Shear Resistance of Concrete Connections Between Existing RC Frames and Newly-Added PCaPC Frames For Retrofit

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SUMMARY:
Attaching advanced lateral force resisting system from outside the existing buildings has been a well accepted technique to enhance the seismic performance of these buildings. The newly-added system needs to be firmly fastened to the existing one. In this paper, the results of an experimental test on the shear strength of concrete interface with both post-installed anchors and prestressing steel rods is reported. The interface is critical for fastening a precast prestress frame to an existing reinforced concrete building. Various combinations of post-installed anchors and prestressing rods were tested to prove that it is appropriate to add up the contribution of the two components to obtain the total shear strength of the interface. In addition, the applicability of this technique to existing building with very low strength concrete was examined. An equation of estimating the shear strength of interface with both post-installed anchors and prestressing steel rods is proposed.

Keywords: Seismic Retrofit, Shear Resistance, Prestressing Rod, Post-Installed Anchor, Concrete Connections

1. INTRODUCTION

The seismic performance of old buildings in Japan, especially of those built before 1981 is usually inadequate to meet the requirement of the current seismic design provisions so that a large amount of existing buildings need to be seismically retrofitted to be better prepared for the next major earthquake. Following the 1995 Kobe earthquake and the enforcement of the Act on Promotion of Seismic Retrofitting of Buildings, seismic inspection and retrofit of existing buildings have been widely carried out in Japan. The effect of this continuing effort during the past 15 years has been proved by the most recent M9.0 Tohoku-Pacific earthquake. The observed building damage due to the ground shaking of this earthquake was not severe and not proportionate to the intensity of this earthquake. One of the most commonly used retrofit methods for reinforced concrete (RC) buildings is to attach a precast prestressed concrete (PCaPC) frame from outside the existing buildings so that the two form an integral system to resist the lateral action of earthquakes (Fig. 1.1, JBDPA (2001)).

Connecting slabs are generally used to connect the newly added PCaPC frame to the beams in the existing RC building. Either post-installed anchors or prestressing steel rods can be used to provide the shear capacity of the interface between the existing beam and the connecting slab as suggested by JBDPA (2002). This kind of retrofit has become favorable mainly because they can be carried out without suspending the occupancy of the building. In addition, they generally do not impair any opening on the facade. The PCaPC frame, which consists of precast concrete beam and columns compressed together by prestressing strands, could also provide additional self-centering capacity to the system to make the building more resilient. In this study, it is proposed to use both the post-installed anchors and the prestressing steel rods for the connection and the combined shear strength of the interface is investigated through an experimental program on the connecting slab.

In real structures, such a connecting slab may be subject to a complicated combination of shear, compressive and tensile stress due to the eccentricity. In the current experiment, however, only in-plane shear action is considered. Furthermore, this kind of retrofit is generally applicable to concrete whose compressive strength σ_B is higher than 18MPa. There are, however, buildings made of concrete with lower strength. It is also of interest to investigate if the retrofit method is also applicable to buildings with very low strength concrete.

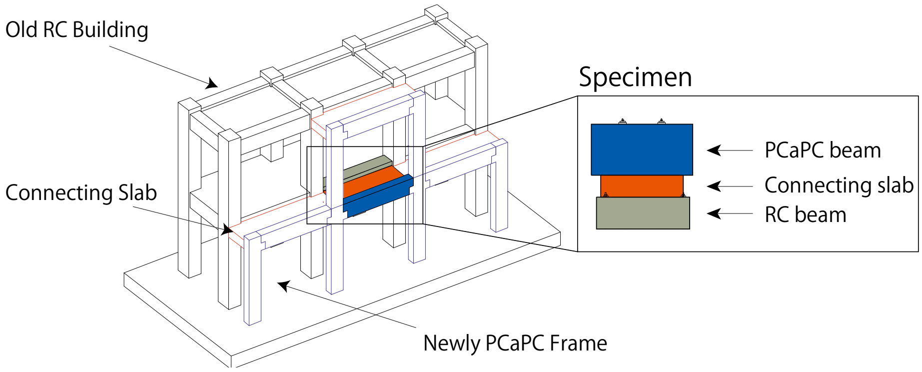


Figure 1.1. Attached PCaPC Frames for Retrofit and Specimen

2. EXPERIMENTAL PROGRAM

2.1. Specimens

Ten full-scale specimens were tested, each consisting of a PCaPC beam, a connecting slab and a RC beam. The PCaPC beam represents the newly attached frame and the RC beam represents the existing RC building (Fig. 1.1). The test parameters and major material properties of the specimens tested are listed in Table 2.1. and Table 2.2., and the configuration of specimens are shown in Fig. 2.1. The test parameters were a type of connecting bars, a level of prestressing force and a concrete strength of the existing RC beam. First, the tested connecting slabs, or more specifically the connecting interface between the connecting slab and the RC beam, have connecting bars which can be classified into three types: (1) that with only two prestressing rods at $0.5P_y$ prestressing force; (2) that with only five post installed anchors; (3) that with both two prestressing rods and five post-installed anchors. Here, P_y is a nominal yield strength of prestressing rods. Next, for the specimens with both prestressing rods and post-installed anchors, the level of prestressing force per rod is varied. The prestressing force per rod was taken to be 0, $0.5P_y$ and $0.9P_y$. Now, we have five types specimen. Finally, the concrete strengths are 10MPa and 18MPa for each type specimen.

