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Maintenance of the deteriorated infrastructure in Thailand

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Abstract

Infrastructure has been aged and deteriorated by various causes. Maintenance of its safety and serviceability are crucial and many countries have raised concerns about the existing infrastructure. Infrastructure in Thailand has been also aged. This paper shows results of studies relating to maintenance of infrastructure in Thailand. Inspection of the existing infrastructure in Bangkok was conducted by both visual and non-destructive testing (NDT) showing many deteriorations, and damages. Most of the deteriorations are caused by low concrete covering depth and water leakage. While in marine environment, chloride attack is the main cause. Evaluation criteria for NDT results are studied and modified to be suitable for concrete used in Thailand. To predict the service life of structure, studies have also been conducted to evaluate the corrosion rate of reinforcing steel in reinforced concrete structures in different deterioration mechanisms and environmental conditions. Together with inspection results, service life of structure can be predicted, and maintenance can be well planned. The results of this study could raise awareness of the important of infrastructure maintenance in Thailand.

1. Introduction

The maintenance of infrastructure is crucial, as it is essential to ensure its safety and serviceability throughout its service life. Moreover, the infrastructure is designed for a very long service life such as 50 year or 100 year. As a result, the number of aged infrastructures is increasing. American Society of Civil Engineers (ASCE) reported evaluation results of infrastructure conditions in USA as grade C⁻ in 2021 which was improved from grade D+ in 2017 [1]. A number of research and developments have been conducted and implemented to ensure acceptable conditions of infrastructure. Moreover, USA passed the Bipartisan infrastructure law to continue invest more than 2 trillion USD to improve condition of their infrastructure in 2021. Japan Society of Civil Engineers (JSCE) also reported infrastructure conditions in Japan as grade B to D depending on type of infrastructure [2]. As a result, around 200 trillion JPY of infrastructure maintenance budget is required in the next 30 year. In Thailand, infrastructure is recently constructed compared to other developed countries. For example, the department of highways (DOH) has around 16,363 bridges in 2018 which their average ages are 30 year to 40 year [3]. Soon, infrastructure in Thailand will be aged and deteriorated more severely. To prepare for maintenance of infrastructure in Thailand, this paper presents results of studies including infrastructure condition inspection results, non-destructive testing and reinforcing steel corrosion rate measurement.

2. Methodology

In this paper, two main works were conducted. One is actual structural condition inspections to observe general structural deterioration condition in Bangkok. Another is an experimental study to measure electrochemical properties of corrosion of reinforcing steel including corrosion rate and concrete electrical resistivity.

2.1 Actual structural condition inspection

The elevated steel reinforced concrete roadway in Bangkok opened in 1981 was selected as a case study. Its structural conditions were inspected by visual according to DPT 1501 standard [4] and non-destructive tests. Details of non-destructive tests are shown in Table 1. Inspections were conducted on each main structural member such as column, cross beam, prestressed concrete beam, and slab.

2.2 Measure electrochemical properties of corrosion of reinforcing steel

2.2.1 Electrical resistivity of concrete

Electrical resistivity of concrete is easily measured by Wenner's four probe method according to EN 12390-19 [7] and DPT 1902 standard [8]. It can be used to evaluate corrosion risk of reinforcing steel as shown in Table 2. However, due to different types of binder used in concrete, concrete electrical resistivity is significantly affected. So, evaluation criteria of corrosion risk should be modified.

Concrete with different mix proportions such as water to binder ratio (0.4 and 0.6), types of binder (OPC and 30% fly ash replacement), and chloride content (0, 2, and 6 $kg \cdot m^{-3}$) were used to prepare 15 cm cube plain concrete specimens. Specimens were cured by plastic wrapped for 28 day before conditioning in high relative humidity sealed container. Electrical resistivity was measured at 30, 60 and 150 day on a side surface of specimens after being conditioned by Wenner's four probe method as shown in Figure 1.

2.2.2 Corrosion rate of reinforcing steel

The corrosion rate of reinforcing steel is very important for prediction service of structure. Corrosion rate depends on various factors both concrete and surrounding conditions. So, measurement of corrosion of reinforcing steel in different concrete and conditions were conducted.

