Static and Fatigue Bond Characteristics of Interfaces between CFRP Sheets and Frost Damage Experienced Concrete

by J.G. Dai, Y. Saito, T. Ueda, and Y. Sato

Synopsis: Both short and long-term performances of repaired or strengthened concrete structures using external FRP bonding are greatly affected by states of bonding substrates, which are covercrete and may have experienced various damages. One of them is frost damage in cold regions. This paper intends to investigate how the initial frost damages in concrete influence the static and fatigue bond performances of CFRP/concrete interfaces. Concrete specimens were exposed to freeze and thaw cycles before being bonded with CFRP sheets. The initial frost damage of concrete was controlled approximately at three different levels in terms of its relative dynamic modulus of elasticity, which was 100% (non frost damage), 85% and 70%, respectively. Test results showed that failure modes of CFRP/concrete bonded joints with initial frost damage in concrete were the delamination of covercrete. By contrast the joints without initial frost damage failed in a thin concrete layer as usual. Moreover, CFRP/concrete joints with and without initial frost damage showed different manners in their interface bonding strength and stiffness. If the initial frost damage existed in concrete substrate the effective bond length of CFRP/concrete joints was increased due to the decrease of the bonding stiffness and interfacial fracture energy. Fatigue testing results indicated that the linear slopes of *S-N* curves of CFRP/concrete bonded joints were not influenced by the initial frost damage. The initial frost damage did not shorten the fatigue life of CFRP/concrete joints if a same relative tensile stress level was kept in the FRP sheets, where the relative tensile stress level was defined as a ratio of the applied tensile force in FRP sheets for the fatigue tests to the maximum static pullout one achieved in each test series.

Keywords: concrete; external bonding; fatigue; frost damage; FRP; relative dynamic elastic modulus

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INTRODUCTION

Reliable interface bond between FRP and concrete substrate is of critical importance for external strengthening technology using FRP. Up to date extensive studies have been carried out to clarify the interfacial bond mechanisms as well as to formulate design concepts which can prevent the strengthened members from various premature bond failures.[1-4] These extensive studies were mainly meant for verifying short-term performance of upgraded FRP/concrete composite systems in terms of their strength, stiffness and ductility. Comparatively, there are increasing but still limited studies related to long-term durability of the interface bond, which is affected by various adverse environmental factors such as freeze-thaw (FT) cycling, moisture ingress, fatigue loading and in particular their combinations. [5-9]

In the past, bond properties of FRP/concrete interfaces were usually studied based on newly cast concrete substrate. However, the reality is that concrete substrate before the installation of FRP has experienced various damages under different prevailing environmental conditions, among which is FT cycling in cold regions. Frost damage of concrete results in a decrease of concrete strength, eventual disintegration and complete loss of material near the surface.[10] However, sole frost damage seldom leads to spalling of the whole concrete cover, which can tell that the cover should be replaced before that FRP is bonded. Therefore, it is necessary for us to investigate how the initial frost damage existing in concrete substrate affects the bonding strength, stiffness and durability of FRP/concrete interfaces, or on the other hand, to evaluate what the threshold value of the initial frost damage should be for the purpose of achieving reliable external bonding. It has been reported that FT cycling does not affect adversely the performance of CFRP strengthened concrete beams or CFRP/concrete bonded joints.[5] There also had other reports showing that both the flexural capacity and deformability of FRP strengthened RC beams after exposures to a certain number of FT cycles are decreased at failure.[11, 12] In all of these reports, however, FT cycling acted on the whole FRP/concrete composite system rather than concrete substrate directly. It is considerable that installation of FRP can protect the concrete cover from environmental attacks. As a result it is difficult for us to see directly the effects of frost damage of concrete on the interface bond. In order to get convincing durability design data related to FT cycling a helpful way is to study the bond between FRP and concrete that has experienced frost deterioration.[13]

Fatigue durability of FRP/concrete interfaces is another important issue. It was reported that fatigue life of RC members is increased after FRP strengthening. The fatigue failures of strengthened members are usually governed by the fatigue fracture of steel reinforcement rather than overall debonding of FRP from the concrete substrate.[14, 15] However, Cyclic loading tests on FRP/concrete bonded joints showed that the interface debonding propagates progressively with the increase of fatigue cycles.[6, 9] An FRP/concrete interface with sufficient anchorage length, may keep its interface bonding capacity after expected fatigue cycles. Nevertheless, the progressive local interface deficiency affects adversely the serviceability of FRP strengthened members and may trigger a durability failure with a combination of other environmental attacks. It was also found that fatigue life of concrete under compression is shortened if the concrete has experienced frost damage.[16] This implies a necessity of studying the fatigue performance of FRP/concrete bonded joints where concrete substrate has initial frost damage.

