Evaluation of repair effect for chloride-damaged reinforced concrete by using patch repair materials containing fly ash

フライアッシュを含む断面修復材による塩害劣化鉄筋コンクリートの補修効果の評価

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Abstract

As a conventional repair method for reinforced concrete (RC) structures deteriorated by the chloride attack, the patch repair has been applied widely. When the patch repair method is applied, the re-deterioration has been sometimes observed around the joint part of the substrate concrete and the repair material. One of the major deterioration mechanisms after the patch repair is the macro-cell corrosion caused by the remained chlorides in concrete or by the corrosive products like chloride ions (Cl^-) supplied through the inadequate joint part. In this study, the effects of fly-ash-mixing and the dosage of $LiNO_2$ in the patch repair materials on the basic properties of repair materials, and on the penetration due to the chloride around the interface between the substrate concrete and the repair material were investigated.

In this study, the effects of fly-ash-mixing and the dosage of $LiNO_2$ in the patch repair materials on the basic properties of repair materials, the ability of repair materials to resist chloride ion penetration and on the chloride-induced steel corrosion due to the supply of Cl^- ions around the interface between the substrate concrete and the repair material were investigated. As a result of this study, the fly-ash-mixing decreased compressive strength of repair mortar a little but the dosage of $LiNO_2$ could increase the tensile strength and the bond strength. The effective diffusion coefficients of Cl^- obtained by using the concrete-PCM-joint specimens were

larger than the cases of using concrete-mortar-joint specimens. The patch repair material like PCM showed high resistance to Cl^- penetration but seemed to promote the Cl^- penetration into concrete near the joint surface between concrete and patch repair materials. On the other hand, the PCM containing both fly ash and $LiNO_2$ showed a high protection effect against steel corrosion in the RC joint specimens and the macro-cell current around the joint face was also suppressed compared with the case of a general PCM.

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Chapter 1 Introduction

1.1 Background

Both Japan and Taiwan are an island state surrounded by oceans. Therefore, reinforcement concrete (RC) structures in coastal are prone to chloride-induced steel corrosion. When steel corrodes, the rust (oxidation products) would lead to volume expansion of steel, and then induced concrete peel off. It influences the durability of RC structures. In order to solve this issue, patch repair is one of the methods to repair this kind of deteriorated structures. However, how to choose a suitable repair material according to the condition is very important. Commonly, mechanical properties as selected standard to repair materials, and the ability to resist the deteriorate factors also plays an important role. If the repair material cannot resist chloride-induced corrosion effectively, the repaired RC structures would re-deteriorate again which influence the durability of repairing RC structures. Furthermore, the bonding strength is also one of the important in choosing repair material. If the compatibility is weak of repair material and concrete, the interface would become the weak point to let the deteriorate factor attack the repaired RC structure. In conclusion, choosing suitable materials in repair method is important to increase the durability of RC structures.

1.2 Research motivation and objective

In Japan and Taiwan, patch repair is the very common method to repair the corroded RC members. Because Portland cement mortar is low cost and high compatibility with substrate concrete, it is widely used to repair deteriorated RC structures in Japan and Taiwan. However, the Portland cement mortar could not effectively resist the degradation factor, such as chloride ions (Cl^-), water, carbon dioxide (CO_2), oxide (O_2) and so on. According to the past researches, mortar-based patch repairing materials are added to the polymeric material to improve the mechanical properties, e.g. compressive strength, tensile strength, bonding strength and flexural strength. The polymer-modified cement is considered to solve the problem of degradation factor attacking and increase the durability of the repaired RC structures. However, the polymeric material is very expensive compared to the cement.

Fly ash is a byproduct in thermal power generation. Fly ash is used as a supplementary cementitious material, when used in conjunction with Portland cement, contributes to the properties of the hardened concrete through hydraulic or Pozzolanic activity. Its Pozzolanic properties provide good durability and achieve economic benefits in the construction without sacrificing its intensity. The advantage of fly ash is not only reducing the cost, and the ability to resist Cl^- is also outstanding. On the other hand, the rust prevention effect of NO_2^- had been found in many researches which can regenerate the passive film to control the formation of rust. It is very useful to increase the durability of repaired RC structures. In this study, the effects of fly-ash-mixing and the dosage of $LiNO_2$ in the patch repair materials on the basic properties of repair materials, the ability of repair materials to resist chloride ion penetration and on the chloride-induced steel corrosion due to the supply of Cl^- ions around the interface between the substrate concrete and the repair material were investigated.

Chapter 2 Literature Review

RC structures have been taken an important role as the infrastructure in the world. From the 1970s to the present, through years of wind, rain, and frequent earthquake, many RC buildings or structures have shown deterioration phenomenon, even cause danger. However, these structures constructed in the coastal area or those affected by de-icing salt have been often deteriorated by the chloride attack (Page 1975, Glass and Buenfeld 1997). There are many methods for chloride penetration of crack-free concrete, such as diffusion, capillary absorption, hydrostatic pressure and evaporative transport. Among them, diffusion is predominant. Diffusion occurs when the concentration of chloride on the outside of the concrete member is greater than on the inside. This chloride ions moving through the concrete to the steel. The conditions when wet-dry cycles and oxygen are easy for it to occur steel corrosion. The corrosion products (iron oxides) of steel member may induce mechanical stress that can cause the structure of cracks and damage to the concrete. As the conventional repair method for RC structures deteriorated by the chloride attack, the patch repair has been applied widely.

2.1 The chloride corrosion mechanism

Steel bars embedded in concrete mainly protected by two mechanisms to prevent corrosion, including physical and chemical properties. In terms of physical mechanism, concrete blocks the harmful ions from the outside. In terms of chemical mechanism, the pore aqueous solution in concrete is highly alkaline (pH = 12.5 to 13.5), which helps to form a passive film on the surface of the steel, protects the steel from the intrusion of harmful factors, and provides durability. In order to achieve a stable equilibrium, iron will releases electrons and forms low-energy and stable compounds with other elements. Corrosion is a condition in which a metal reacts with the surrounding environment to cause damage. Whether the corrosion will happen or not depending five conditions:

(1) Anode

In the electrochemical reaction, a phenomenon of charge transfer is performed on the electrode plate, wherein an anode is a place where the oxidation reaction occurs, and the metal emits electrons to form an ion state.

$$Fe \rightarrow Fe^{2+} + 2e^{-} \tag{2-1}$$

(2) Cathode

The cathode reaction is the reduction of water, which water absorbs free electrons to form hydroxide ions in the basic solution.

$$H_2O + \frac{1}{2}O_2 + 2e^{-} \rightarrow 2OH^{-} \tag{2-2}$$

The iron ions and hydroxide ions (OH^-) in the anode and cathode can reactions into ferrous hydroxide.

$$Fe^{2+}2(OH)^{-} \rightarrow Fe(OH)_2 \tag{2-3}$$

Then the ferrous hydroxide can react with oxygen in the water to form iron hydroxide.

$$4Fe(OH)_2 + 2H_2O + O_2 \rightarrow 4Fe(OH)_3 \tag{2-4}$$

$$Fe(OH)_3 \rightarrow FeO(OH) + H_2O \tag{2-5}$$

$$3Fe + 8OH^{-} \rightarrow Fe_3O_4 + 8e^{-} + 4H_2O \tag{2-6}$$

The $FeO(OH)$ is red rust and the Fe_3O_4 is black rust.

(3) Conductive path

The electrochemical transfer is required for the oxidation and reduction reaction in the electrochemical reaction. In a poorly conductive path, the corrosion reaction will be slowed down and difficult to proceed.

(4) Current

Corrosion must have a sufficient amount of electrons to allow the reaction to proceed smoothly. Sufficient electromotive force can drive the flow between the electrons to form a current, conduce to the oxidation or reduction reaction will proceed smoothly.

(5) Electrolytic solution

Electrochemical corrosion must react in a humid environment. When this environment, the movement rate of ions is faster than the diffusion of metal oxides, so the rate of corrosion will be faster.

These five conditions form a group of systems that can be defined as an electrochemical cell.

2.2 The chloride attack deteriorating process of concrete structure

According to Japanese concrete standard specification for concrete structures-Maintenance (JSCE 2013), the definitions of chloride attack deterioration stages are shown in Table 1 and grades of the appearance of structure and state of deterioration are shown in Table 2. There are four stages of chloride attack deterioration which are the initial stage, propagation stage, acceleration stage and deterioration stage.

Under the initial stage, it is decided by initially contained chloride ion concentration and diffusion of Cl^-. Before the chloride ion concentration on the surface of steel reaches the marginal concentration, corrosion would not happen. Therefore, there is no defect of appearance. Under propagation stage, it is corrosion initiates. However, there is no defect of appearance is found. Under acceleration stage, cracks occur due to the expansion of the rust products so that the rate of steel corrosion would increase. The change of appearance can separate into two sessions. First, when cracking occurs due to corrosion, leaching of rust would be observed. Then, numerous cracks occur due to corrosion so that not only leaching of rust is observed but partial peeling or spalling is found. In other words, the cracks happen lead corrosion amount of steel to increase. Under deterioration stage, load bearing capacity is reduced considerably due to the increase of corrosion amount. However, not only leaching of rust is observed in this stage but peeling and spalling are found and crack width is large. Great displacement and deflection also can be observed in this stage.

Japanese concrete standard specification for concrete structures - maintenance also provide some repair method for chloride attack deteriorate RC constructions to achieve the designated required performances in every deteriorate stage which is shown in Table.3.

Table 1 Definitions of chloride attack deterioration stages

Stage of chloride attack deterioration	Definition	Stage determined by
Initiation stage	Until the chloride ion concentration on the surface of steel reaches the marginal concentration for the occurrence of corrosion	1. Diffusion of Cl^- 2. Initially contained chloride ion concentration
Propagation stage	From the initiation of steel corrosion until cracking due to corrosion	Rate of steel corrosion
Acceleration stage	Stage in which steel corrodes at high a rate due to cracking	Rate of steel corrosion of steel with cracks
Deterioration stage	Stage in which load bearing capacity is reduced considerably due to the increase of corrosion amount	

Table 2 Grades of the appearance of structure and state of deterioration

A grade of structural appearance	State of deterioration
I-1 (initiation stage)	No defect of appearance is found. The marginal chloride ion concertation for the occurrence of corrosion has not been exceeded.
I-2 (propagation stage)	No defect of appearance is found. The marginal chloride ion concertation for the occurrence of corrosion has not been exceeded. Corrosion initiates.
II-1 (first half of acceleration stage)	Cracking occurs due to corrosion. Leaching of rust is observed.
II-2 (second half of acceleration stage)	Numerous cracks occur due to corrosion. Leaching of rust is observed. Partial peeling or spalling is found. The corrosion amount of steel increases.
III (deterioration stage)	Numerous cracks occur due to corrosion. Crack width is large. Leaching of rust is observed. Peeling and spalling are found. Great displacement and deflection are observed.

Table 3 Grades of structural appearance and standard methods of repair or strengthening

A grade of structural appearance	Standard method
I-1 (initiation stage)	Surface treatment*
I-2 (propagation stage)	Surface treatment, patching, cathodic protection, and electrochemical desalination
II-1 (first half of acceleration stage)	Surface treatment, patching, cathodic protection, and electrochemical desalination
II-2 (second half of acceleration stage)	Patching
III (deterioration stage)	FRP bonding, patching, external cable, jacketing and thickness increasing

*Preventive method

In propagation stage, surface treatment is selected generally. Although steel bar is corroded, the amount of corrosion is low and the performance degradation is not serious. However, sometimes patching or electrochemical desalination is chosen to remove the Cl- had already intruded into the concrete. Cathodic protection method is chosen to prevent steel corrosion from progressing.

In the first half of the deterioration acceleration stage, electrochemical desalination method for removing the Cl- from concrete or patching repair for removing the concrete intruded by Cl- because cracks happen in concrete cover due to steel corrosion. Therefore, surface treatment and cathodic protection method are also considered after patching to prevent further corrosion of steel bar.

In the second half of acceleration stage, utilizing patch repair to remove the concrete intruded by Cl- and applying surface treatment simultaneously is advised. Furthermore, it also needs to consider the replacement of steel bars to avoid the repaired structure to re-deteriorate again.

In the deterioration, the stage has been reached and the load-bearing capacity been reduced. Methods for restoring the load bearing capacity by such means as the adhesion of steel or FRP plates/sheets, use of external cable, jacketing and thickness increasing should be applied in addition to the patch repair through the addition or replacement of steel bars. These methods should be implemented after removing the concrete cover and applying surface treatment for preventing re-deterioration.

2.2 Method of patch repair

According to the Concrete Diagnostic Technology of the Japan Concrete Engineering Society, the maintenance and management procedures for the implementation of concrete structures are shown in Fig.1. In order to maintain the performance of the concrete structure, it is necessary to regularly maintain and manage the repair according to the current degree of deterioration of the concrete. The main purpose of repair is to suppress or degrade a deteriorated component, then Increase or restore the durability. The concrete repair method is shown in Fig. 2. The repair work must select the appropriate repair method and materials according to the deterioration factors and the degree of component deterioration. In general, depending on the type of structural deformation, the deterioration mechanism and the degree of cracking, the construction method may be performed alone or in combination with a plurality of construction methods.