Like electrical resistivity, different concrete mix proportions were prepared. Reinforcing steel was 150 mm long with of 12 mm diameter steel reinforcement and coated with epoxy paint at both ends leaving only 60 mm length as an exposed region as shown in Figure 2. Reinforced concrete specimens were prepared with a covering depth of 2 cm. Reinforcing steel was corroded by chloride attack, carbonation, and high moisture of concrete. Specimens attacked by chloride and high moisture, they were cured by plastics wrapped for 28 day and conditioned in high humidity closed container. For specimens attacked by carbonation, they were cured by plastics wrapped for 28 day and placed in carbonation chamber with 4% CO_2, 50% RH and 40°C to accelerate carbonation.

Linear polarization resistance (LPR) at period after being exposed of 0, 10, 27, 60, 120, 150, and 180 day were measured. The potentiodynamic polarization resistance (PPR) was also tested at the end of exposure period to obtain the Tafel slope for corrosion rate calculation.

The setup of electrochemical testing set up is shown in Figure 3. The test was performed by using Metrohm PGSTAT302N, three-electrode system. The working electrode (WE) was the embedded steel bar. The counter electrode (CE) was stainless steel plate that was placed on top of the exposed surface together with applying the conductive gel. The reference electrode (RE) was saturated copper/copper sulfate (CSE) electrode placed onto the center of the exposed surface area. The LPR was performed as following conditions:

Waiting for stable Open circuit potential (OCP) for 5 min.

Applying the potential of ± 10 mV about the OCP with the scan rate of 0.5 $mV \cdot s^{-1}$.

Measured polarization resistance was compensated for the Ohmic drop effect of the concrete cover as shown in Equation (1).

Figure 1. Measurement of concrete electrical resistivity.

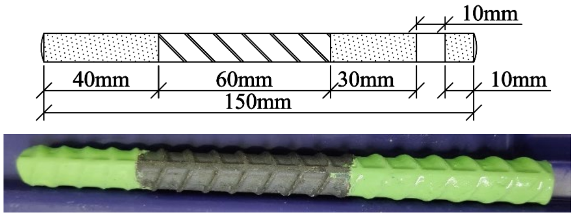

Figure 2. Reinforcing steel preparation.

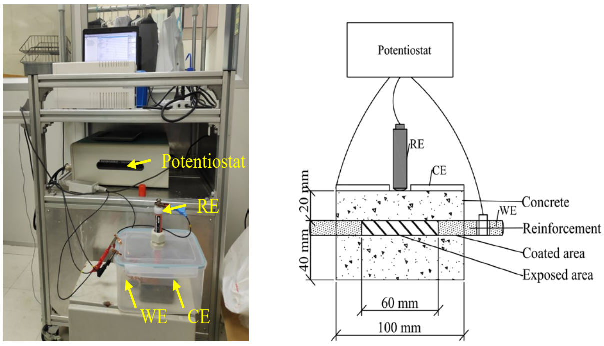

Figure 3. Electrochemical measurement of reinforcing steel.

Table 1. Non-destructive tests details.

Non-destructive test	Objective	Reference
Rebound hammer	Estimate concrete quality and compressive strength	[5]
Cover meter	Measure cover depth of reinforcing steel	[6]
Wenner's four probe	Measure electrical resistivity of concrete	[7] and [8]
Moisture meter	Measure moisture content of concrete	-
Carbonation depth	Measure pH neutralization depth of concrete	[8]

Table 2. Relationship between corrosion risk and electrical resistivity [9].

Corrosion risk	Electrical Resistivity (kOhm-cm)
Very high	< 5
High	5 – 10
Low	10 – 20
Moderate	> 20

Table 3. Classification of corrosion severity.

Description	Corrosion rate ($mg.cm^{-2}.year^{-1}$)	
	[10]	[11]
Passivity	< 0.91	< 1
Low corrosion	0.91 - 4.55	1 - 3.5
High corrosion	4.55 - 9.1	> 3.5
Extremely high corrosion	> 9.1	> 12

$$R_p' = R_p + R_c \quad (1)$$

where, R_p' is measured polarization resistance from the instrument (kΩ), R_p is the actual polarization resistance of the steel (kΩ) and R_c is electrical resistance of the concrete cover (kΩ).

While the PPR was performed as following conditions:

Waiting for stable Open circuit potential (OCP) for 5 min.

Applying the potential of ± 200 mV about the OCP with the scan rate of $0.5 mV \cdot s^{-1}$.