RESEARCH SIGNIFICANCE

The effects of initial internal damage of concrete on the bond performances of FRP sheet/concrete joints were rarely observed. The objectives of this paper are to clarify the following two things: (1) How the initial frost damage in concrete affects the anchorage properties of CFRP/concrete bonded joints; (2) How the initial frost damage affects the fatigue failure of CFRP/concrete bonded joints.

EXPERIMENTAL PROGRAM

Test setup

A test system, which was firstly developed by Kobayashi et al.[17] for evaluating the fatigue bond characteristics of CFRP/concrete interfaces, was applied in this study. As shown in Fig. 1, each specimen prepared for this test system included two concrete blocks, which were externally bonded with a common CFRP sheet and were fixed on two separated H-shape steel beams, respectively. Two H-steel beams were connected with a hinge in the middle. Static or cyclic vertical force acting on the two H-shape steel beams through a top loading frame could rotate two concrete blocks and resulted in a direct tension in CFRP and a direct shear at the CFRP/concrete interface while avoiding eccentric loading effects. More details on the dimension of the test system can be found in Reference[17]. In this study, the pullout force in CFRP was calculated from the exerted vertical force. Strain gages were mounted on CFRP with an interval of 20mm to monitor the interface debonding process and to record the local bond stress and slip information.

Materials and specimen details

Concrete used in all specimens was early-strength type composed of water, cement, sand and gravel with a mass ratio of 0.58:1:3.2:4.3. In order to reach the desired levels of frost damage rapidly, no AE agent was used. The mechanical properties of CFRP sheets and epoxy resin used in this study can be seen in Table.1.

The geometry of the testing specimens is shown in Fig. 2. The concrete blocks had the dimension of 100×100×400mm, and CFRP attached to each concrete block consisted of two layers of CFRP sheets. The bonding area between each concrete block and CFRP was 150×60mm. A 40mm un-bonded area was set between FRP and concrete block near the loaded end of CFRP. Four steel bars with diameters of 10mm were set in each concrete block to prevent it from possible bending failure during the operations after the frost damage, and also, to simulate approximately the state of concrete cover in practical engineering. Experimental parameters were the level of frost damage in concrete and the maximum stress level in FRP sheets for fatigue tests. Arrangement of all specimens with different test variables is summarized in Table 2. In total, 38 concrete blocks were cast. So that 19 CFRP/concrete bonded joints included in four series S1, S2, S3, and S4 (see Table 2) were prepared. Concrete was cast in two different times. S1 and S2 series belonged to the same batch. The left S3 and S4 belonged to another batch.

Process of introducing frost damage into concrete

Before CFRP sheets were bonded to concrete, concrete blocks were exposed to FT cycles up to the desired levels of frost damage, which were evaluated by measuring concrete's relative dynamic elastic modulus (RDEM) according to ASTM method.[18] The FT cycles were conducted in an environmental chamber. The temperature variations in one cycle are shown in Fig. 3. Three target levels of frost damage in this study corresponded approximately to the RDEM of 100% (non-frost damage), 85% and 70%, respectively. The authors did not intend to introduce more serious frost deterioration into concrete because it was supposed that the covercrete should be replaced before external FRP bonding in case that very severe frost damage can be observed by naked eyes in actual engineering. From this study, it seemed that concrete block even with the RDEM of about 70% was not in that situation.

Figure 4 shows the relationship between the number of FT cycles and the RDEM of concrete. Concrete in S1 and S2 series decreased their RDEM from 100% to 70% just after 24 FT cycles. But concrete in S3 and S4 series decreased their RDEM from 100% to 68% after 75 FT cycles. Before the frost deterioration, Series S1, S2 and S3, S4 had the same mixing proportion and their tested compressive strength was 34.8 and 36.1MPa, respectively. It is not clear why two batches of specimens showed so different deterioration manners. Since they were cast in different time, a possibility is that unexpected variations in vibrating or other operations led to the difference in the frost resistance of two batches of specimens. It seems that the loss of concrete strength can be related more to the loss of RDEM of concrete rather than the number of FT cycles. So the RDEM instead of the number of FT cycles is used in this study as a quantitative index to evaluate frost damage. Concrete blocks after achieving target frost damage had conventional surface treatment process (sand blasting) and then were externally bonded with CFRP sheets.