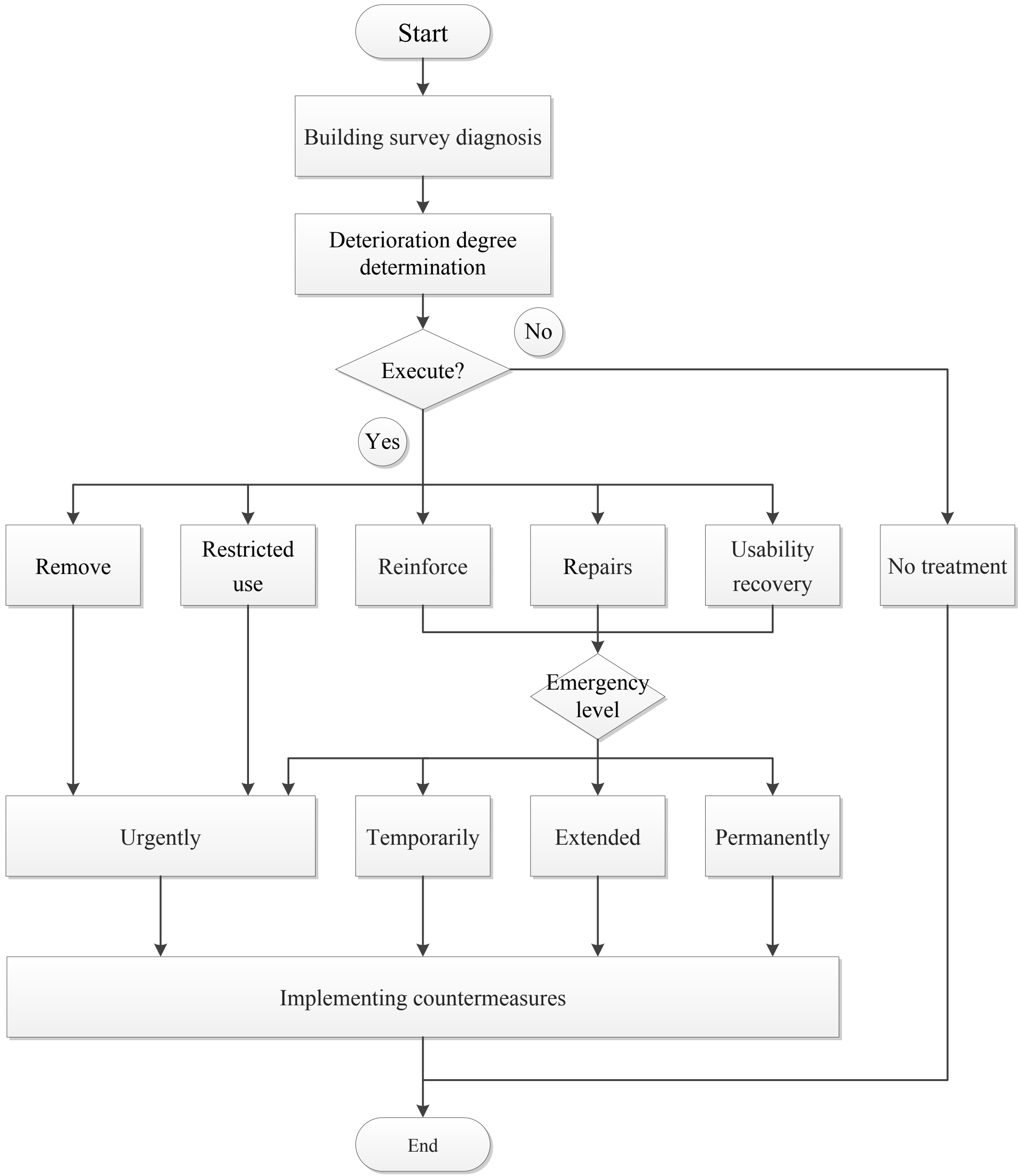

Fig. 1 Concrete structure implementation maintenance management process

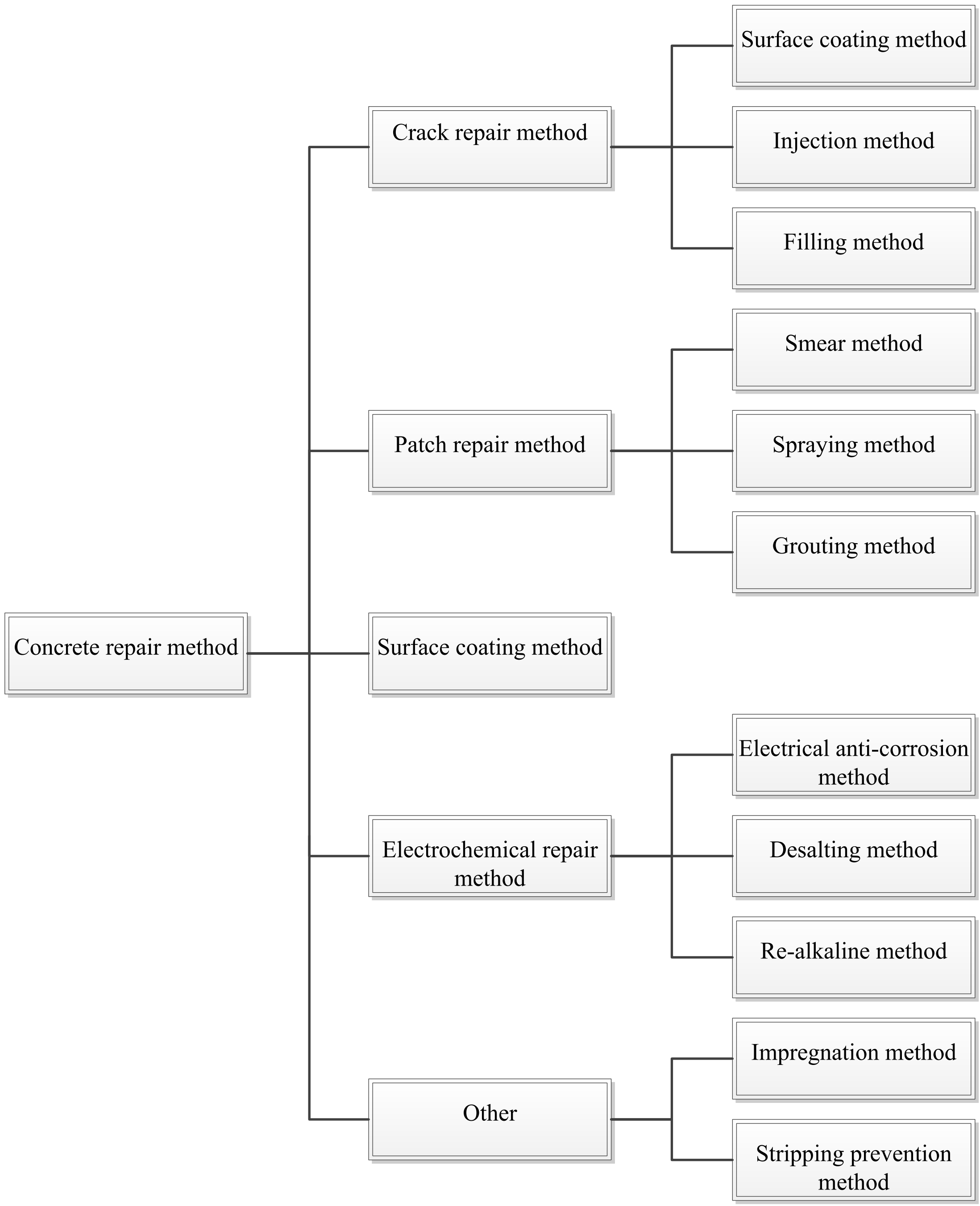

Fig. 2 Concrete repair method

The patch repair is usually applied in the case of repairing the un-filling region or low-quality region of RC structures, the crack and peeling of concrete, the corrosion-induced deteriorated concrete and removing the deterioration factor which has diffused in the concrete such as chloride ions (Makishima et al. 2010, JSCE 1997). For the corrosion-induced deteriorated concrete region in an RC member, the patch repair method generally includes removing all the substrate concrete with the high chloride content and then applying rust treatment on the steel bar (Makishima et al. 2010, Ohama 1996). Additionally, a suitable finishing material should be used to cover the repair area to renew the surface and seal the defective region, as shown in Fig. 3. According to the Japanese recommendation guideline for concrete repair and surface protection of concrete structures (AIJ 1997), it shows three practical methods of the patch repair for various deteriorated regions in RC members including plastering method, the spraying method, and filling processing method. Fig. 4 shows that the application range of patch repair. It is necessary to choose the appropriate materials on the basis of the specified performance, standard and construction method.

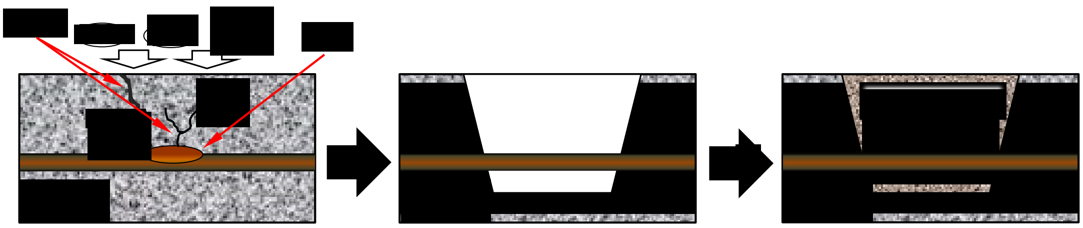

Fig. 3 Patch repair method

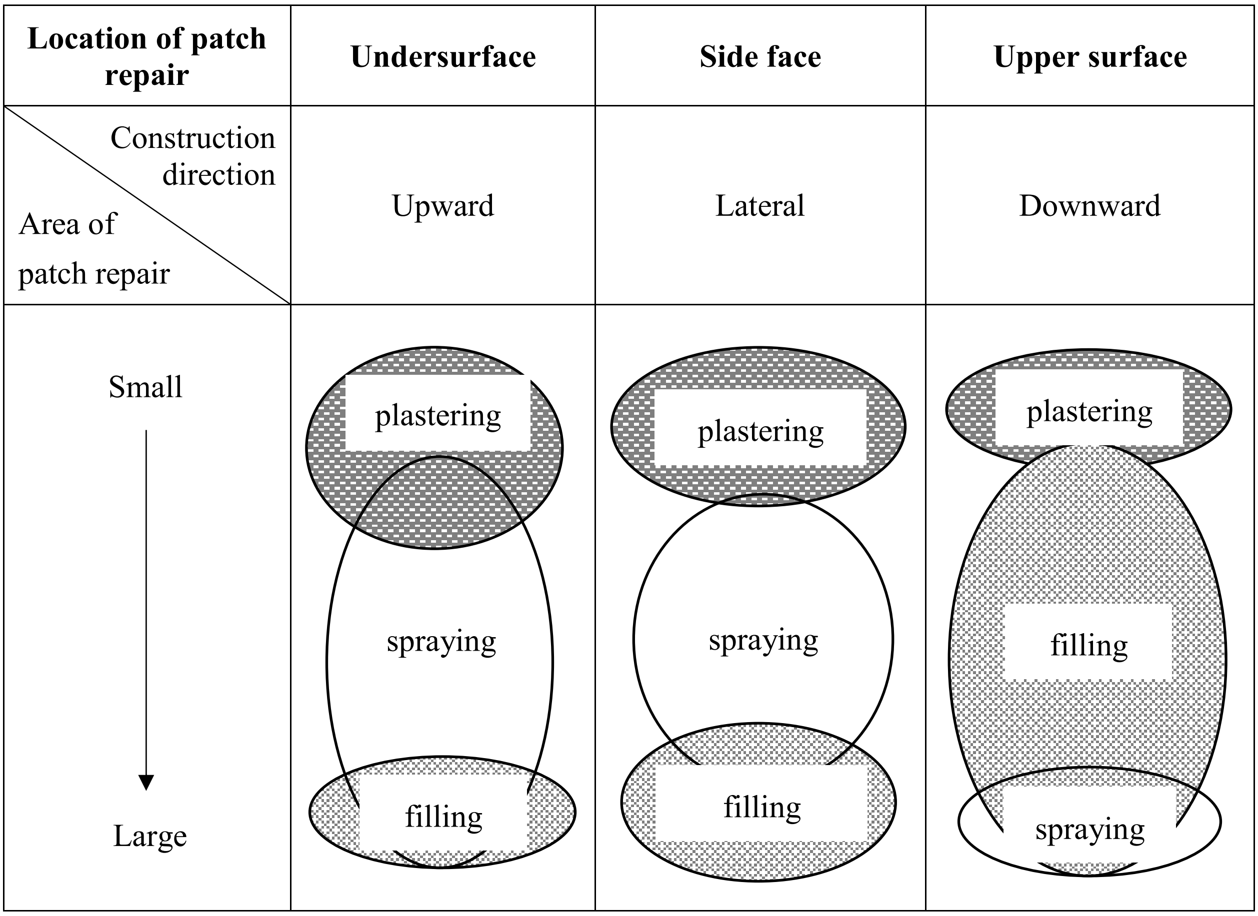

Fig. 4 The application range of patch repair

2.3 The necessary performance of patch repair materials

The materials used in the patch repair basically needs to have the higher mechanical properties than concrete (e.g., compressive strength, flexural strength and tensile strength) and good compatibility with the substrate concrete (AIJ 1997). However, in addition to the mechanical properties, the physical properties should be considered to select appropriate materials in the repair work. For example, if the compressive strength of the repair material is much higher than the substrate concrete, it is easy to let the repaired region to take the unexpected excessive loading. Therefore, although the high cement dosage in the repair material can increase the compressive strength effectively, the strength deterioration may be induced by the crack or dry shrinkage (CHES 2005, Li 2000, Cusson and Mailvaganam, 1996, Morgan 1996). Elastic modulus is an important measure of rigidity. The higher modulus material imposes a serious constraint on the transverse contraction of the lower modulus material. To avoid this kind of situation happens, the repair material should be chosen similar elastic modulus with substrate concrete (Li 2000). Dry shrinkage is the property of cement paste. The existing substrate concrete restrains the shrinkage of the repair material so that the stress is induced on repair material. The stress makes the crack occur easily. When the crack is happening, the structure would re-deteriorate again. As the result, it is recommended to choose low dry shrinkage rate or no dry shrinkage products for the repair material. In nowadays, high early-age strength materials are used commonly so as to meet construct requirement. Although the early-age strength is increased, the properties of the dry shrinkage and thermal expansion are also increased. In order to meet the requirement of construction, it is necessary to avoid using high early-age strength products in repair material (Li 2000, Cusson and Mailvaganam 1996, Morgan 1996, Abbasnia et al. 1913). Additionally, if the repair material doesn' t have enough the bonding strength with the substrate concrete, it is not possible to perform its original properties. The treatment of interface also plays an important role in the substrate concrete, such as removing the deteriorated concrete part and cleaning the dust on the substrate concrete interface and so on (Li 2000, Morgan 1996).

The three repair materials that are used in construction are polymer-modified cement mortar, epoxy mortar, and cement mortar. There are many commercial repair materials and the most of them are classified as polymer cement mortar (PCM). The advantages of polymer-modified cement mortar are its strong bonding with substrate concrete and its low shrinkage rate, which disfavors cracking. It also exhibits high flexural strength, tensile strength, and resistance to deterioration. It is also easy to mix and use in construction. Some experiments have shown that cement-based materials form stronger bonds with substrate concrete than do resin materials, while polymer-modified cement has higher bond strengths. When the patch repair method is applied, the re-deterioration has been sometimes observed around the joint part between the substrate concrete and the repair material (Kim et al. 2016, Park et al. 2009, Park 2012). One of the major deterioration mechanisms after the patch repair is the macro-cell corrosion caused by the remained chlorides in concrete or by the corrosive products supplied through the inadequate joint part (Pruckner and Gjørv

2002, Barkey 2004). Past papers reported that the macro-cell corrosion was promoted when repair materials have the high electrical resistance like PCM due to the difference of the electrical properties between concrete and repair materials (Gu et al. 1997, Nanayakkara and Kato 2009, Miyazato and Otsuki, 2010).