The corrosion current density (i_{corr}, $\mu A \cdot cm^{-2}$) of steel bar is determined by using Stern-Geary equation as shown in Equation (2).

$$i_{corr} = \frac{B}{A \times R_p} \quad (2)$$

where, B is calculated from Equation (3) and A is exposed surface area of reinforcing steel (cm^2)

$$B = \frac{\beta_a \times \beta_c}{2.303(\beta_a + \beta_c)} \quad (3)$$

where, β_a and β_c are anodic and cathodic Tafel slopes (mV), respectively, determined from potentiodynamic polarization resistance.

Corrosion rate (CR) in the unit of $mg \cdot cm^{-2}.year^{-1}$ can be calculated according to Faraday's law as shown in Equation (4)

$$CR = 3.65 \times K_2 \times i_{corr} \times EW \quad (4)$$

where, CR is mass loss rate of the steel ($mg \cdot cm^{-2}.year^{-1}$), K_2 = constant equals to 0.008954 $g \cdot cm^2 \cdot \mu A^{-1}.m^{-2} \cdot day^{-1}$ and EW is an equivalent weight equals to 27.92 for Fe.

The classification of corrosion severity based on corrosion rate is shown in Table 3.

3. Results and discussion

3.1 Actual structural condition inspection

Generally, visual inspection results show that structural condition of 30 years elevated roadway is still in good condition. There is no sign of structural deficiency, even some minor deterioration and damages were observed. This may be due to the structure was designed with high safety factor in the past. Figure 4 shows examples of damage frequently being observed. Water leakage due to joint damage or drainage system failure is the majority type of found damage. Even water leakage does not affect structural safety, but corrosion rate of reinforcing steel is increased and shortens the service life of structure. Consequence damage generated due to water leakage is also difficult to repair because of limited space. This will cause a large amount of repair budget in the future. Other damage observed were cracking, delamination, spalling of concrete surface.

Non-destructive inspection results as shown in Figure 5 to Figure 9 are used to evaluate structural conditions and identify the cause of damage. Rebound hammer results in Figure 5. show that estimated concrete compressive strength from rebound number is higher than requirements. Concrete covering depth is always a problem in Thailand due to bad quality control during construction. Results in Figure 6. show a significantly lower concrete covering depth than the requirement of 40 mm. Low covering depth increases the rate of ingression of aggressive substances such as chloride, carbon dioxide or moisture causing faster deterioration. The retaining wall, cantilever deck and slab showed lower covering depth than other members. As a result, spalling and delamination of concrete surface were largely observed on those members due to corrosion of reinforcing steel. In Figure 7 and Figure 8, results of moisture content and electrical resistivity mainly indicate presence of moisture or chloride in concrete. Both will increase the corrosion rate of reinforcing steel. Similarly, members that show high moisture content or low electrical resistivity due to water leakage or exposed to sources of water/moisture/chloride such as rain, soil, splash water, etc. show higher degree of damages. Surprisingly, results of average carbonation depth in Figure 9 are less than 10 mm even structure is in traffic congested area for more than 30 year which is significantly lower than covering depth. Therefore, delamination/spalling of concrete surface of structure observed in this case study was caused by corrosion of reinforcing steel due to high moisture content in concrete and low concrete covering depth. To delay deterioration of this structure in the future, controlling moisture content in concrete is recommended. Repairing crack on deck surface, preventing drainage blocked, repairing water leakage along drainages/damaged joint, preventing accumulation of surface water or applying waterproof coating layer should be done.

Figure 4. Damage observed by visual inspection (a) water leakage (b) spalling (c) spalling and delamination (d) crack.

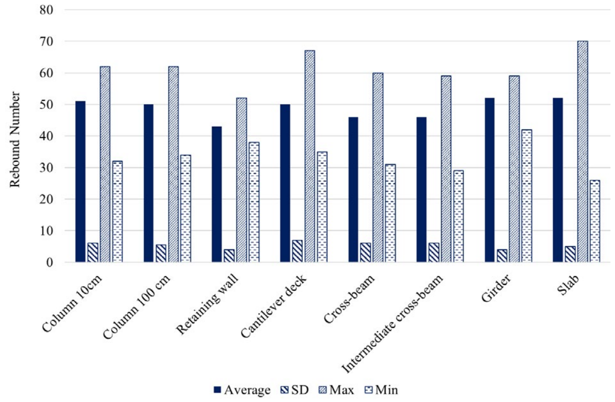

Figure 5. Results of rebound number of concrete.