TEST RESUTLS AND DISCUSSION

Effects of frost damage on failure modes

Failure modes of CFRP/concrete bonded joints under static and fatigue loading are listed in Table 2. Two different failure modes were observed: peeling of FRP from concrete substrate (see Fig. 5 left) and concrete cover delamination (see Fig. 5 right).

For all non frost-damaged specimens (S1 series in Table 2), both static and fatigue failures were due to the peeling of CFRP from concrete substrate, in which just a very thin concrete layer (several millimeters) was attached with the peeled CFRP as reported popularly in the past studies. By contrast, all frost damage-experienced specimens failed due to the delamination of covercrete, in which a very thick concrete layer (more than 15 mm) was attached to the CFRP. Only specimen S4-0-1 was an exception. All five specimens in S4 series were supposed to have the uniform initial frost damage as they experienced same history of FT cycling in a same environmental chamber. S4-0-1 had much higher test bond strength than the others in S4 series (see Table 2), implying that S4-0-1 might have a higher frost resistance compared to the other specimens in this series. However, the reason for the higher frost resistance could not be found. It is considerable that two factors may lead to the delamination of covercrete. One is that frost damage caused change of tensile and compressive strength of concrete; and another is that steel reinforcement existed near the covercrete and made concrete at that location relatively weak after FT exposure. As a result, cracks induced by stress concentration were easier to penetrate deep into the concrete blocks once they formulated in concrete substrate. Then the cracks found an easier way to propagate along the location where steel reinforcement existed. In engineering practice, particularly for the strengthening of frost damage experienced slab system, this kind of delamination of covercrete is rather possible to occur under fatigue loading processes and results in a durability failure since it leads to an exposure of steel reinforcement.

Effects of frost damage on bond force-transferring capacity of CFRP/concrete joints

Frost damage leads to a change of concrete strength and deformability.[16] As a consequence, it affects the bond force capacity transferred from concrete to externally bonded CFRP sheets. As shown in Fig. 6, when the RDEM of concrete decreases from 100% to 70% in series 1 and series 2 specimens, testing compressive strength of concrete decreases about one-third (from 34.8MPa to 23.4 MPa). Similarly, when the RDEM decreases from 100% to 68% in series 3 and 4 specimens, tested compressive strength of concrete also decreases about one-third (from 36.1MPa to 25.7MPa). Comparatively, in the case that the RDEM decreases to 85%, there is almost no change of compressive strength in concrete. Figure 7 indicates the experimental relationship between the compressive strength of concrete and the bond force-transferring capacity of CFRP/concrete joints with different RDEM of concrete. The predicted relationship between the compressive strength and the bond force transfer capacity is also given in Fig 7 based on the following formulation proposed by the authors[19]:

$$P_{\max} = (b_f + 2\Delta b_f)\sqrt{2E_f t_f G_f} \tag{1}$$

where b_f(mm) is the width of FRP sheets; Δb_f is an additional bond width taking into account the influences of FRP sheets' bond width on the bond strength since the bond strength is usually high in case of using narrow bond width and vice versa. Δb_f can be taken as 3.7mm based on experiments[19]; $E_f t_f$ (kN/mm) is the tension stiffness of FRP sheets, and G_f (N/mm)is the interface fracture energy, and can be taken as $0.524 f_c'^{0.236}$ (N/mm) in case of using with most commonly used commercial adhesives.