On the other hand, fly ash, a byproduct from a thermal power plant, has many advantages to enhance concrete performances, such as pore refinement, environmental protection, cut costs (Tahir and Sabir, 2005), increase flowability of fresh concrete and late strength of concrete, improve resistance against ASR or ingress of corrosive factors, and so on (Bilir et al. 2015, Fanghui et al. 2015). However, a shortage of fly ash is currently resulting in low early-age strength (Thomas, 2007). If fly ash is mixed into repair materials like PCM, a dense pore structure of the materials with relatively low electrical resistance would be achieved in addition to the ecological advantage.

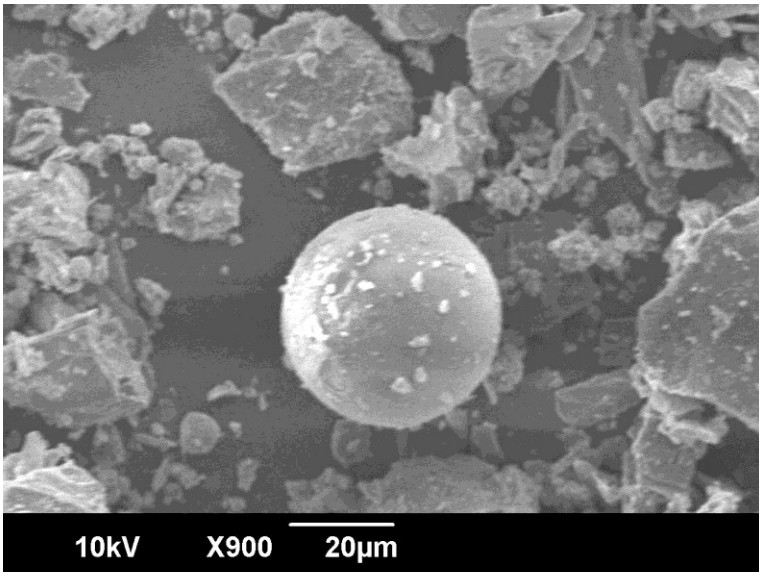

Fig. 5 The Scanning Electron Microscope (SEM) shoot a picture of fly ash

Moreover, past papers reported that $LiNO_2$ could prevent corrosion of steel in concrete effectively. Lithium nitrite and calcium nitrite are generally used in the construction industry as a means to protect reinforced concrete structures from corrosion. A study was conducted in Korea to experimentally determine the most effective dose and performance of Lithium Nitrite corrosion inhibitors. This experiment employed the molar ratio of nitrite ions to chloride ions (NO_2^-/Cl^-) as a test parameter. This study concluded that a lithium nitrite dosage of 0.6 in the nitrite-chloride ion molar ratio is a successful dosage for mortar containing chlorides (Lee and Shin 2007).

Chapter 3 Mechanical Properties Of Repair Materials

3.1 Materials

Mix proportion of concrete used for specimens is shown in Table 4. Water to cement ratio (W/C) was 60 %. Ordinary Portland cement (density: 3.16 g/cm^3, specific surface area: 3280 cm^2/g, R_2O: 0.56 %), fine aggregates (density: 2.57 g/cm^3, F.M.: 2.79), coarse aggregate (density: 2.57 g/cm^3，Gmax: 15 mm) were used. The air content and slump of concrete are 4% and 9 cm. The compressive strength of concrete after curing for 28 days was 36.9 N/mm^2. The concrete used for making the RC joint specimens were premixed Cl^- of 8.0 kg/m^3 for accelerating chloride-induced steel corrosion in concrete.

Table 4 Mix proportion of concrete

W/C	s/a	G_{max}	Unit mass (kg/m^3)					
(%)	(%)	(mm)	C	W	S	G	WRA*	AEA*
60	48	15	300	180	833	903	0.5	0.0095

*WRA: water reducing agent, AEA: AE agent

Seven kinds of repair materials shown in Table 5 were selected for this study. In Table 5, FA(J) is a normal mortar using Japanese fly ash type II specified in JIS A 6201 as the substitute of 15 % of cement, while FA(T) uses Taiwanese class F fly ash that is specified in American Society for Testing and Materials (ASTM). Fly ash quality as an admixture has been established as a JIS standard (see Table 6). Two classes of fly ash are defined by ASTM C618: Class F fly ash and Class C fly ash. The chief difference between these classes is the amount of calcium, silica, alumina, and iron content in the ash. The chemical properties of the fly ash are largely influenced by the chemical content of the coal burned. The Class F fly ash is Pozzolanic in nature and contains less than 7% lime (CaO). Possessing Pozzolanic properties, the glassy silica and alumina of Class F fly ash require a cementing agent, such as Portland cement, quicklime, or hydrated lime mixed with water to react and produce cementitious compounds. The properties of fly ash are shown in Table 7.

Taiwanese fly ash shows smaller density and specific surface area than those of Japanese fly ash, although it has a higher amount of CaO which contributes the activity of the chemical reaction.

P is a commercial PCM contains PAE powder polymer used as a patch repair material in Japan, while PFA(J) and PFA(T) are P-based mortars admixing Japanese and Taiwanese fly ash respectively as the substitute of 8.5 % of dry mortar.

Lithium nitrite is the lithium salt of the nitrous acid, with formula $LiNO_2$.Lithium nitrate ($LiNO_3$) undergo thermal decomposition yield the evolution of lithium nitrite, the reaction formula is as follows:

$$2LiNO_3 \rightarrow 2LiNO_2 + O_2 \text{ (at 500°C)} \quad (3\text{-}1)$$

This compound is hygroscopic and soluble in water. It is used as a corrosion inhibitor in the mortar. Unlike calcium nitrite inhibitors, lithium nitrite is particularly valued for corrosion inhibition and resistance of carbonation when an accelerated hardening process is not used and when a high concentration of 10% or more cement is added by weight. PFA(J)Li and PFA(T)Li are PFA-based mortars with the dosage of 40% $LiNO_2$ solution. The $LiNO_2$ solution was mixed as a part of the mixing water. The dosage amount of $LiNO_2$ was calculated on the assumption that 70 % of total $LiNO_2$ will diffuse to concrete and NO_2^-/Cl^- molar ratio becomes 1.5 if concrete contains Cl^- of 8 kg/m^3.

Table 5 Mix proportions of repair materials

Names	W/B (%)	P/C (%)	Unit mass (kg/m^3)						
			Dry mortar			W	Japan FA	Taiwan FA	40% $LiNO_2$
			C	P	S				
FA(J)	42	0	544	-	1120	269	96	-	-
FA(T)		0	544	-	1120	269	-	91	-
P		7	1775			238	-	-	-
PFA(J)		7	1624			256	92	-	-
PFA(T)		7	1624			256	-	88	-
PFA(J)Li		7	1624			192	92	-	64
PFA(T)Li		7	1624			192	-	88	64

Table 6 JIS standard for fly ash as an admixture for concrete

Type	Silicon dioxide (%)	Hygroscopic moisture (%)	Loss on ignition (%)	Density (g/cm^3)	Fineness		Flow value ratio (%)	Activity index (%)	
					Residue on 45 μm sieve (%)	Blaine's specific surface area (cm^2/g)		Material age: 28 days	Material age: 90 days
I	45.0 or more	1.0 or less	3.0	1.95 or more	10	5,000	105	90	100
II			5.0		40	2,500	95	80	90
III			8.0				85		
IV			5.0		70	1,500	75	90	70

Table 7 Properties of fly ash

Nation of FA	Density (g/cm^3)	Specific surface area (cm^2/g)	Glass content (%)	Ignition loss (%)	Chemical composition (%)						
					SiO_2	Al_2O_3	CaO	Fe_2O_3	Na_2O	K_2O	Na_2Oeq
Japan	2.33	3240	71.4	2.80	61.6	22.3	1.91	4.53	0.71	1.41	1.64
Taiwan	2.25	2530	75.3	2.82	60.7	20.7	2.71	5.70	1.18	1.24	2.00

Experimental data is collected to determine whether meets the specifications for patch repair material that are set by the Architectural Institute of Japan (AIJ 1997). According to AIJ (1997), Table 8 presents the required compressive and pull-out bonding strengths for patch repair material.

Table 8 Guideline of the patch repair material (AIJ 1997)

Test	Compressive strength	Flexural strength	Bonding strength
Required value	20 MPa	6.2 MPa	1 MPa

3.2 Preparation and curing of specimens

Specimens prepared for this study were mortar bars (40x40x160 mm), mortar cylinders (φ50x100 mm) of repair materials and joint cylinders (φ100x200 mm).

Joint specimens were made by bonding the concrete part and the repair material part simulating the joint area after applying patch repair method. The joint cylinders were prepared for investigating both the bond strength between concrete and repair mortar and Cl^- penetration around the joint face. The making process of the joint cylinders is shown in Fig. 6. As shown in this figure, the concrete cylinders were split after curing for 14 days in water at 20 ˚C and the repair material part was cast using the cylinder mold. The joint cylinders were cured for 28 days in water at 20 ˚C and subjected to the bond test and the Cl^- penetration test.

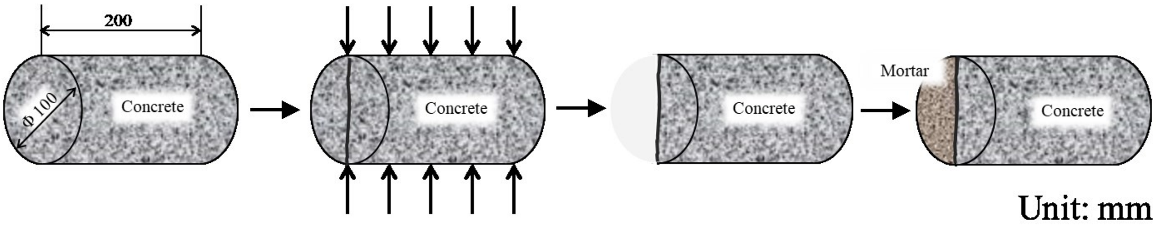

Fig. 6 Preparation process of a joint cylinder

The bonding way of concrete and repair material for joint specimens was upper-lower joint as shown in Fig. 7. The repair material part of the joint specimens was cast at the day after the casting of the concrete part. The joint face of concrete was roughened by removing the cement paste of concrete surface. The joint specimens after curing for 14 days in the moist condition at the temperature of 20 ˚C were stored in the constant humidity of 95% R.H. at 40˚C constantly.

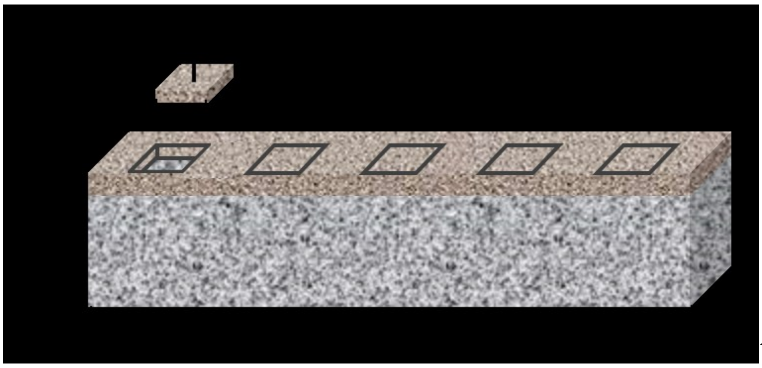

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Fig. 7 Outline of the joint specimen for bond strength test of repair materials

3.3 Tests

3.3.1 Mechanical properties tests

As the basic mechanical properties, compressive strength and flexural strength of repair materials were measured by using mortar bars at the age of 28 days following JIS R 5201. The tensile strength was measured by using mortar cylinders at the age of 28 days with the splitting test following JIS A 1113.

As the essential mechanical property for patch repair material, bond strength against the concrete substrate was measured at the age of 28 days. In this study, two methods were applied to obtain the bond strength. One is the method specified in JIS A 1171 to measure the bond strength of polymer cement mortar. The other method is an original one proposed in this study (Fig. 8) that uses the joint cylinders to split along the joint surface and bond strength can be calculated as the tensile strength measured in JIS A 1113. The proposed method doesn' t need a special device to pull off the repair material from the concrete surface and the loading condition is similar to the situation around the joint between concrete and patch repair material compared with the method of JIS A 1171.

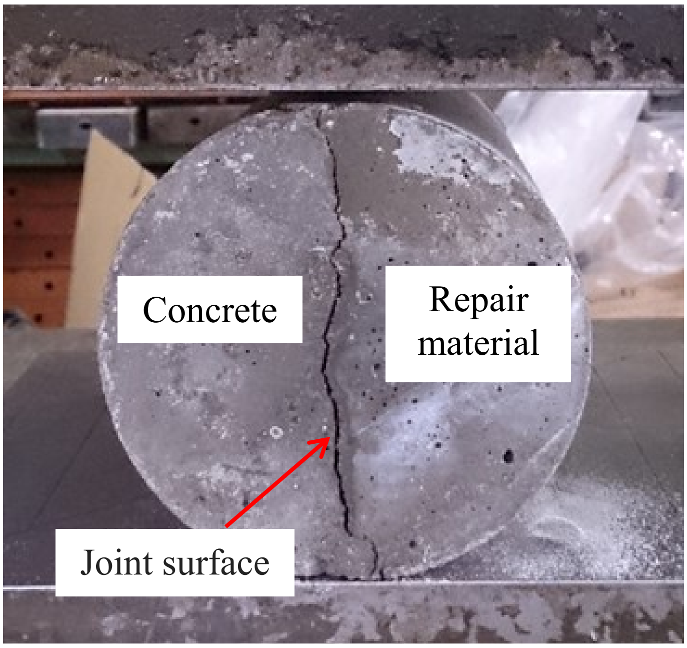


Fig. 8 Split loading for bond strength test using a joint specimen

3.3.2 Cl^- Penetration test

From the longitudinal center part of a joint cylinder, two discs (φ 100×50 mm) including the joint surface were cut out at the age of 28 days. They were allowed to dry in a laboratory condition for 24 h before they were epoxy coated leaving only one sawn surface free of coating, the joint disc was subjected to the Cl^- penetration test. The immersion test into 10% NaCl solution for 91 days was carried out following JSCE G 572 (see Fig.9). After completing the immersion, spraying the 0.1 N $AgNO_3$ solution on the split surface of the joint discs after the splitting at the vertical surface to the joint surface as shown in Fig. 10 (Otsuki et al., 1993). The white precipitate part was measured by a caliper at ten points as the Cl^- penetration depth from the exposed surface.