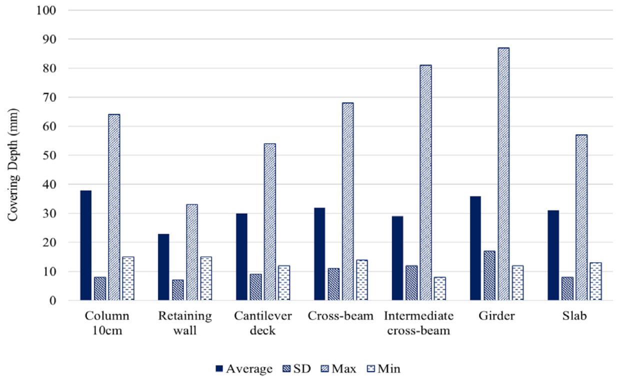

Figure 6. Results of concrete covering depth.

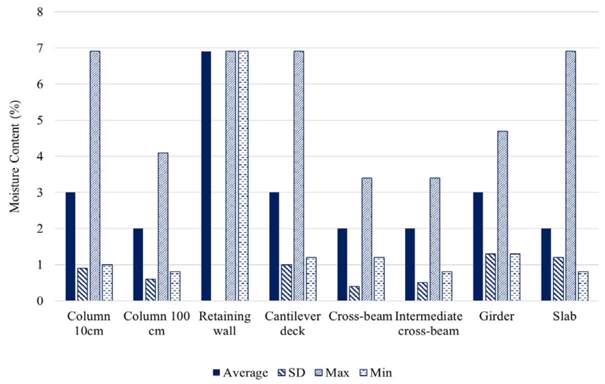

Figure 7. Results of concrete moisture content.

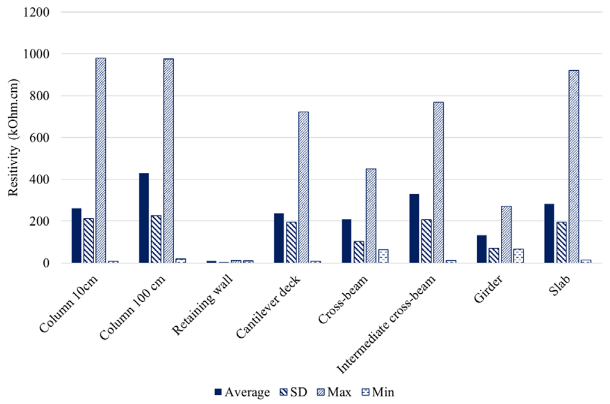

Figure 8. Results of concrete electrical resistivity.

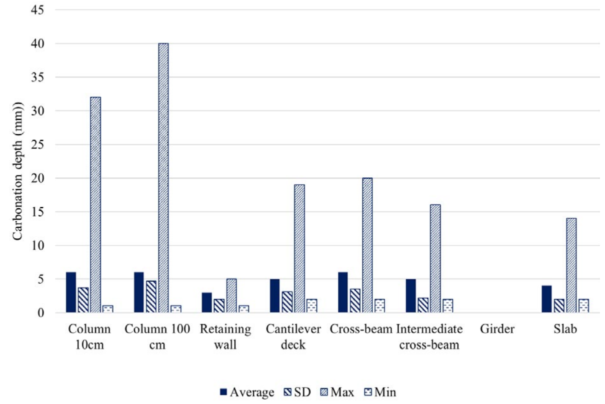

Figure 9. Results of carbonation depth.

3.2 Electrochemical properties of corrosion of reinforcing steel

3.2.1 Electrical resistivity of concrete

Figure 10 shows results of effects of concrete mix proportion on concrete electrical resistivity. Specimens were named as w/b-Binder type-Chloride content. For example, 4PCCL2 means water binder 0.40, ordinary Portland cement concrete and chloride content 2 $kg \cdot m^{-3}$. 6FACL0 means water binder 0.60, coal fly ash concrete and chloride content 0 $kg \cdot m^{-3}$. As shown, the type of binder significantly affects electrical resistivity. When fly ash is used, electrical resistivity of concrete increases significantly due to densification and disconnecting of pore structure and changing of pore solution of concrete [12,13]. Higher chloride content also decreased electrical resistivity significantly due to hygroscopic properties of chloride salt. Increased moisture content in concrete pore solution. Specimens with longer age, electrical resistivity increased. This is due to cement hydration progressing with time causing denser pore structure. While water binder ratio affecting strength of concrete show minor effect on electrical resistivity. Even low water binder reduces the size of pore structure, but they are still connected.