Basically, the bond force-transferring capacity of CFRP/concrete joints tends to decrease with the compressive strength and the RDEM of concrete. The accuracy of perdition seems acceptable at an average level although the corresponding bond transfer capacity to a similar RDEM shows rather big scatter in the experiments especially when the RDEM has the lowest value. Two series of specimens with similar RDEM (70% and 68% respectively) had almost the same compressive strength (23.4 and 25.8MPa respectively) but showed rather big difference in the bond force- transferring capacity. The reason can be attributed to different history of FT cycling for Series S2 and S4 specimens. Series 4 specimens experienced more FT cycles than Series 2 specimens although both series had similar compressive strength of concrete. It was reported that the compressive and tensile strength of concrete have different deterioration manners with the increase of FT cycles.[20] The tensile strength of concrete firstly shows a stable deterioration but followed by a sudden one with the increase of FT cycles. Comparatively the deterioration of compressive strength is quite stable. Therefore, Series S4 and S2 specimens had similar compressive strength after different FRP cycling. But Series S4 specimens possibly had lower tensile strength than Series 2 specimens, leading to a comparatively low bond force-transferring capacity of FRP/concrete interfaces because the tensile strength of concrete is more critical for the bond. The relationship between the compressive and tensile strength of concrete is changed depending on the history of FT cycling. This factor implies that present approaches for predicting the bond force-transferring capacity of FRP/concrete interfaces using the compressive strength of concrete as a parameter may lead to an unsafe prediction if there is noticeable initial frost damage in concrete. On the other hand, a reasonable level of damage like a threshold value of the RDEM of concrete is needed for guaranteeing a reliable external bonding. Current study shows that CFRP/concrete bonded joints with a moderate frost damage (RD = 85%) in the concrete have neither decrease nor big variation of the bond force-transferring capacity compared with non frost-damaged joints. A further decrease of RDEM to 68%, however, leads to a substantial loss of the interfacial bond force-transferring capacity.

Effects of the initial frost damage on bonding stiffness and effective bond length

Attentions should be paid not only to the anchorage strength but also to the anchorage length of CFRP/concrete bonded joints when the initial frost damage of concrete becomes a concern. Once concrete has experienced frost damage, plastic strain and stiffness degradation of concrete consequently affect the bonding stiffness of CFRP/concrete joints. Figure 8 shows observed load-slip relationships of all CFRP/concrete bonded joints at the loaded end under static loading. It can be seen that interface bonding stiffness decreases with increase of the initial frost damage of concrete. For FRP/concrete bonded joints, the authors proposed a following effective bond length model[21], which can explain well the relationship among the interface bonding strength, interface bonding stiffness and the effective bond length:

$$L_e = \frac{\sqrt{2E_f t_f}}{B\sqrt{G_f}} \ln(\frac{1+\alpha}{1-\alpha}) \quad (2)$$

where L_e is the effective bond length; $E_f t_f$ is the tension stiffness of FRP sheets; G_f is the interfacial fracture energy related to the maximum pullout force as shown in Eq.1; α is taken as 0.96 for anchorage design; and B is a parameter related to the stiffness of load-slip curves of FRP/concrete joints at loaded end in a pullout test by using the following expression[19]:

$$P = f(s) = P_{\max}(1 - \exp(-Bs)) \quad (3)$$

where P is pull out force at loaded end; s is the corresponding interface slip at loaded end; and P_{max} is the maximum pullout force as indicated in Eq.1.

It is not difficult to know from Eq. 3 that lower stiffness of a load-slip curve corresponds with in a smaller value of B. Small B leads to an increase of effective bond length according to the model shown in Eq. 2. In addition, it can be seen in Eq.2 that the decrease of interfacial fracture energy G_f, the square root of which is proportional to the bond force capacity P_{max}, also results in an increase of effective bond length. Discussion in the last section shows that a decrease in concrete's RDEM to 68% led to a substantial decrease of bond force-transferring capacity, which also causes the increase of the effective bond length. Therefore, degradation of CFRP/concrete joints in both stiffness and strength due to the frost damage should be reflected appropriately in a reliable anchorage design if concrete is deteriorated by frost attacks before external FRP bonding.