The white precipitate is silver chloride (AgCl) which is formed by the reaction formula as follows:

$$AgNO_3 + NaCl \rightarrow AgCl + NaNO_3 \quad (3\text{-}2)$$

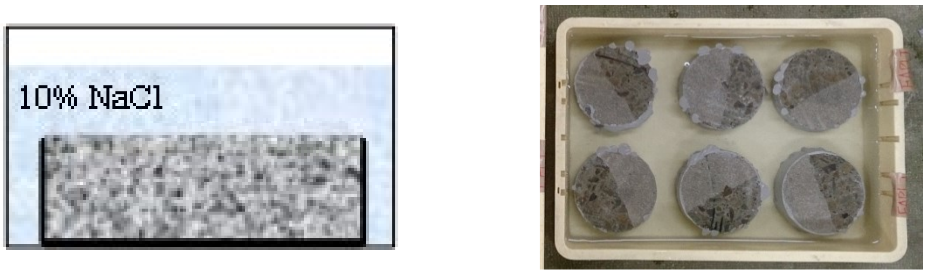


Fig. 9 Immersion of mortar in a 10% NaCl solution

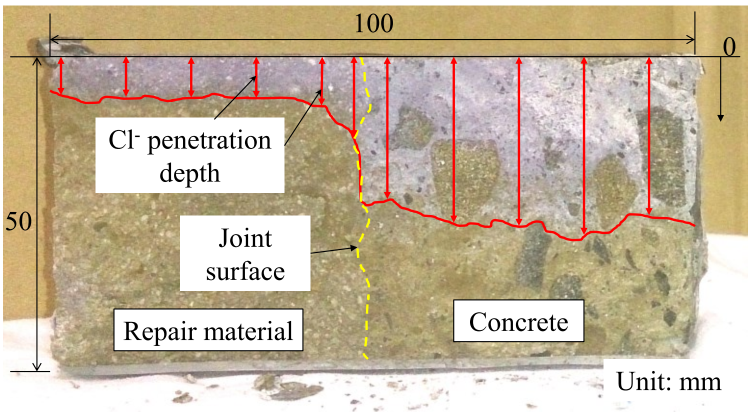


Fig. 10 The split surface of specimens after being sprayed 0.1 N $AgNO_3$ solution

The rapid chloride penetration test (RCPT) was the test proposed for rapid qualitative assessment of chloride permeability of concrete. The RCPT method has been used to investigate on the mineral admixture effect on the resistance of chloride ion penetration (Li and Roy 1986, Wee et al. 2000), the influence of aggregate fraction (Yang and Su 2002, Wee and Suryavanshi 1999), curing condition (Aldea et al. 2000), and pore size (Li and Roy 1986, Aldea et al. 2000) on the penetration of chloride ions. The test was the electrochemical Cl^- migration test following JSCE G 571 (JSCE, 2012). The lateral surface of discs was painted with epoxy resin in order to create one-dimensional chloride flow throughout the joint disc. The outline of this test is shown in Fig. 12. The joint disc was set at the mid part of the migration cell and a direct current constant voltage of 15V was supplied between the anode and the cathode cell. The cathode container contained 0.5 mol/L of NaCl solution and the anode container contained 0.3 mol/L of NaOH. During the test, the Cl- concentration in the anode cell was measured periodically until the Cl- flow became steady-state. Then, the effective diffusion coefficient of Cl^- was calculated by using the following equation (Andrade, 1993).

The diffusion coefficient is the parameter which may characterize a concrete in order to predict its long-term performance, that is, its resistance to the penetration of ions. The calculation of D from electrical measurements has to be based in the fundamental of transport processes in electrolytes, very well established in the traditional books of Electrochemistry Science (Bard et al. 1982, Bockris 1974, Costa 1981, Forker 1986, Glasstone 1947, Newman 1991, Southampton Electrochemistry Group 1990). The solution for transport processes is Nernst-Planck (Bockris 1974) equation.

Nernst-Planck equation: $$J_i = -\frac{1}{kT} D_i(x) A(x) \rho_i(x) \frac{d}{dx}[\mu_i(x) + z_i e V(x)] \qquad (3\text{-}3)$$

Where J_i is particle flux, $D_i(x)$ is diffusion coefficient profile, $A(x)$ is cross section, $\mu_i(x)$ is chemical potential profile, $V(x)$ is potential profile, z_i is ionic valence, T is temperature, k is Boltzmann constant, e is elementary charge. Flux includes phenomena such as diffusion, migration, and convection. However, some considerations have to be made before an appropriate application of equation (3-3) is tried.

(1) Only what happens inside the concrete disc is influencing the measurements.

(2) The dealing with convection in equation (3-3) can be neglected if only what happens inside the concrete disc is considered.

(3) The diffusion component of equation (3-3) is considered negligible in comparison to that due to migration.

(4) The concrete disc is thin enough to allow to reach a steady- state condition is few hours.

(5) The concentration of chlorides in one chamber of the cell is much higher than in the other.

Once all these assumptions are considered the equation can be expressed in the following way:

$$J_{ct} = \frac{V^{II}}{A}\frac{\Delta_{c_{ct}}^{II}}{\Delta t} \tag{3-4}$$

$$D_e = \frac{J_{ct}RTL}{|Z_{ct}|FC_{ct}(\Delta E - \Delta E_c)} \times 100 \tag{3-5}$$

Where J_{ct} is the Cl$^-$ flux at the steady-state (mol/(cm^2 · year)), D_e is the effective diffusion coefficient (cm^2/year), V^{II} is the volume of the solution in the anode cell (L), A is the cross-sectional area of the specimen (cm^2), $\Delta_{c_{ct}}^{II}/\Delta t$ is the rate of Cl- concentration change in the anode cell ((mol/L)/year), R is the universal gas constant (8.31J/(mol · K)), T is the absolute temperature (K), Z_{ct} is the electrical charge of chloride ($= -1$), F is the Faraday constant (96,500C/mol), C_{ct} is the Cl- concentration in the cathode cell (mol/L), $\Delta E - \Delta E_c$ is the electrical potential difference between specimen surfaces (V), and L is the thickness of the specimen (mm).

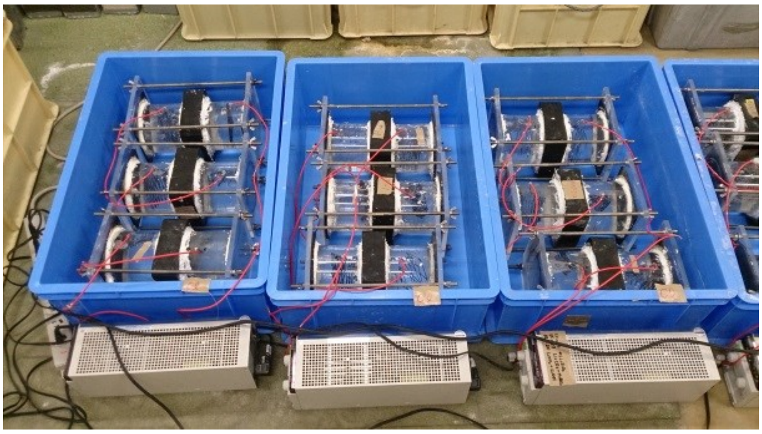


Fig. 11 Test arrangement of electrochemical migration test

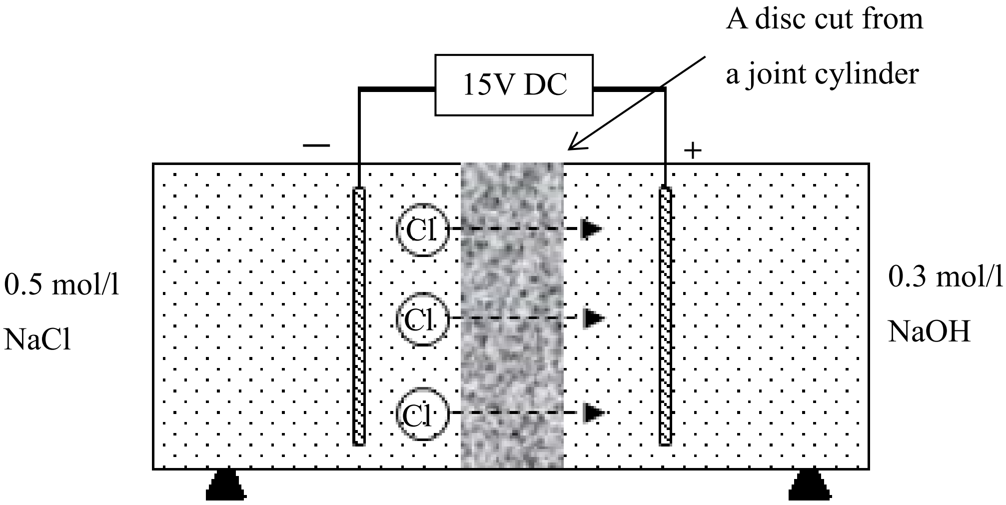


Fig. 12 A schematic of electrochemical migration cell

3.3.3 Electrochemical tests for evaluating steel corrosion in RC joint specimens

Electrochemical measurement items for RC joint specimens were the polarization resistance of the steel bar and the electrical resistivity. As a reference electrode and a counter electrode for these electrochemical monitoring, saturated silver chloride (Ag/AgCl) and titanium mesh were used respectively. These electrodes and the RC joint specimen were electrically connected through absorbent cotton containing tap water. Polarization resistance was measured by the rectangular wave electric current polarization method, as the deference of impedances at 800 Hz and 0.1 Hz of electric current frequency. Electrical resistivity was obtained as the impedance at 800 Hz of electric current frequency. In addition, a distribution of macro-cell current density in the RC joint specimen with the divided steel bar was measured by the electric meter.

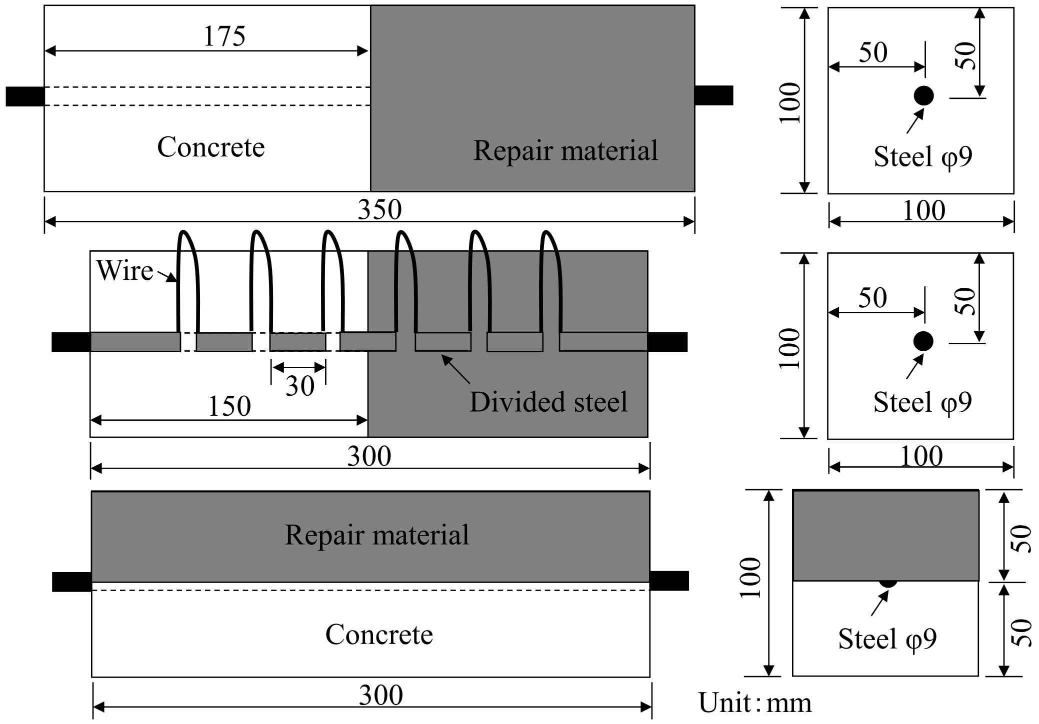


Fig. 13 Outline of a joint RC specimen

3.4 Results and discussion

3.4.1 Flowability and Basic Strength of Repair Materials

Flow values of repair materials measured by the method following JIS R 5201 are shown in Fig. 14. From this figure, although the mortar flow of PCM "P" shows relatively small value, the fly-ash-mixing increases mortar flow values of PCM. Ball-bearing effect of fly ash may promote to increase the mortar flow. For the specimens with the fly ash of Taiwan and fly ash of Japan, the result shows fly ash of Taiwan reduces the flow values. As an example, the PFA(T) in the specimens yields the flow value of approximately 158 mm, which is less than that (around 161 mm) at PFA(J). Moreover, $LiNO_2$ must be dissolved in water during the mixing procedure. The dosage of the $LiNO_2$ solution decreases the flow value because of the reduction of the actual mixing water replaced by the $LiNO_2$.

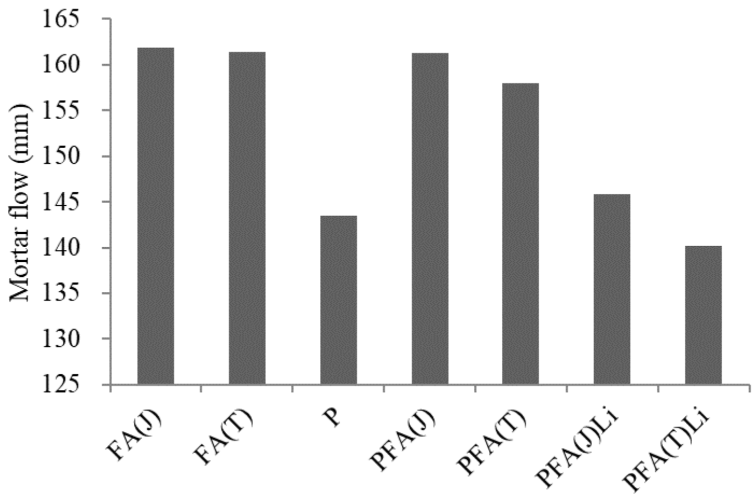


Fig. 14 Mortar flow values of repair materials

The compressive strength of repair materials is shown in Fig. 15. According to Fig. 15, the compressive strength of the PCM specimens with fly-ash-mixing is lower than P or FA. The Pozzolanic reaction proceeds, continuously consuming Ca $(OH)_2$. The resulting colloidal C-S-H and C-A-H filled repair material pores, making the repair materials denser. Reaction formula would be as follows:

$$Ca(OH)_2 + SiO_2 + H_2O \rightarrow CaO \cdot SiO_2 \cdot H_2O \quad (3\text{-}6)$$

$$Ca(OH)_2 + Al_2O_3 + H_2O \rightarrow CaO \cdot Al_2O_3 \cdot H_2O \quad (3\text{-}7)$$

From the chemical properties of fly ash in Table 7, the SiO_2 and Al_2O_3 proportion of fly ash in Japan are both higher than Taiwan's fly ash which could explain the higher compressive strength of (J) specimens than (T) specimens. Fly-ash-mixing reduces the compressive strength of specimens at the early age.

