain and cracking strain.

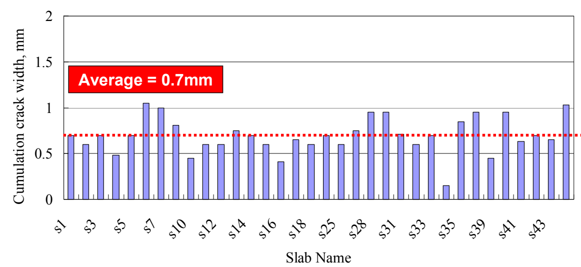


**Figure 6** Inspection results of cumulative crack width.

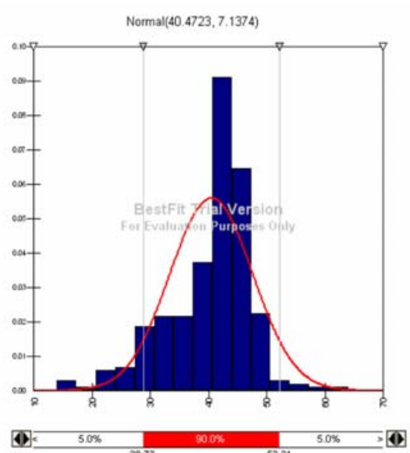


**Figure 7** Distribution of measuring result of covering depth.

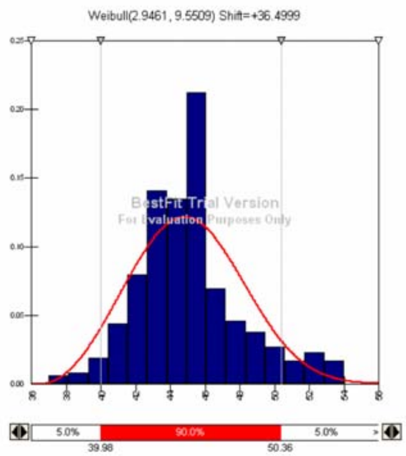


**Figure 8** Distribution of measuring result of rebound number.

*3.3 Service life evaluation.*

Although load carrying capacity of the structure is not a problem as discussed in the last section, structure durability has to be evaluated. In this study, service life of the structure is defined as the time that corrosion of reinforcing steel has been initiated (remedy-free service life). From the surrounding environment, the structure is considered vulnerable to carbonation. Therefore, carbonation depth is estimated by using Equations (5) to (7) [5, 6].

$$C \geq \gamma_i \cdot x_c \quad (5)$$

$$x_c = \alpha_1 \cdot \alpha_2 \cdot k \cdot \sqrt{t} \quad (6)$$

$$k = 17.5 \cdot k_r \cdot (w/b)^3 \quad (7)$$

where, C is reinforcing steel covering depth (mm) which can be obtained from the actual inspection result, $x_c$ is carbonation depth (mm) calculated from Equation (6), $\gamma_i$ is safety factor depending on service life of the structure, t is age of the structure (year), k is carbonation rate factor (mm/year$^{0.5}$) calculated from Equation (7), $\alpha_1$ is wetness factor, $\alpha_2$ is factor representing severity level of carbonation depending on relative humidity and concentration of $CO_2$, w/b is water to binder ratio, $k_r$ is factor representing effect of replacement ratio of fly ash on carbonation rate. The details of those equations can be found elsewhere [5, 6].