Failure processes of CFRP/concrete bonded joints under fatigue loading

For the static failures of CFRP/concrete bonded joints, local interface debonding is initiated with increase of tensile force in CFRP sheets. Once the pullout force in CFRP sheets reaches a critical value, macro-debonding occurs and leads to an overall failure of the whole joint. However, for the fatigue failures of CFRP/concrete bonded joints, the local interface debonding can be caused by gradual increase of the local slip even the tensile stress in CFRP sheets is kept at a certain value that is smaller than the maximum pullout force. The local interface debonding can propagate toward the free end with the increase of fatigue cycles. Once the maximum bond force that the left bonded length can bear becomes less than the tensile force given in FRP sheets, a fatigue failure of the whole bonded joint will occur. Figure 9 shows a typical bond stress – slip relationship of CFRP/concrete joints, which was observed at a same location but under different loading cycles. It can be seen that the local slip increases gradually with the fatigue cycles. Accompanied with increase of the local slip, the local bond stress reaches peak value first and then degrades gradually. After a certain number of fatigue cycles, the local bond stress became negligibly small, implying the occurrence of local debonding. Nature of the fatigue local bond stress- slip of CFRP/concrete joints is the same as that observed under static loading. The only difference is in that the softening speed of local bond stress-slip curves changes with the tensile stress level in FRP sheets. Figs.10.a and 10.b show this difference by comparing the average fatigue bond stress-slip relationships under two different levels of pullout force, which are 73% and 51% of the maximum pullout capacity, respectively. The other testing variables except the level of pullout force are same. Average bond stress-slip curves are used here for the convenience of comparison since it is well known that local bond stress-slip curves of FRP/concrete joints show rather big scatter with the change of locations even in a static bond test. The average bond stress is calculated using the pullout force divided by the whole bonded area, and the slip used here is the overall slip occurring at the loaded end. It is observed that the average bond stress-slip relationship shows significantly nonlinear property and the slip reached at the first fatigue cycle is about 0.25mm when the pullout force in CFRP sheets is 73% of the maximum pullout force (See Fig. 10.a). And also, the increase of overall interface slip is rather rapid and causes a fatigue failure after 285 cycles only. Comparatively, the average bond stress-slip curve in Fig. 10.b shows some nonlinearity at the first fatigue cycle but not as significant as that in Fig. 10.a. The overall slip achieved at the first cycle is only about 0.14 mm. The increase of slip with the number of fatigue cycles shows much slower speed compared to that indicated in Fig. 10.a. Therefore, the fatigue life of CFRP/concrete bonded joints under a lower stress level in FRP sheets becomes longer. Figure 11 shows the relationship between fatigue life (number of loading cycles till failure) and the stress level in CFRP sheets, namely *S-N* curve. It can be seen that *S-N* curve can be approximately described as a linear one if *N* is expressed at a logarithmic scale, and the fatigue life decreases with increases of the stress level in CFRP sheets. By understanding the failure mechanisms of CFRP/concrete joints under fatigue loading, it can be known that an overall fatigue failure of a CFRP/concrete joint can be prevented by increasing the bond length. However, increase of bond length does not help the prevention of a CFRP/concrete joint from a static bond failure due to the existence of effective bond length.

Experimental results on the *S-N* relationships for CFRP/concrete joints with initial frost damages are included in Fig. 11. It is indicated that the *S-N* relationships for all CFRP/concrete bonded joints can become a unique one if *S* is expressed by the relative stress level, which is the ratio of the applied tensile force in FRP sheets to the maximum static pullout one in each test series. In other words, the fatigue life is independent of the initial frost damage for any CFRP/concrete joints if a same relative stress level is kept in the CFRP sheets. However, the *S-N* curves will be different if the *S* is expressed by the average bond stress in the fatigue tests for CFRP/concrete joints (see Fig.12). As shown in Fig.12, the slopes of the regressed lines for the average bond stress versus fatigue life relationships are similar for the CFRP/concrete joints with different levels of initial frost damages. However, the fatigue life of CFRP/concrete bond joints, in which the RDEM is decreased to 68% in concrete, is shortened at a same level of average bond stress. In order to prevent fatigue failures of CFRP/concrete bonded joints, therefore a limit for the average bond stress (strain level in FRP sheets) should be linked to the initial frost damage in the concrete substrate. A careful and quantitative inspection on the concrete substrate hence becomes critical.

CONCLUSIONS

1. For both static and fatigue tests, CFRP/concrete bonded joints with and without initial frost damage in concrete showed two different debonding modes, namely covercrete delamination and FRP peeling from concrete, respectively.
2. When the relative dynamic elastic modulus of concrete decreased to 85%, CFRP/concrete bonded joints did not show loss of static bond strength. With a further decrease of the relative elastic dynamic modulus of concrete to 68%, CFRP/concrete bonded joints showed reduction both in bond strength and in interface bonding stiffness.
3. The effective bond length of CFRP/concrete joints experienced substantial frost damage was increased because the frost damage decreased the bonding stiffness and the interfacial fracture energy (the maximum pullout force) of CFRP/concrete joints.
4. Increase of the tensile stress in FRP sheets shortened the fatigue life of all CFRP/concrete bonded joints in both frost damaged and non-frost damaged cases. The *S-N* curves, however, were unique for all CFRP/concrete bonded joints regardless of the initial frost damages their concrete experienced when *S* was expressed by the ratios of the applied tensile forces in FRP sheets to the maximum pullout ones obtained under different levels of frost damages. For a same average bond stress applied for CFRP/concrete joints, decrease of RDEM to 68% in concrete shortened the fatigue life.
5. Observations in this study remind that external bonding technology using FRP is meant for deteriorated concrete members. Careful and quantitative inspections should be carried out to evaluate various damages concrete substrate experienced. Frost deterioration is one of the main concerns. More experiments are necessary for the purpose of reflecting this concern in a refined strengthening design. A possible solution is to give a threshold value for the initial frost damage in concrete or to build up a quantitative link between the strain limit in the FRP sheets and the initial frost damage.