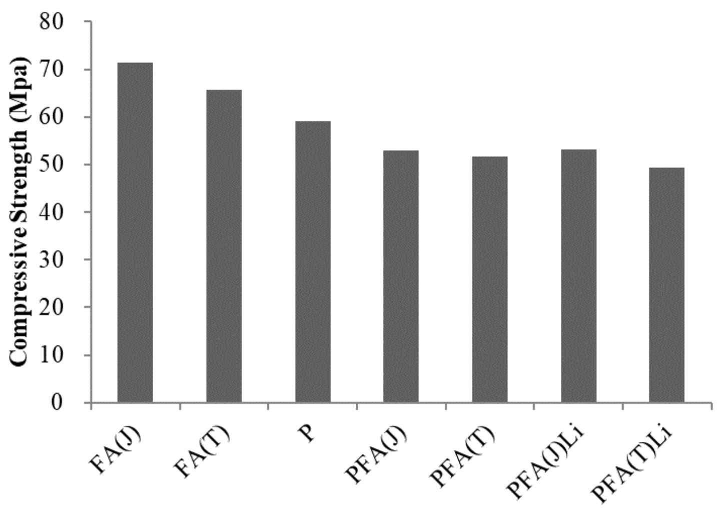


Fig. 15 Compressive strength of repair materials at 28 days age

Fig. 16 shows the tensile strength of the specimens. This value is not a performance metric for the patch repair material that is specified by the AIJ (1997). The fly ash does not contribute to tensile strength, and tensile strength must be supported by polymer materials (PAE). Wang proposed in the literature (Wang and Wang 2008), the PAE latex decreases the bulk density and the compressive strength of cement mortar markedly, but only slightly decreases the flexural strength, and thus improves the toughness significantly; it also increases the tensile bond strength. In addition, this study observed the tensile strength of the specimens with PFALi that are cured in 28 days increases when the mixing $LiNO_2$.

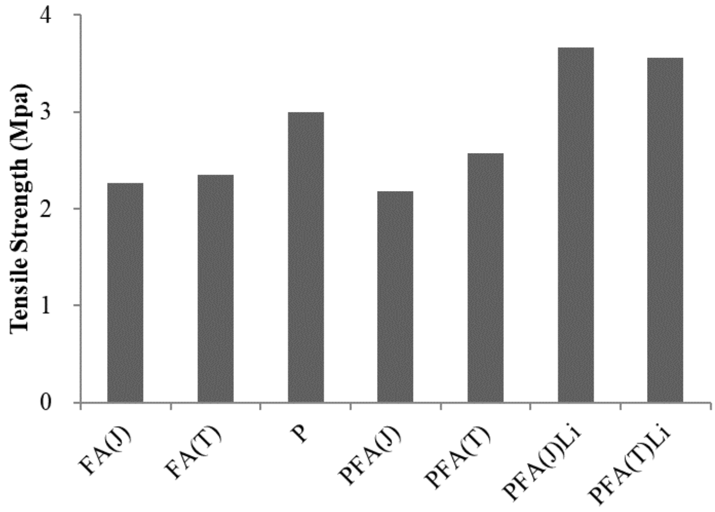


Fig. 16 Tensile strength of repair materials at 28 days age

For the flexural strength (Fig. 17), fly ash has little effect on flexural strength; although fly ash and $LiNO_2$ reduce the flexural strength of the specimens, the flexural strength of the specimens that are cured for 28 days exceeds with the requirement of 6.0 MPa. From these figures, the fly-ash-mixing decreases the compressive and flexural strength a little. However, PFA keeps quite high and enough strength to apply for the actual repair cases. The influence of the difference of fly ash type on the mechanical strength is not remarkable in this study.

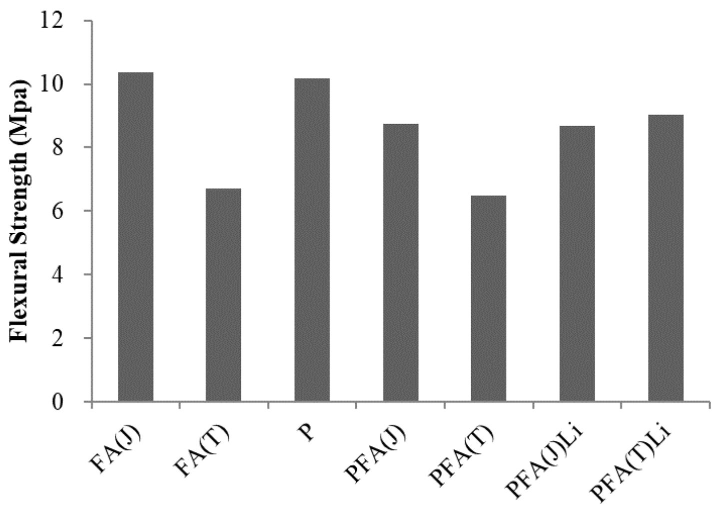


Fig. 17 Flexural strength of repair materials at 28 days age

The (Tensile strength/Compressive strength) percentages of repair materials are shown in Fig. 18. According to the result, the dosage of $LiNO_2$ increases all kinds of strength and especially, it increases (Tensile strength/Compressive strength) percentage. Past paper reported that the dosage of $LiNO_2$ could densify the microstructure of mortar (Hori et al. 1991). In this study, the hydration of cement in PCM may be affected by the $LiNO_2$ and the dense microstructure would contribute to realizing the higher strength of PFALi.

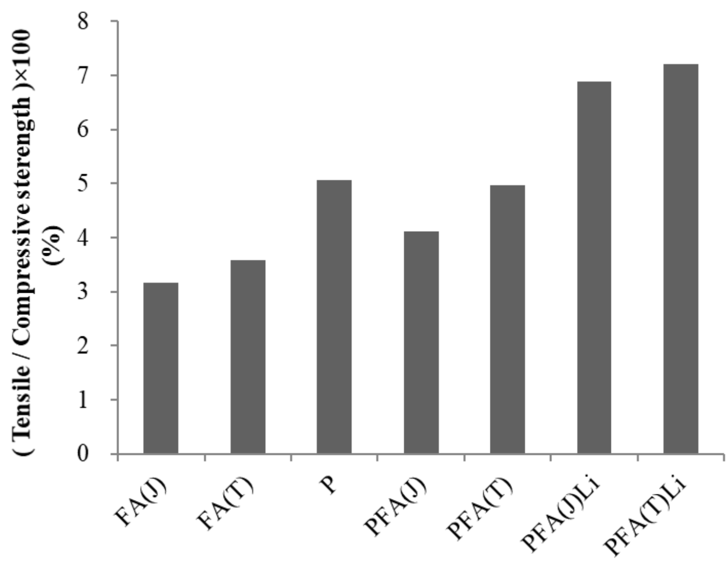


Fig. 18 (Tensile strength/ Compressive strength) percentage of repair materials

3.4.2 Bond Strength

The bond strength is a very important indicator of repair materials. Increasing the bond strength of the repair mortar to the concrete can effectively avoid problems such as shedding, shrinkage, and cracking of the repair material. The bond strength data measured by the different two methods are shown in Fig. 19-21. In this study, the tendency of the data obtained by these two methods is generally agreed. The bond strength data measured by the pull off method tend to vary widely because it’s difficult to control the failure surface same. On the other hand, the split method could easily control the failure surface and all of the joint specimens in this study showed the failure at the interface surface. Then, the proposed split test seems to be an effective method to measure the bond strength of patch repair materials.

The results of two bond strength test in 28 and 91days age are shown in Fig. 19

and Fig. 20. In 28 days age of curing time, the bond strength values of PFA cases are lower than the value of P and those values of PFALi cases are higher than the value of P. Such tendency is in accordance with the results of the compressive strength or the flexural strength as shown before. However, the bond strength remains almost constant as the curing period increases. According to Fig. 21, since the bond strength data of all repair materials in this study exceed 1.0 N/mm^2 which is the standard level specified in some Japanese codes, these repair materials could be judged applicable from the viewpoint of the bond strength.

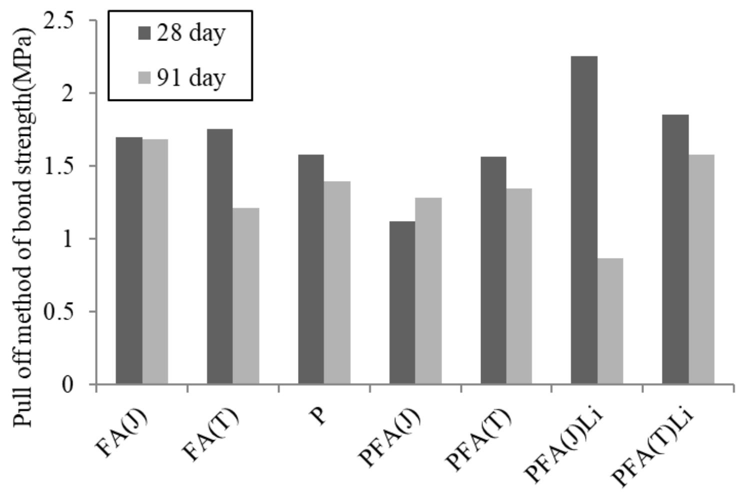


Fig. 19 Pull off method of bond strength

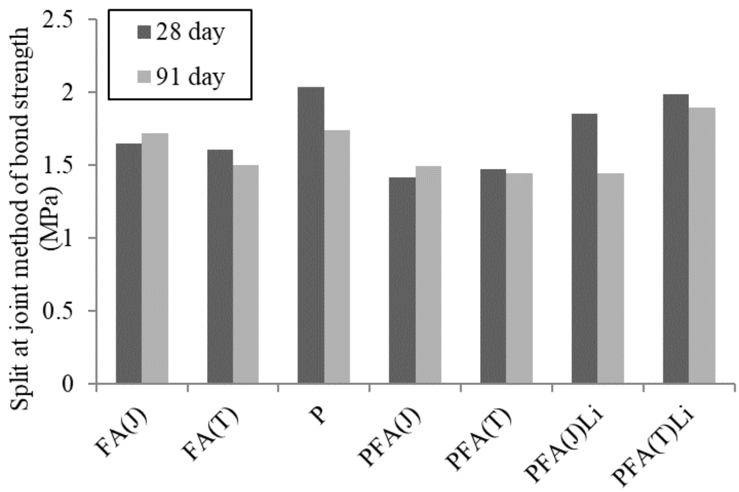


Fig. 20 Split at joint method of bond strength

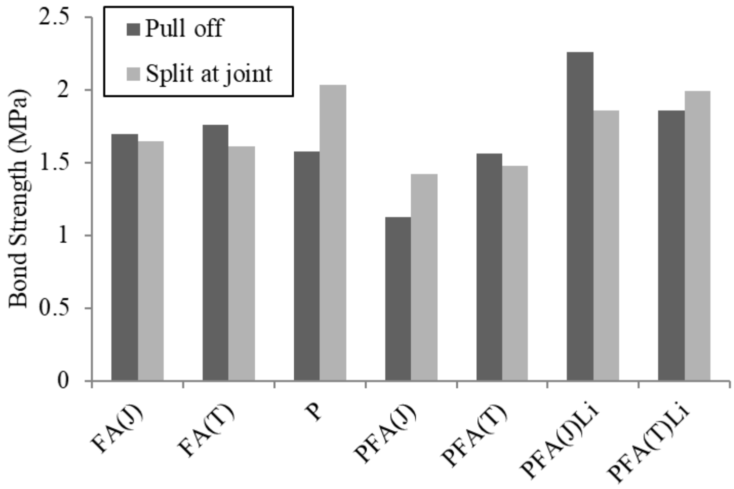


Fig. 21 Bond strength of repair materials

In another case, the mix proportion of repair materials shown in Table 9 (Juan 2017). In the research, there are three control variables, which are fly ash replacement ratio, W/B ratio, and S/W ratio. Fly ash replacement ratio is mass of fly ash by mass of total cementitious materials (cement and fly ash). The factor S/W ratio herein is defined as the ratio of the mass of polymer solid by the total water content of materials on the basis of the reference (Diab 2013). The results of bond strength in 28 days age are shown in Fig. 22. The bonding strength results of pull off method and split method would be discussed, the thickness of repair material in pull off method is much thinner than the specimen of the split method. It means that the pulling off method may be influenced by environmental factors lead to drying shrinkage, such as the case is shown in Fig. 23.