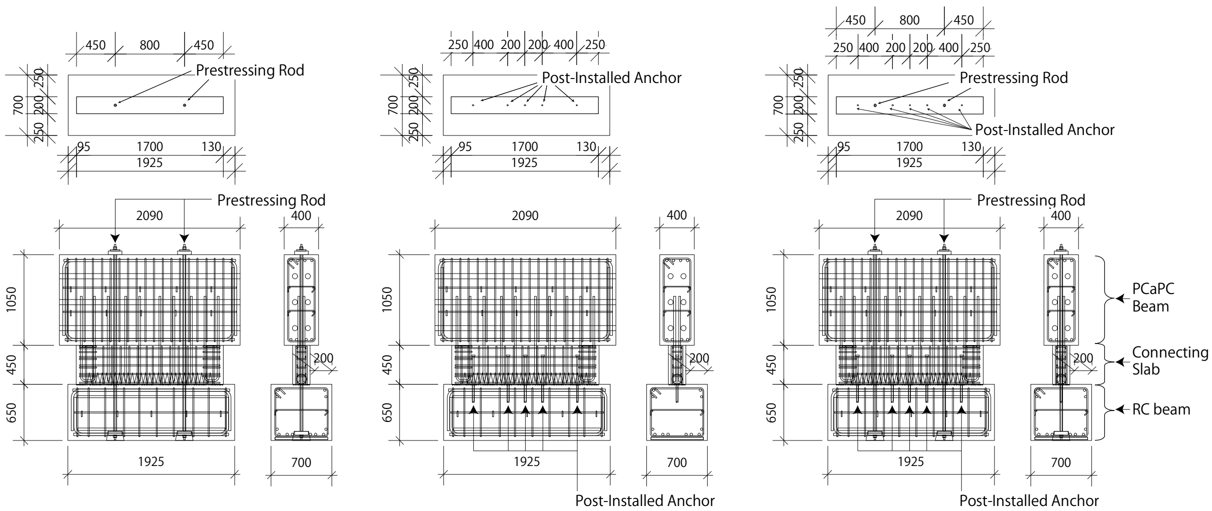
Prestressing rods ϕ17 (Type B, SBPR 930/1080) provided by JIS G 3112 are used. Grout was injected into the sheath for the prestressing rod after the prestress was finished. Deformed rebars D16 (SD345) provided by JIS G 3112, and film tube type-organic anchors for the post installed anchors.

Table 2.1. Parameters of Specimens

Specimens	Connecting Bars		Initial Prestress per Bar	Specified Design Strength of RC Beam
	Prestressing Rod	Post-Installed Anchor	P_0	F_c
			kN	MPa
10-0.5P	○	-	106 ($0.5P_y$)	10
18-0.5P	○	-	106 ($0.5P_y$)	18
10-A	-	○	-	10
18-A	-	○	-	18
10-0PA	○	○	0	10
18-0PA	○	○	0	18
10-0.5PA	○	○	106 ($0.5P_y$)	10
18-0.5PA	○	○	106 ($0.5P_y$)	18
10-0.9PA	○	○	190 ($0.9P_y$)	10
18-0.9PA	○	○	190 ($0.9P_y$)	18

Table 2.2. Material Properties

Specimens	Steel				Concrete					
	Prestressing Rod		Post-Installed Anchor		PCaPC Beam		Connecting Slab		RC Beam	
	Yield Strength	Young's Modulus	Yield Strength	Young's Modulus	Compressive Strength	Young's Modulus	Compressive Strength	Young's Modulus	Compressive Strength	Young's Modulus
	${}_p\sigma_y$	${}_pE_s(\times10^5)$	${}_p\sigma_y$	${}_aE_s(\times10^5)$	σ_B	$E_c(\times10^4)$	σ_B	$E_c(\times10^4)$	σ_B	$E_c(\times10^4)$
	MPa		MPa		MPa		MPa		MPa	
10-0.5P					56.0	4.14	24.6	3.49	8.9	2.06
18-0.5P					52.9	4.03	25.8	3.08	17.1	2.67
10-A					56.5	4.16	24.8	3.45	9.0	2.08
18-A					53.0	4.03	26.3	3.12	17.3	2.68
10-0PA	1241	2.00	401	-	58.2	4.23	25.3	3.32	9.3	2.16
18-0PA					53.2	4.03	27.4	3.22	17.7	2.70
10-0.5PA					58.9	4.26	25.5	3.27	9.4	2.19
18-0.5PA					53.2	4.03	27.5	3.23	17.8	2.71
10-0.9PA					60.3	4.32	25.9	3.17	9.6	2.25
18-0.9PA					53.3	4.03	27.8	3.26	17.9	2.71