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Table 1--Mechanical Properties of FRP and adhesive

Material	E (GPa)	f_t (MPa)	t (mm)	ε_u (%)
CFRP sheets	230	3550	0.11	1.5
Adhesive	2.41	44.7	1.0	1.85

Note: E: Elastic modulus; f_t : tensile strength; t: design thickness; and ε_u: Elongation.

Table 2-- Details of specimens and test results

Specimen code[1]	RD (%)	τ_{aver} (MPa)	f_c' (MPa)	P_{max} (kN)	S_{max} (mm)	Fatigue life	Failure mode[2]	Note[3]
S1-0-1	100	2.3	34.8	20.9	0.37	1	FP	static test
S1-0-2		3.1		28.3	0.35	1	FP	
S2-0-1	70	3.3	23.4	29.3	0.65	1	CD	
S2-0-2		2.3		20.9	0.45	1	CD	
S3-0-1	85	2.8	34.1	25.5	0.49	1	CD	
S3-0-2		2.6	(36.0)	23.5	0.49	1	CD	
S4-0-1	68	2.8	25.7	25.1	0.59	1	FP[4]	
S4-0-2		1.8	(36.0)	16.2	0.27	1	CD	
S4-0-3		2.1		18.8	0.18	1	CD	
S4-0-4		1.7		15.2	0.32	1	CD	
S1-1	100	2.0	34.8	73%	>0.41	285	FP	fatigue test
S1-2		1.7		64%	>0.48	23,074	FP	
S1-3		1.4		51%	>0.37	1,495,810	FP	
S2-1	70	1.9	23.4	67%	>0.34	5,710	CD	
S2-2		1.4		50%	>0.46	>2,000,000	CD	
S3-1	85	2.1	34.1	78%	>0.27	7,363	CD	
S3-2		1.7		63%	>0.31	83,858	CD	
S3-3		1.4		52%	>0.37	>2,000,000	CD	
S4-3	68	1.2	25.7	60%	>0.43	1,425,729	CD	

Note: RD: relative dynamic elastic modulus (RDEM); τ_{aver}.:the average interfacial bond stress at failure; f_c': compressive strength of concrete; P_{max}: the maximum pullout force and the pullout force in FRP sheets in percentage of the maximum static pullout force in static and fatigue tests, respectively; and S_{max}: the maximum slip at loaded end at failure;

1: Notation of specimen code S-x-y-z: x means test series (S1, S2, S3 and S4) with different RDEM of concrete; y means different stress levels for fatigue tests (0, 1, 2 and 3 among which 0 means static test); and z means number for specimens with the same test variables.

2: FP: FRP peeling from concrete substrate; and CD: Covercrete delamination;

3: Some specimens designed for fatigue tests failed in the first cycle and then the test result became a static one.