Table 9 Mix proportions of repair materials

Names	W/B (%)	Fly ash replacement ratio	S/W (%)	Unit mass (kg/m^3)					
				C	S	W	P	FA	$LiNO_2$
F00SP00	50	0	0	504	1512	252	0	0	0
F25SP00		25%	0	451	1352	282	0	113	0
F25SP05			5	443	1330	277	14	111	0
F25SP05-Li			5	443	1330	208	14	111	69

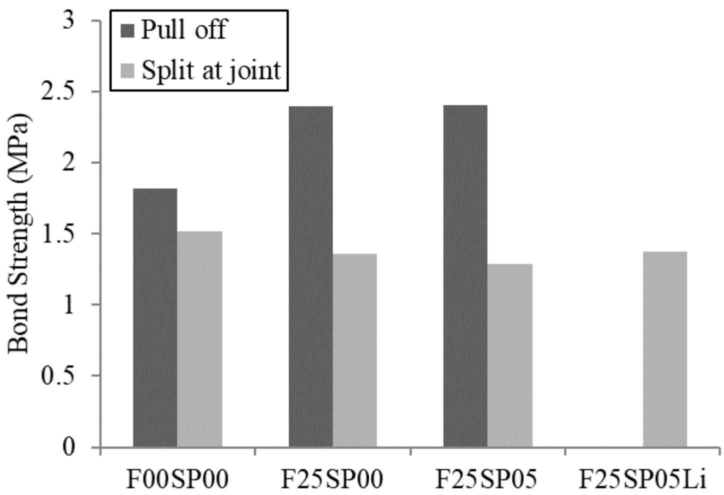


Fig. 22 The results of bond strength

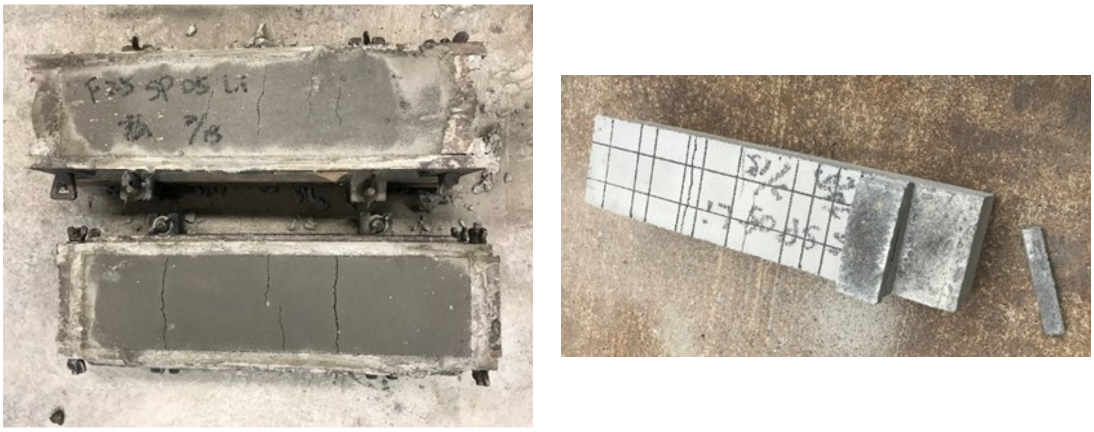


Fig. 23 The bond strength specimen before the practice of pull off method

Based on the above experimental results, the bond strength data measured by the different two methods are shown in Fig. 24. Wherein, the pull off method and the split method to measure bonding strength they are the proportional relationship to each other. According to size effect, the bigger interface would show lower strength. Therefore, if the results of splitting method still satisfy the standard of JIS which means the bond strength of the repair material is no problem.

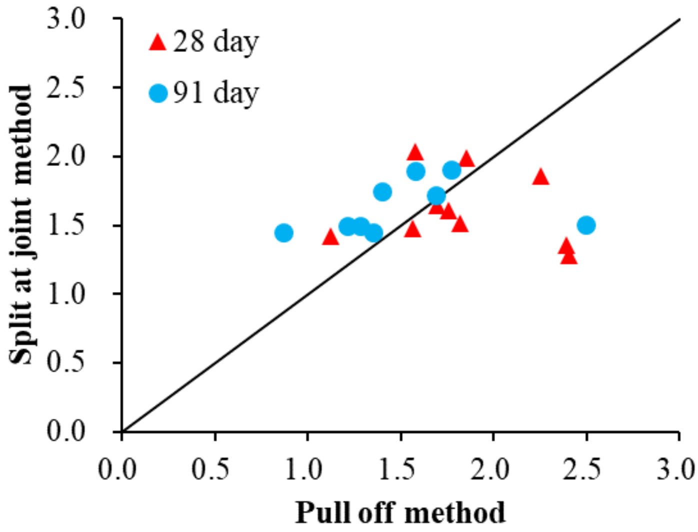


Fig. 24 The bond strength at the age of 28 and 91 days

Chapter 4 Cl^- ions Around The Joint Part

4.1 Critical content of chloride ion in concrete against steel corrosion

Reinforced concrete (RC) building is one of the most durable architectural forms. The concrete is seriously threatened in terms of durability in coastal areas, which is one of the most corrosive natural environments in the world. Sodium chloride (NaCl) is a main component of seawater. The diffusion of chloride ion into the RC elements is one of the major problems being faced by the concrete structure. Therefore this subject has been the focus of many researchers in order to solve the problem or decrease the vulnerability of structures. When chloride ions enter into concrete, the chemical processes leading to corrosion commences. It was discovered found from studies that Pozzolanic materials reduce the ingress of chloride by improving the microstructure condition and chloride binding behavior (Batis et al. 2005, Dhir and Jones 1999, Hewlett et al. 1999). Kampala et al. (Kampala et al. 2013) studied the durability of the calcium carbide and fly ash stabilized silty clay against wetting and drying cycles to ascertain its performance in pavement applications. Durability stands amongst the mechanical properties of the fly ash stabilized with lime and can be defined as the capacity of the material to retain its integrity when exposed to destructive weathering conditions (Dempsey and Thompson 1967).

The content of chloride ion in concrete is an important factor for whether or not the steel bar has rust. There is no uniform standard for chloride ion content in various countries, as shown in Table 10.

Table 10 Standard of chloride ion content in various countries

Country	Specification	Specified value of chloride ion content
United Kingdom	BS 8110-85Part1	Calcium chloride as a percentage of cement weight ($CaCl_2$). 1. General RC: 0.4% (0.768 kg/m^3) 2. Sulfate-resistant cement: 0.2% 3. PC (high temperature curing RC): 0.1% 4. Admixture: 2% of admixture or 0.03% of cement
France	DTU 21.4	1. Without steel of concrete and protective layer 4 cm thick of RC, cement weight: 2% ($CaCl_2$) (3.838 kg/m^3) 2. The protective layer 2cm of RC, where 1% of cement weight ($CaCl_2$) 3. The amount of chlorine salt in the mixing water for RC is 0.25g/l% (Cl^-) (0.413kg/m^3)
Germany	DIN	Soluble chloride ion as a percentage of pellets 1. the pre-tensioning pre-stressed concrete PC; PC filler (DIN 4227):0.02% (Cl^-) 2. RC (DIN 1045), post-tensioned pre-stressed concrete PC (DIN 4227):0.04% (Cl^-) (0.72 kg/m^3)
United States	ACI 301-72	Calcium chloride(CaCl2) accounts for less than 2% of cement weight (3.383 kg/m3)
	ACI 318-83	Soluble chloride ion as a percentage of cement weight (Cl^-) 1. PC: 0.06% 2. Salt damage environment: 0.15% (0.45 kg/m3) 3. RC (normal environment): 0.30% (equivalent to 0.9 kg/m3, cement per cubic meter, 0.9 kg Cl^-) 4. RC (dry state): 1.00%
	ACI 222R-85	Chloride ion as a percentage of cement weight (Cl^-) 1. PC: 0.08% 2. RC: 0.20% (0.6 kg/m^3)
Japan	AIJ JASS 5-1991	The chloride ion content of concrete is limited to 0.3 kg/m^3 or less. If it is necessary to exceed it, even if there is a countermeasure against deterioration, the chloride ion content must not exceed 0.6 kg/m^3.
	JSCE	Concrete contains chloride ions 1. General RC, post-tensioned pre-stressed concrete PC: 0.6 kg/m^3 2. special consideration for durability of RC, or after consideration of salt corrosion conditions of pre-tensioned concrete PC: 0.3 kg/m^3
Taiwan	CNS-3090	Maximum water-soluble chloride ion content in concrete (according to the water-soluble method) 1. Pre-tensioned concrete: 0.15 kg / m^3 2. Reinforced concrete: 0.3 kg / m^3

4.2 Immersion into salt water

The distributions of Cl^- penetration depth around the joint surface measured by the spraying $AgNO_3$ solution after the immersion into the salt water for 91 days are shown in Fig. 25 and Fig. 26. The distributions of Cl^- measured by the EPMA analysis are shown in Fig. 27.

$AgNO_3$ spraying allows easy and fast measurement of chloride penetration depth. Judging from the change in color, the parts that have suffered Cl^- erosion appear white, and the rest is brown as the picture show Fig.25. From Fig. 26, there is a gap in the penetration depth of Cl^- at the joint surface of the specimen. The penetration depth of Cl^- in the concrete part is deeper than that in the repair material part. These results show that the repair materials have the higher resistance performance against the Cl^- penetration than that of concrete. The joint surface is not the deepest penetration point for all cases in Fig. 26, which means that the bond between concrete and repair materials works enough to resist the Cl^- penetration.

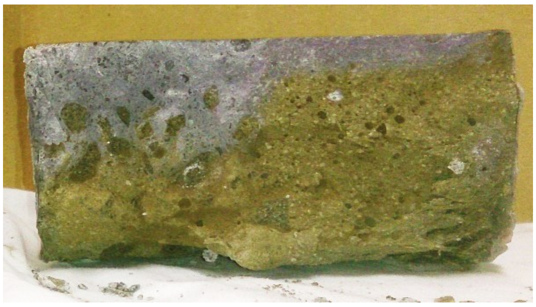
Comparing repair materials each other, normal fly ash mortar "FA(J)" and "FA(T)" show the deeper Cl^- penetration than the cases of PCM. In Fig. 26, the difference of Cl^- penetration between the case of the general PCM "P" and the cases of the fly ash mixed PCM "PFA" or the cases of "PFALi" containing $LiNO_2$ is minor. So, all of these PCM-based repair materials have the higher resistance performance against Cl^- penetration than the cases of fly ash mortar "FA". On the other hand, Cl^- penetration depths of concrete jointed by PCM-based repair materials, especially "PFALi", are larger than the cases of FA(J). The possible reasons are those that the concentrated Cl^- at the surface of PCM-based repair materials promoted the Cl^- penetration into concrete or the joint surface between concrete and PCM-based repair materials had very minor defects to promote the Cl^- penetration into concrete.

(a) FA(J)

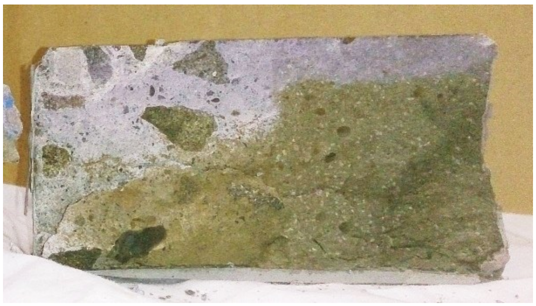
(b) FA(T)

(c) P

(d) PFA(J)



(e) PFA(T)



(f) PFA(J)Li

(g) PFA(T)Li

Fig. 25 Distribution of spraying $AgNO_3$ solution around the joint surface

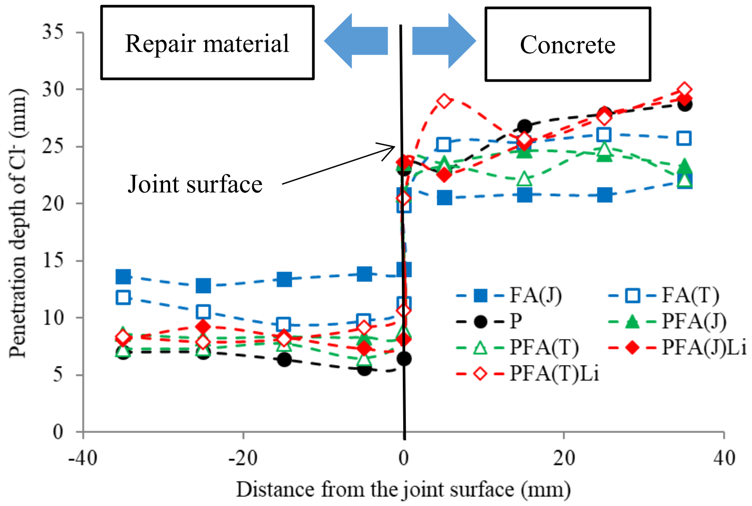
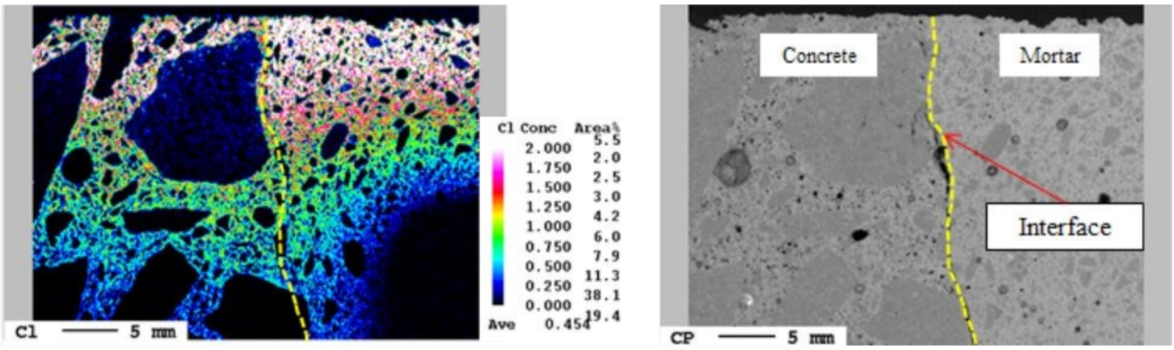


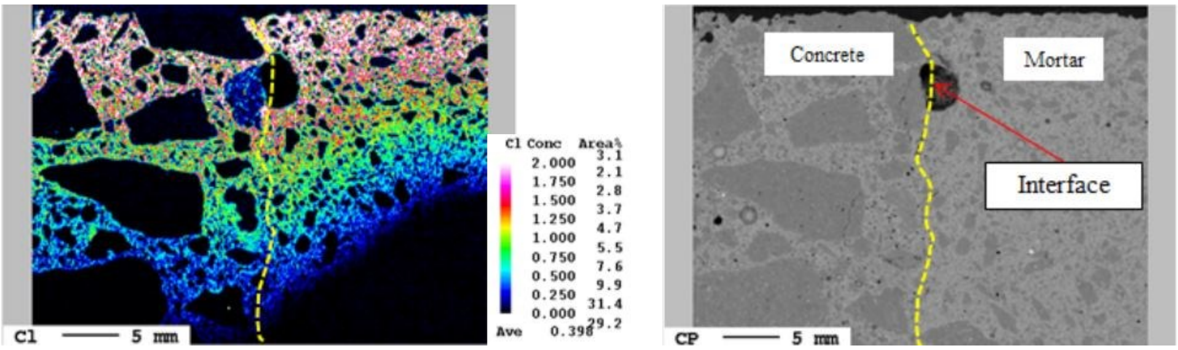
Fig. 26 Distribution of Cl^- penetration depth around the joint surface

Electron Probe Micro Analyzer (EPMA) is an electron microscope equipped with an analytical instrument that irradiates the surface of a solid sample with an electron beam narrowed down in a vacuum, observes the structure and morphology of its surface and local element analysis on the micron order. When an electron beam is irradiated on a sample at a constant accelerating voltage, incident electrons interact with the sample to generate various signals. Secondary electrons generated by electron beam irradiation, reflected electrons, characteristic X-rays, Auger electrons, absorption current, cathode luminescence and the like. Among these signals, EPMA performs elemental analysis by measuring characteristic X-rays. Since the energy of characteristic X-rays is a value unique to each element, element analysis can be performed by measuring this energy.