As a result, classification of corrosion risk based on electrical resistivity measurement as shown in Table 2 is not applicable for fly ash concrete. For example, resistivity of 6 $kg \cdot m^{-3}$. chloride contaminated fly ash concrete (4FACL6 and 6FACL6) as shown in Figure 10 were higher than 20 $kOhm \cdot cm^{-1}$. But corrosion of reinforcing steel has already been initiated because chloride content is higher than critical chloride content of 1.2 $kg \cdot m^{-3}$. According to Table 2, corrosion risk is moderate that is not correct. So, a new evaluation guideline for corrosion risk is proposed in the next section.

3.2.2 Corrosion rate of reinforcing steel

Examples of results of LPR and PPR of reinforcing steel are shown in Figure 11 and Figure 12, respectively. Corrosion rate of reinforcing steel exposed to different environmental conditions can be calculated based on compensated LPR and the Tafel slope according to Equation (1-4). By comparing calculated corrosion rate of reinforcing steel and measured electrical resistivity of concrete, relationship between corrosion risk and electrical resistivity based on Alonso *et al.* [9]

was modified to consider effect of fly ash as shown in Table 4. So, classification of corrosion risk of reinforcing steel in fly ash concrete can be more accurate. The details are shown below.

Corrosion rate of reinforcing steel was calculated according to Equation (1-4).The measured parameter values used in calculation were reported by Dong *et al.* [11,13].

A relationship between measured concrete electrical resistivity, chloride content and corrosion rate are proposed as shown in Equation (5-6) independently between OPC concrete and FA concrete because FA concrete has higher electrical resistance than OPC concrete [16].

For OPC concrete:

$$CR = \frac{18.4}{\rho} + 1.33 \cdot \left(\left[CL^{-}\right] + 0.1\right)^{1.15} - 1.15 \qquad (5)$$

For fly ash concrete:

$$CR = \frac{31.9}{\rho} + 0.79 \cdot \left(\left[CL^{-}\right] + 0.1\right)^{0.78} - 0.6 \qquad (6)$$

where CR is the corrosion rate of reinforcing steel ($mg \cdot cm^{-2} \cdot yr^{-1}$), ρ is the electrical resistivity of concrete ($k\Omega \cdot cm$), [Cl^{-}] is the content of chloride in concrete ($kg \cdot m^{-3}$).

The corrosion rate of reinforcing steel in OPC concrete at the criteria of concrete electrical resistance [9] is determined from Equation (5) as shown in Table 4.

Electrical resistance of FA concrete is calculated from Equation (6) as a proposed criteria to justify severity of corrosion for FA concrete as shown in Table 4. According to JSCE [14], concrete with a covering depth of 10 mm tends to be cracked by corrosion of reinforcing steel when corrosion amount reached 10 $mg \cdot cm^{-2}$. Corrosion cracking time is classified as 4 categories as shown in Table 5. Then corrosion rate to cause cracking time according to classified categories is determined. From the relationship between corrosion rate and electrical resistivity as shown in Equation (5-6), corrosion cracking time can be classified based on measured electrical resistivity as shown in Table 5. The remaining service life of structure until cracking can be evaluated by concrete electrical resistivity.

The corrosion rate of reinforcing steel in concrete exposed to different environments such as chloride, carbonation and high moisture were also calculated according to Equation (1-4) and measured parameters values and compared as shown in Table 6 [11,15,16].

As expected, reinforcing steel attacked by chloride shows the highest corrosion rate. Corrosion rate in carbonated concrete depends on moisture content of concrete. Wet concrete shows a higher corrosion rate due to corrosion current flow easier. For uncarbonated concrete, reinforcing steel is still passivated. Then, corrosion current flow difficultly. The corrosion rate is low. According to JSCE [14], corrosion cracking time after corrosion initiation due to chloride, dry carbonated, wet carbonated and wet uncarbonated concrete are 2.85, 14.29, 7.14, and 25 year, respectively.