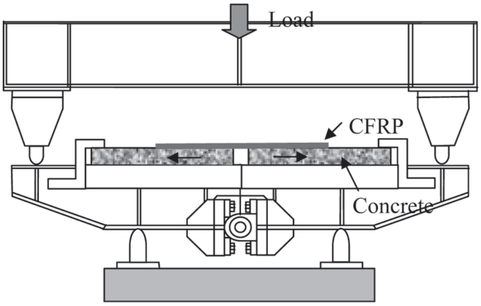

Figure 1 — A sketch of test system

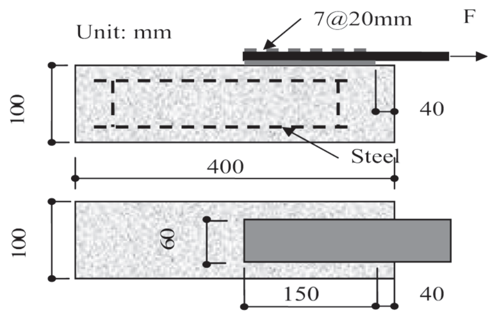

Figure 2 — Dimensions of CFRP/Concrete bonded joints

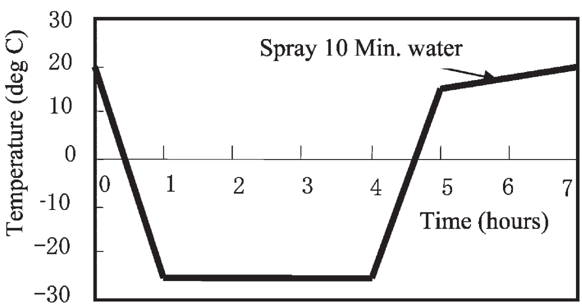

Figure 3 — Temperature variations in a freeze-thaw cycle

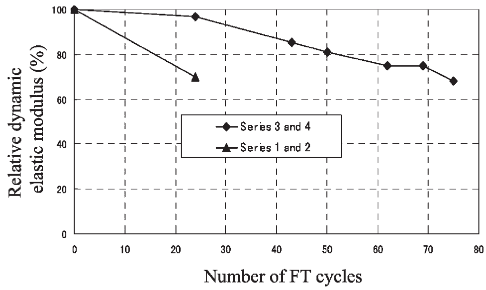

Figure 4—Change of relative dynamic elastic modulus of concrete with FT cycles

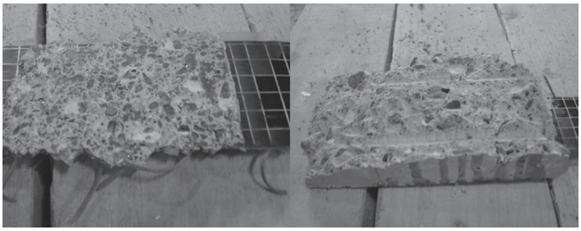

Figure 5—Two different debonding modes for non-frost and frost damaged cases (Left: FRP peeling (FP), Right: Covercrete delamination)