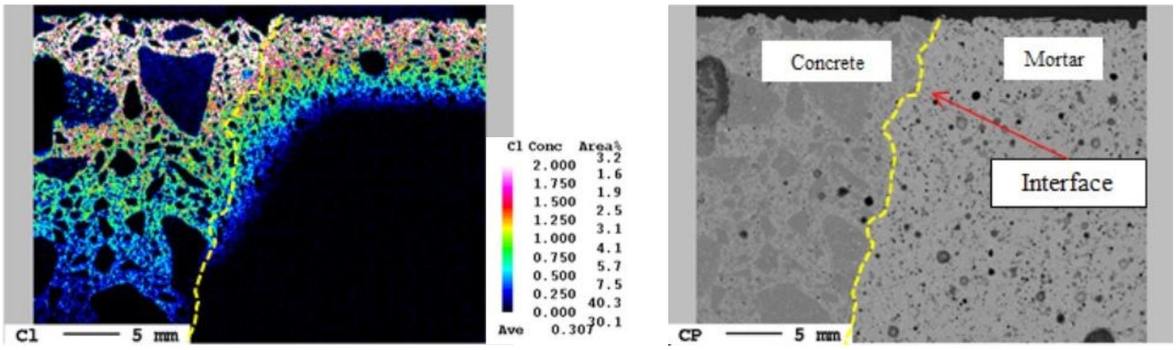
In the Fig. 27, it can be seen that Cl^- ions migrate not only to the deeper part in concrete but also through the joint surface into the repair materials. The PFALi-concrete joint specimens show the higher Cl^- content in concrete than the other cases. Such tendency agrees with that shown in Fig. 26.



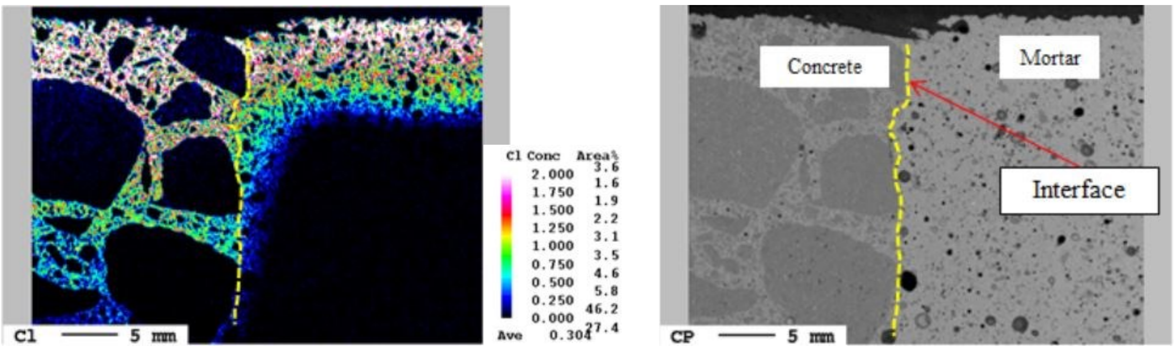
(a) FA(J)



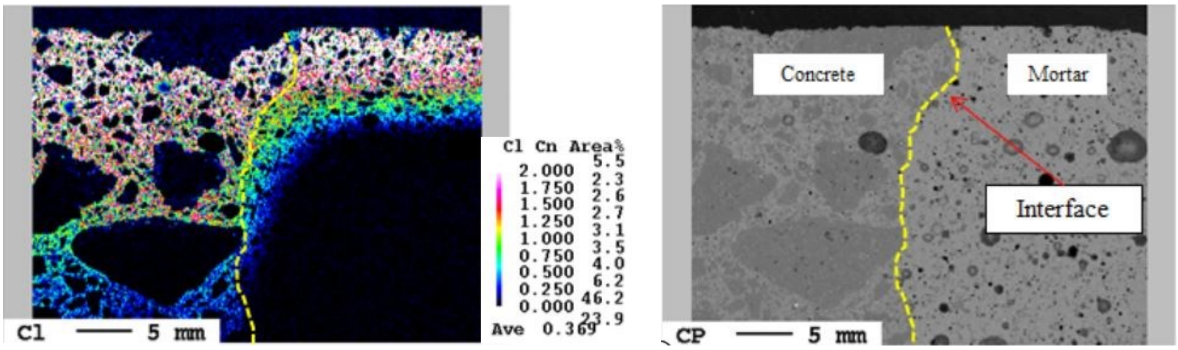
(b) FA(T)



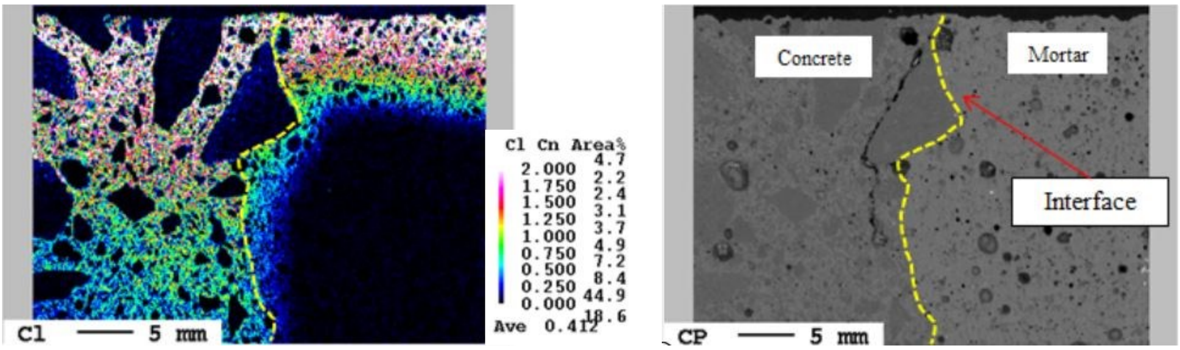
(c) P



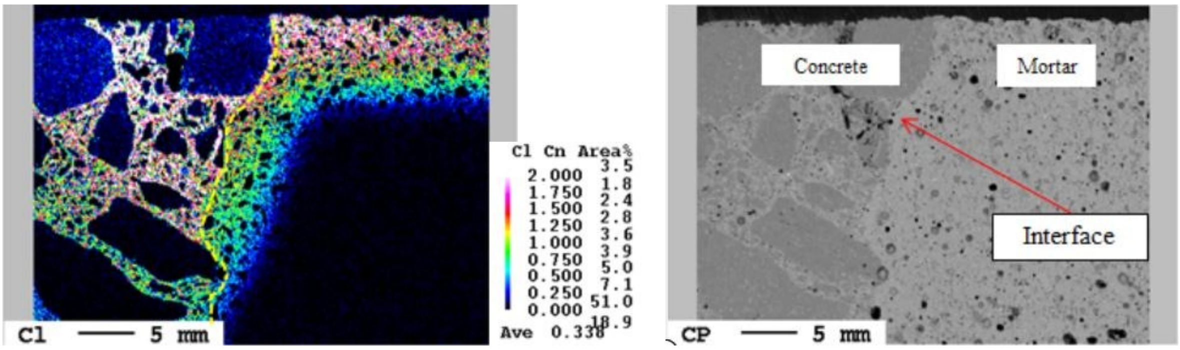
(d) PFA(J)



(e) PFA(T)



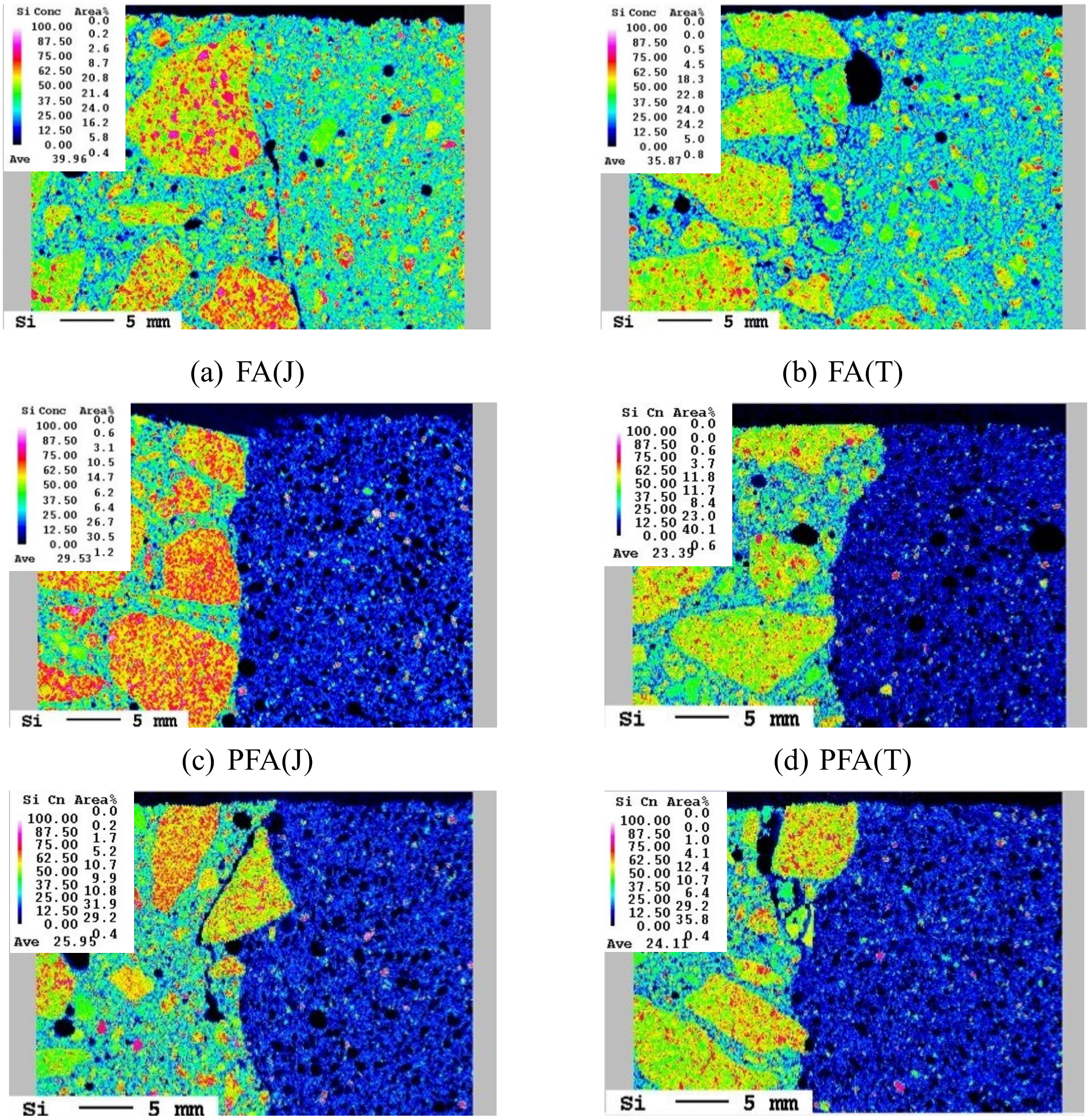
(f) PFA(J)Li



(g) PFA(T)Li

Fig. 27 Cl^- distribution around the joint surface measured by EPMA analysis

Besides that using the results of EPMA to determine the Pozzolanic activity. According to the "J" and "T" specimens to measure the content of Si, respectively. The "J" specimens show the higher Si content in mortars than the "T" cases. This means that the quality and the Pozzolanic activity of Japan's fly ash are better. Such situation agrees with that compressive strength.



(a) FA(J) (b) FA(T)

(c) PFA(J) (d) PFA(T)

(e) PFA(J)Li (f) PFA(T)Li

Fig. 28 Si amount around the joint surface measured by EPMA analysis

4.3 Electrochemical migration of Cl^-

Variation data of Cl^- concentration in anode cell with time during the electrochemical Cl^- migration treatment are shown in Fig. 29 and the calculated effective diffusion coefficients of Cl^- are shown in Fig. 30. From these figures, FA(J) shows the least Cl^- penetration and the smallest effective diffusion coefficient of Cl^-, while PFA(T)Li shows the highest Cl^- penetration and the largest effective diffusion coefficient of Cl^-. In Fig. 29 or Fig. 30, Cl^- penetration depth in the repair material

"FA" is larger than that of PCM-based repair materials but the effective diffusion coefficient of Cl^- for FA-concrete joint specimen was the smaller than the cases of PCM-concrete joint specimens. It may be because of the acceleration of Cl^- penetration in the concrete part of the PCM-concrete joint specimens.

From the electrochemical viewpoint, the migration cell consists of the parallel circuit of the concrete part and the repair material part. When the constant voltage of 15 V is supplied to this circuit, theoretically, the electric current flowing in the concrete part should be constant regardless of the kinds of the coupled repair materials, while the electric current flowing in the repair materials should be smaller than that of the concrete part due to the higher electrical resistance. Then, the total electric current flowing in this parallel circuit should be inversely proportional to the electrical resistance of the repair materials. However, in Fig. 29 and Fig. 30, the electrochemical migration of Cl^- thorough the PCM-concrete joint specimens are accelerated more than the case of FA mortar-concrete joint specimens. So, the Cl^- penetration in concrete part would be accelerated by the coupled repair materials which have higher electrical resistance like PCM. Such tendency agrees with the results of the immersion test into salt water as shown in Fig. 26 or Fig. 27.