**Table 1** Corrosion initiation time due to carbonation.

| Percentile | Covering depth (mm) | Corrosion initiation time (year) | |
|---|---|---|---|
| | | Uncracked concrete | Cracked concrete |
| Percentile 5 | 28.73 | 137 | 4 |
| Percentile 50 | 40.50 | 272 | 8 |
| Percentile 95 | 52.21 | 452 | 14 |

**Table 2** Corrosion cracking time.

| Relative humidity (%) | Corrosion rate (μm/year) | Corrosion cracking time (year) |
|---|---|---|
| 40 | 0.03 | 42372 |
| 98 | 50 | 25 |
| 75 | 3.5 | 362 |

It was known that carbonation rate in cracked concrete is faster than that in uncracked concrete [7]. The carbonation rate increasing factor depends on crack width and w/b ratio. As average crack width obtained from inspection result is 0.19mm and w/b ratio is approximately 0.60, carbonation rate accelerating factor is 5.87 [7]. Then carbonation rate of cracked concrete can be calculated. Table 1 shows the corrosion initiation time of uncracked and cracked concrete at different percentile of covering depth obtained from the inspection result.

 

*3.4 Maintenance planning*

In this study, remedial actions is planned to be conducted when corrosion crack was generated. After corrosion was initiated, corrosion rate depends mainly on relative humidity and temperature of the surrounding environment. The structure in this study is located in relative dry and moderate temperature environment. Bentur et al. [8] concluded that corrosion rate at 75% relative humidity, which is the design criteria of the target structure, is 3.5 μm/year. Then corrosion amount can be calculated based on Equation (8).

$$w_{cr} = \left(7.87919 \times 10^{-3}\right) \cdot CR \quad (8)$$

where, $w_{cr}$ is corrosion amount ($mg/cm^2$-year) and CR is corrosion rate (mm/year). JSCE recommended that corrosion crack will be generated when corrosion amount reached $10 mg/cm^2$. Table 2 shows corrosion cracking time based on the above assumption. As shown, corrosion cracking time is 362 year at 75% relative humidity. Therefore, there is no need for remedial action currently. This is because the environment of this structure is not severe. However, it is recommended that periodical monitoring should be conducted.

## 4. Conclusion

Crack pattern observed on the target structure was mainly due to shrinkage of concrete with external restraint. Comparison between predicted concrete shrinkage with tensile cracking strain can be applied to determine the cracking time and cumulative crack width which correlate well with the inspection results. Service life of the structure was evaluated based on the actual inspection results. As shown, predicted remaining service life was very long. So there is no need for remedial action at this moment.

## 5. References

[1] Sancharoen, P., Kato, Y., and Uomoto, T., 2008, Probability based maintenance planning for RC structure attacked by chloride, *Journal of Advanced Concrete Technology*, Vol. 6, No. 3. (In pressed).

[2] JSCE (Japan Society of Civil Engineers), 2005, Standard Specifications for Concrete Structures-2001 "Maintenance", *JSCE*, 2005

[3] JSCE (Japan Society of Civil Engineers), 2005, Standard specifications for concrete structures-2002 "Structural performance verification", *JSCE*, 2005

[4] Tongaroonsri, S., Choketaweekarn, P., and Tangtermsirikul, S., 2005, Tensile strain capacity of concrete, *In Proceedings of the First Annual Concrete Conference*, Thai Concrete Association, 25-27October 2005, Rayong, Thailand. pp. CON74-CON81.

[5] Khunthongkeaw, J., and Tangtermsirikul, S., 2005, Model for simulating carbonation of fly ash concrete, *Journal of Materials in Civil Engineering*, Vol. 17, No. 5, pp. 570-578.

[6] Tangtermsirikul, S., and Khunthongkeaw, J., 2005, Maintenance-free service life design of concrete subjecting to carbonation, *In Proceedings of the International Workshop on Service Life of Concrete Structures – Concept and Design,* 4 February 2005, Sapporo, Japan. pp. 41-52.

[7] Kwon, S.J., 2005, Deterioration analysis in cracked concrete structures exposed to coupled chloride attack and carbonation, *Thesis of Doctoral Degree, Department of Civil Engineering*, Graduate School, Yonsei University, Korea

[8] Bentur, A., Diamond, S., and Berke N.S., 1997, Steel corrosion in concrete, *E&FN SPON*, 1st Edition, London, United Kingdom

 