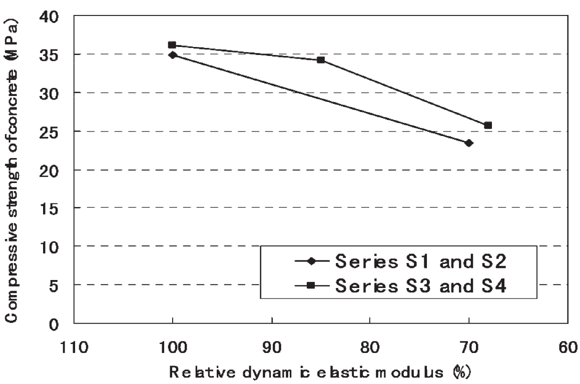

Figure 6 — Effects of frost damage on the compressive strength of concrete

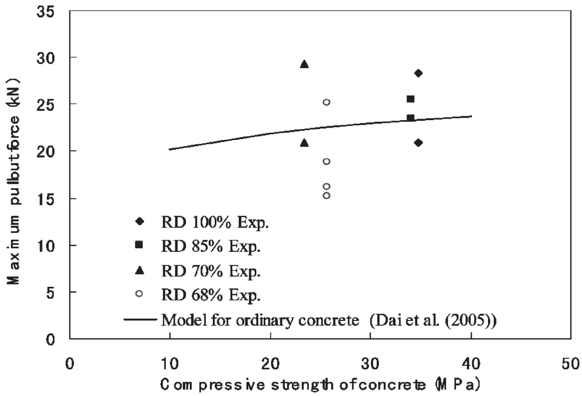

Figure 7 — Effects of frost damage on the interface static bond strength

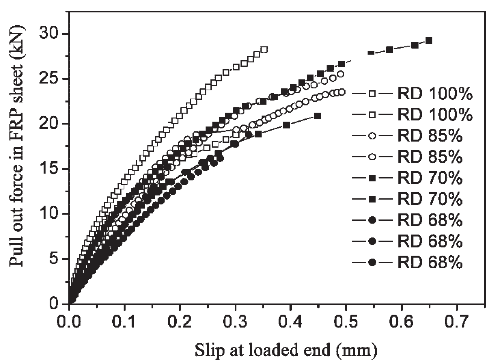

Figure 8 — Load-slip curves at loaded end of FRP sheets

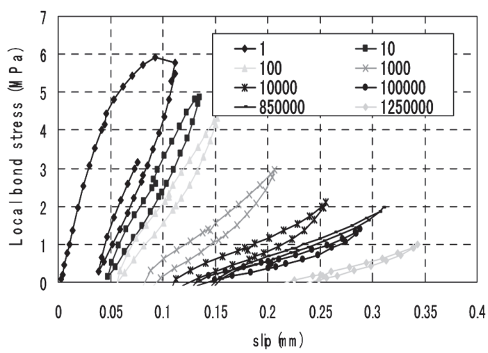

Figure 9 — A typical fatigue local *t-s* curve of CFRP/concrete joints

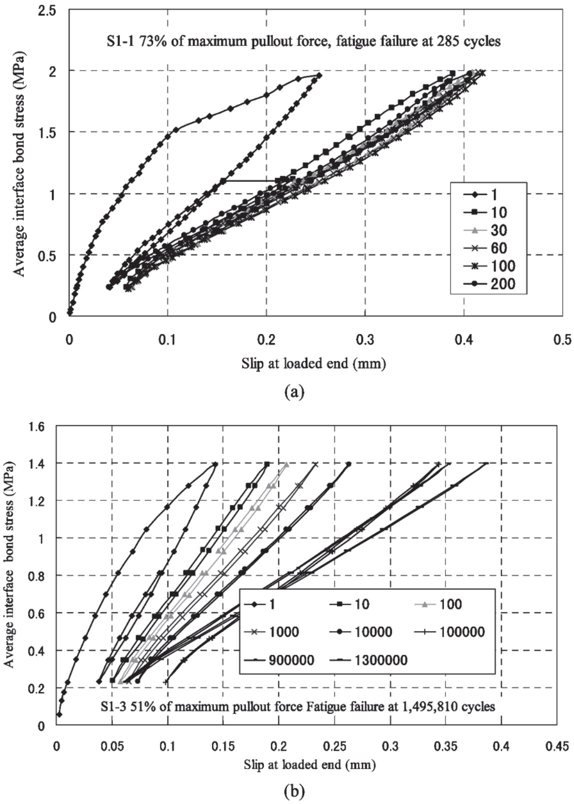

Figure 10 — Average *t-s* curves of CFRP/concrete joints under fatigue loading
(a) Specimen S1-1; (b) Specimen S1-3

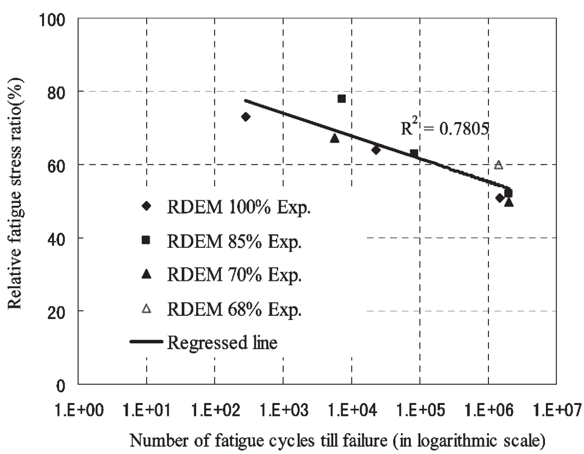

Figure 11 — *S-N* relationship of CFRP/Concrete bonded joints
(S is expressed by a relative stress ratio)

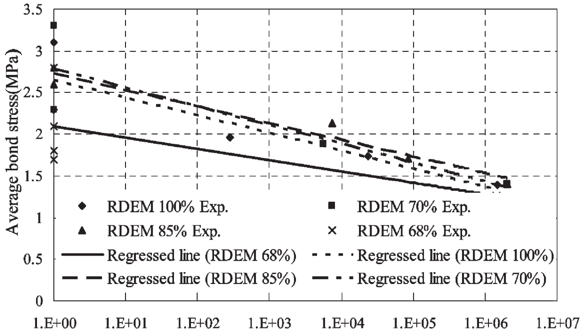

Figure 12 — *S-N* relationship of CFRP/Concrete bonded joints (S is expressed by an average bond stress)