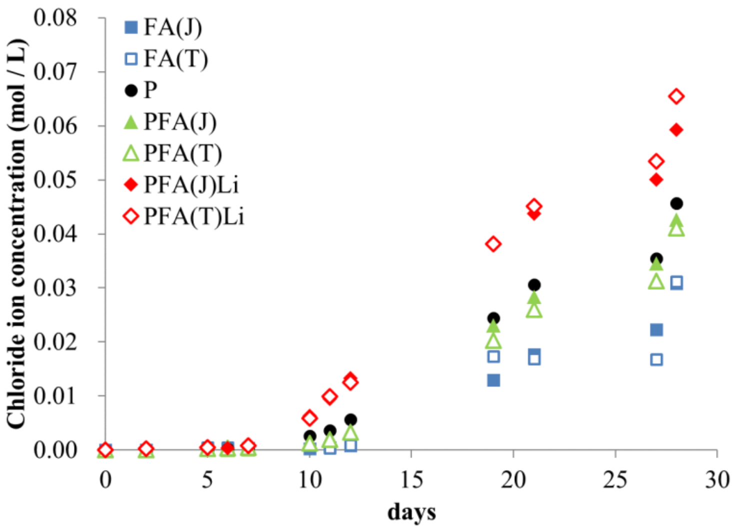


Fig. 29 Evolution of Cl^- concentration in the anode cell

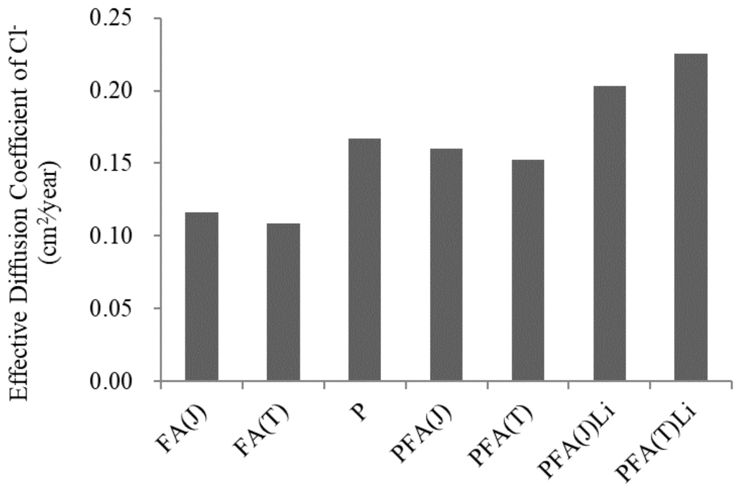


Fig. 30 The effective diffusion coefficient of Cl^-

According to JSCE and public works research institute (PWRI), the apparent diffusion coefficient of Cl^- must be lower than 3.14 cm^2/year and 1.35 cm^2/year. Although all cases of PCM based repair material accelerates the Cl^- penetration into concrete around the joint surface, each diffusion coefficient is lower than the specified. Based on this judgment, the materials recommended in this study are applicable as repair materials.

4.4 Electrochemical Evaluation of Protection Effect against Chloride-induced Steel Corrosion

The protection effect against chloride-induced steel corrosion due to patch repair was evaluated by using RC joint specimens. In these cases, the concrete contained the pre-mixed Cl^- of 8 kg/m^3 to accelerate the steel corrosion. Variation curves of polarization resistance of steel and electrical resistivity of repair materials in RC upper-lower joint specimens are shown in Fig. 31 and Fig. 32 respectively. It is known that the inverse of the polarization resistance is proportion to the corrosion rate of steel. From these figures, PFA and PFALi show larger polarization resistance and concrete resistivity than the cases of P or FA. It can be said that the admixing of fly ash with PCM forms a dense pore structure and suppresses the corrosion rate of steel at the interface of concrete and repair material in the upper-lower RC joint specimens.

Especially, PFALi shows the largest polarization resistance which could be gained by the dosage of $LiNO_2$ having the suppression effect of steel corrosion. Moreover, the electrical resistivity values of FA, PFA and PFALi increase with time, which would be caused by the Pozzolanic reaction of fly ash contributing to improve the long-term strength and durability of the repair materials.

Distribution curves of polarization resistance of steel in the RC right-left joint specimens are shown in Fig. 33. According to this figure, the polarization resistance data of steel in repair materials are larger than those in concrete containing premixed Cl^-. Especially, PFA(J)Li, PFA(T)Li and PFA(J) show larger values than the other cases. In the cases of PFA(J)Li and PFA(T)Li, the polarization resistance will increase in the concrete part near the joint interface with the diffusion of $LiNO_2$ from the repair materials in the future. Regarding the fly ash type, PFA(T) using Taiwanese fly ash shows smaller values than the case of PFA(J) using Type II fly ash although FA(T) shows larger values than the case of FA(J). Since the surface area of Japanese fly ash is larger than that of Taiwanese fly ash, Japanese fly ash could contribute to densify the PCM but the chemical reaction activity in the cement mortar would be relatively low due to the lower content of CaO which is closely related to the Pozzolanic reaction of fly ash in cement mortar.

Distribution curves of macro-cell corrosion current of divided steel in the RC right-left joint specimens are shown in Fig. 34. From this figure, a plus current is detected in the side of concrete while a minus current is detected in the side of repair material, which indicates the macro-cell corrosion current between the anode in the concrete side and the cathode in the repair material side could be measured by using the divided steel bar. The largest macro-cell current is measured in the case of P, which means that the patch repair using a general PCM could cause the macro-cell corrosion if much amount of Cl^- remains in the concrete. On the other hand, Fig. 34 shows that the fly ash mixing into PCM contributes to suppress the macro-cell corrosion and the combination with $LiNO_2$ magnifies the suppression effect.

As a result of this study, the patch repair using PCM-based repair materials like PFALi could accelerate the penetration of Cl^- around the joint part even though the chloride-induced steel corrosion including the macro-cell corrosion could be suppressed due to the fly ash mixing and the dosage of $LiNO_2$ into PCM. Then, the long term repair effect and the steel corrosion after the patch repair should be investigated to conclude the effective repair material.

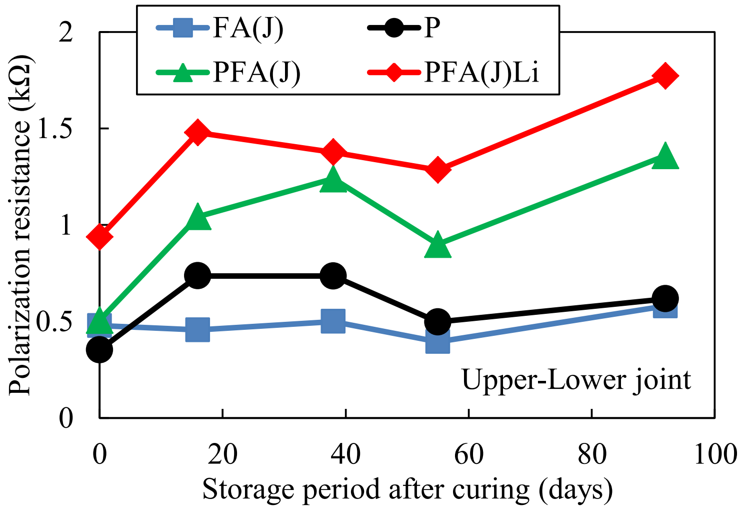


Fig. 31 Variation curves of polarization resistance of steel in RC upper-lower joint specimens

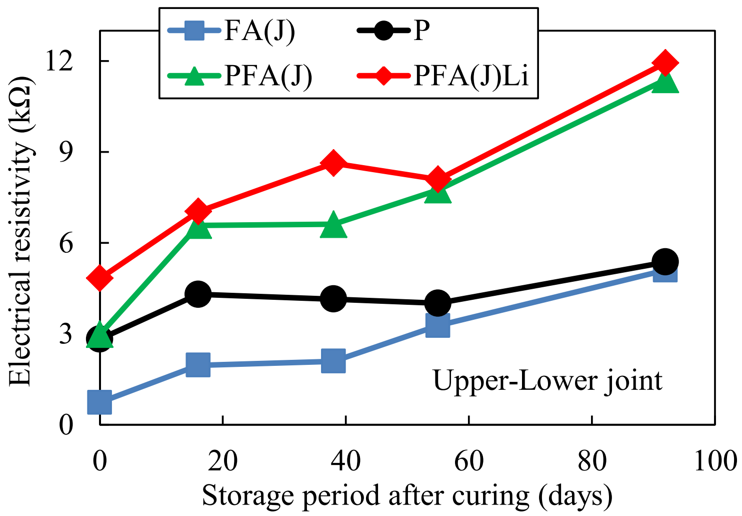


Fig. 32 Variation curves of electrical resistivity of repair materials in RC upper-lower joint specimens

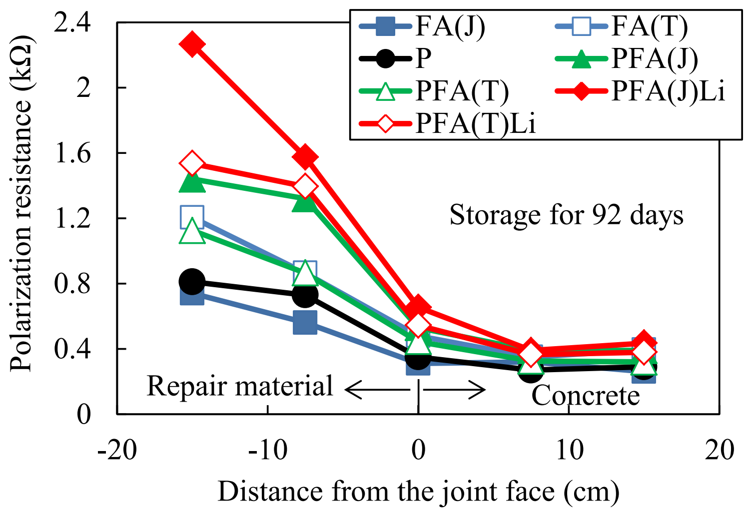


Fig. 33 Distribution curves of polarization resistance of steel in RC right-left joint specimens.

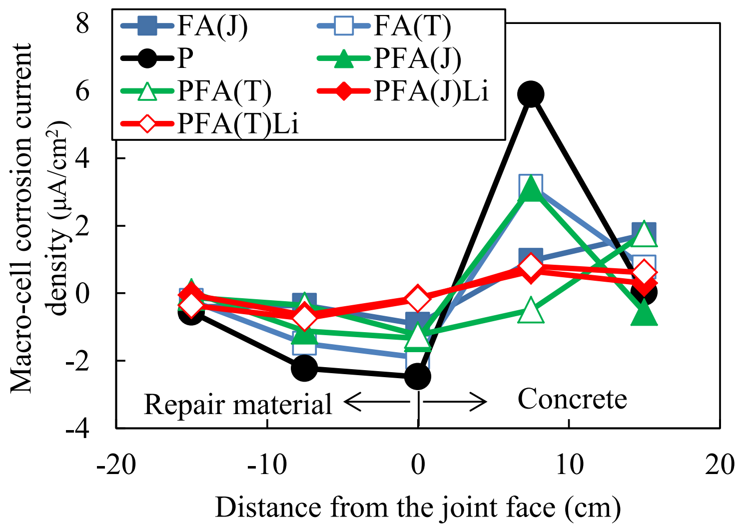


Fig. 34 Distribution curves of macro-cell corrosion current of divided steel in RC right-left joint specimens.

Chapter 5 Conclusions and Suggestions

5.1 Conclusions

The results of this study can be summarized as follows:

(1) Fly ash increases the workability of PCM; however, $LiNO_2$ greatly reduces the flow value to an extent. Restated, admixing of fly ash enhanced the flowability of repair materials.

(2) The compressive, flexural and tensile strength of repair materials decreased with the mixing fly ash although all repair materials showed enough strength level for the actual application.

(3) The bond strength data measured by the proposed method showed generally similar tendency with the data obtained by the conventional pull-off test. The bond strength of PCM-based repair material decreased a little with mixing fly ash but increased with a dosage of $LiNO_2$.

(4) Cl- penetration into the PCM-based repair materials was slower than the cases of normal mortar containing fly ash. Cl^- penetration into concrete around the joint face was accelerated by jointing with PCM-based materials compared with the cases of jointing with the normal mortar. Such tendency was observed in both saltwater immersion test and the electrochemical Cl^- migration test. Although all cases of PCM based repair material accelerates the Cl^- penetration into concrete around the Joint surface, each diffusion coefficient is lower than 1.35 cm^2/year of the specified.

(5) The PCM containing both fly ash and $LiNO_2$ showed a high protection effect against steel corrosion in the RC joint specimens and the macro-cell current around the joint face was also suppressed compared with the case of a general PCM.

(6) As well as the mechanical strength, workability and chloride resistance are considered in this study. For most RC buildings with an age of over 30 years in Japan and Taiwan, which have the average testing compressive strength of approximately 20 MPa, the repairing mortar of this study is recommended for patch repair method.

5.2 Suggestions

According to the “Recommendations for Environmentally Conscious Practice of Reinforced Concrete Buildings” (AIJ 2008) issued by the Japan Institute of Architects in September 2008. For the RC buildings, four types of environmental considerations, including resource-based, energy-saving, substances with reduced environmental load, and long-lived, have been proposed according to the Lifecycle. At the various stages of the life cycle (design phase, construction phase, use and maintenance phase, etc.), reference should be made to various types of issues that should be met, and the relevant personnel (designers, construction supervisors, etc.) must possess the thinking and knowledge. In the case of RC buildings, to extend their life cycle or useful life, their durability can be increased to meet the requirements of long-life buildings.

During the use of the building, with the increase of the use time, due to the heat of hydration and dry shrinkage causes the concrete to crack and add to the influence of the external environment. For example, chloride ion and carbon dioxide, these will gradually corrode the steel bars. Once the steel bars corrode, the corrosion products will cause the steel bars to expand in volume. The pressure on the concrete caused by the expansion will cause the surrounding concrete to begin to produce larger cracks. When the crack propagates to the surface of the component and causes more corrosion of the steel bar, the corrosion of the steel bar due to the gradual reduction in the sectional area leads to a decrease in the durability of the concrete structure. At this time, chloride ions will have a diffusion effect due to the different crack widths and the size of the pores in the concrete, leading to spalling of the concrete protective layer, resulting in a substantial decline in structural performance and affecting its durability.

The use of building structure inspection data to evaluate its possible residual life is an important research in the maintenance of building structure management systems. The contents of building structure inspections generally include concrete compressive strength, carbonization depth, corrosion current, chlorine ions concentration, corrosion rate, protective layer thickness, corrosion test data, location and width of crack. Use these tests to evaluate the durability of building structures.

So far, the repair mortar proposed in this study is suitable for use as a repair material. The mechanical properties are in line with the AIJ regulations and have good resistance to chloride ion. On the other hand, The PCM containing both fly ash and $LiNO_2$ showed a high protection effect against steel corrosion in the RC joint specimens and the macro-cell current around the joint face was also suppressed compared with the case of a general PCM. Afterward, tests related to weather resistance and carbonation resistance can also be conducted. This will have a full understanding of the repair mortar.

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