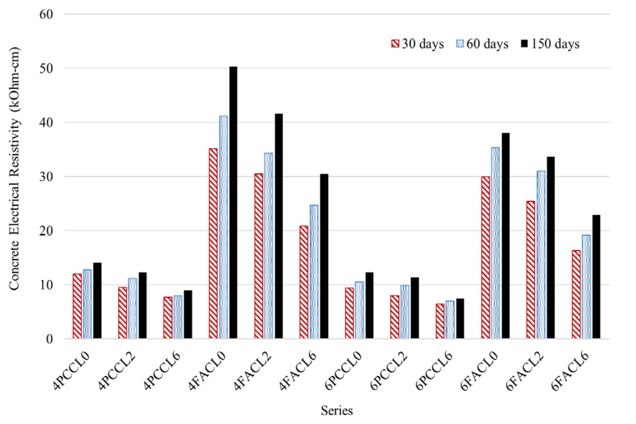

Figure 10. Effects of concrete mix proportion on concrete electrical resistivity.

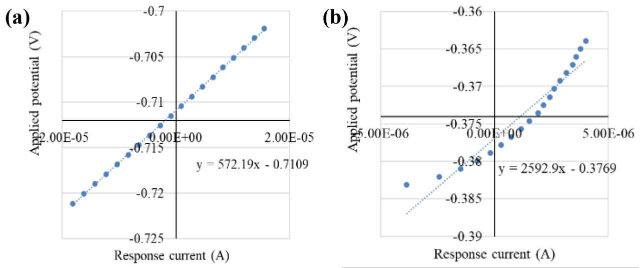

Figure 11. Example of linear polarization resistance results of (a) depassivated steel (b) passivated steel.

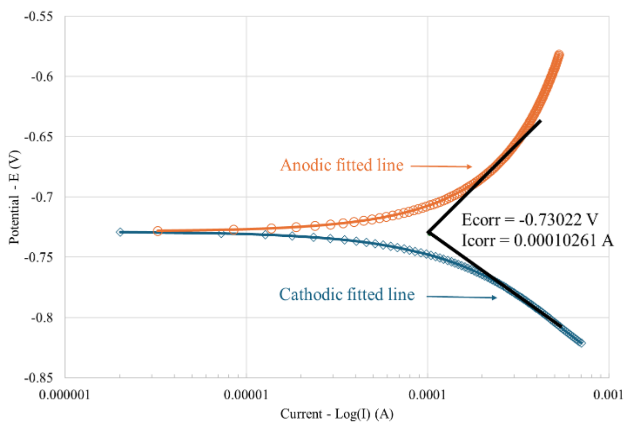

Figure 12. Example of the Tafel slope from potentiodynamic polarization results.

Table 4. Relationship between corrosion risk and electrical resistivity with considering effect of fly ash.

Corrosion risk	Corrosion rate ($mg \cdot cm^{-2} \cdot year^{-1}$)	Electrical Resistivity ($kOhm \cdot cm^{-1}$)	
		OPC concrete [9]	Fly ash concrete
Very high	> 22	< 5	< 5
High	4 to 22	5 to 10	5 to 20
Moderate	0 to 4	10 to 20	20 to 50
Low	0	> 20	> 50

Table 5. Relationship between corrosion cracking time and electrical resistivity.

Corrosion cracking time	Corrosion rate	Electrical Resistivity (kOhm·cm^{-1})	
(year)	(mg·cm^{-2}·year^{-1})	OPC concrete	Fly ash concrete
> 10	< 1	> 13	> 38
5 – 10	1 – 2	11 – 13	29 – 38
3 – 5	2 – 3.3	10 – 11	22 – 29
< 3	> 3.3	< 10	< 22

Table 6. Corrosion rate of reinforcing steel in different environment.

Environment	Corrosion rate (mg·cm^{-2}·year^{-1})
Chloride	3.50
Carbonation - Dry concrete	0.70
Carbonation - Wet concrete	1.40
Uncarbonated - Wet concrete	0.40

4. Conclusions

Maintenance of infrastructure is important to ensure their safety and serviceability. Deterioration and damage of structure depend on various factors. Results of damages inspection of target structures show no significant structural damage. But poor maintenance of joint and water drainage system causes a faster deterioration rate and affects service life of structure in the future. Structural properties were inspected by non-destructive tests. Results showed that low concrete covering depth is the main problem due to bad workmanship during construction. Combined with the water leakage problem, damage to structure is accelerated. Concrete electrical resistivity is easily conducted to evaluate corrosion risk of structure. A new guideline is proposed to consider effect of fly ash normally used in Thailand for more precise corrosion risk evaluation. Also, a new guideline to estimate corrosion cracking time based on electrical resistivity is proposed. Finally, corrosion rate of reinforcing steel in different environments is proposed based on experimental results. Therefore, structure service life can be predicted.

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