(a) With only Prestressing Rods

(b) With only Post Installed

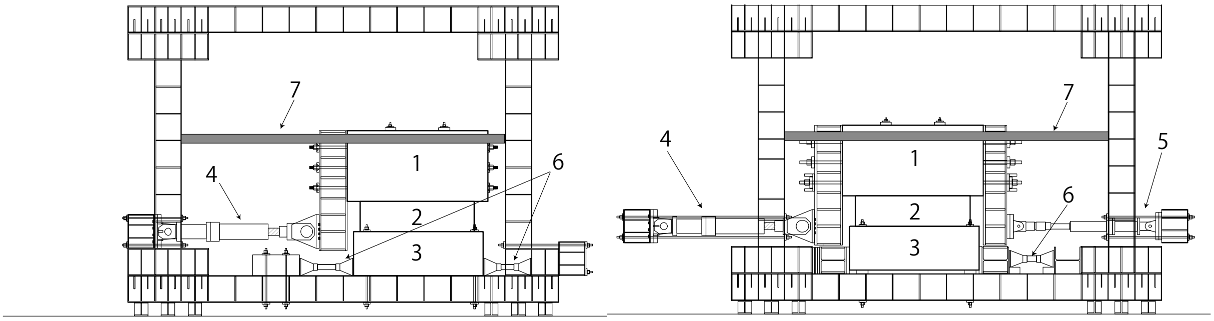
(c) With both Post Installed Anchors and Prestressing Rods Anchors

Figure 2.1. Dimensions and Reinforcement of Specimens

2.2. Testing Method

Depending on the estimated shear strength of the specimens, two loading setups were used (Fig. 2.2.). Type-A employed a single oil jack while Type-B used two. The specimen was installed vertically with the RC beam at the bottom. Shear force is applied at the concrete interface. Cyclic loading with increasing amplitude was carried out. The load history is shown in Fig. 2.3. Before the bond failed, the loading was controlled by force. The increment of force amplitude was first a fraction of the estimate shear strength of the interface ${}_{add}Q$ (Eq. 3.3), then 100kN after the load exceeded the estimate strength ${}_{add}Q$. After the bond failed, the loading was shifted to displacement control and the maximum slip used was 10mm.

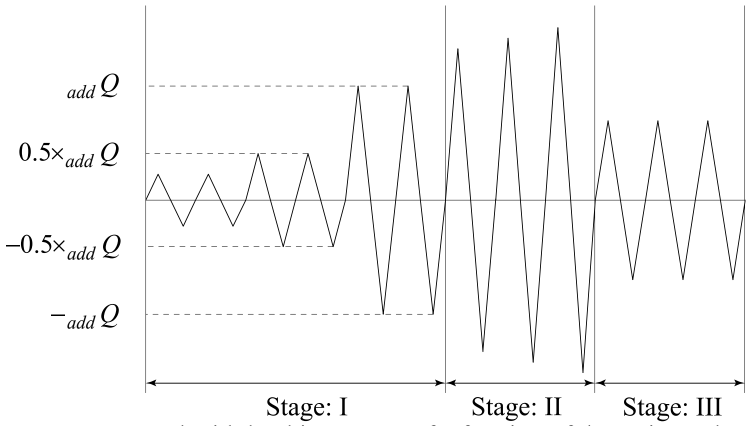
The total load acting on the interface was taken as the sum of measured load in the 1000kN oil jack and that in the 500kN oil jack for Type-B setup. Displacement between the connecting slab and the RC beam was measure. This measured displacement might also have included the shear deformation of the RC connecting slab itself because the transducer was installed on the connecting slab several centimeter above the RC beam's top surface. However, this shear deformation should be less than 0.1mm at 1000kN shear force assuming that the connecting slab remains elastic. Therefore, it is deemed that the measured displacement is close enough to the slip at the interface, especially when the interface behavior after the failure of the bond is concerned.



1~3: Specimen (1: PCaPC beam, 2: Connecting slab, 3: RC beam)
4: 1000kN jack, 5: 500kN jack, 6: Screw jack, 7: Out of plane constraint

(a) Type-A (b) Type-B

Figure 2.2. Loading Setup



Stage: I Force control with load increment of a fraction of the estimated strength $_{add}Q$.
Stage: II Force control with 100kN load increment
Stage: III Displacement control, 10mm maximum slip

Figure 2.3. Loading History

3. METHOD OF EVALUATING INTERFACE SHEAR STRENGTH

3.1. Conventional Evaluating Method

The shear strength of the connecting interface may come from (1) friction of prestressing on the interface; (2) dowel action of the post-installed anchors and (3) adherence of the concrete interface. However, the total shear strength of a connecting slab may not be equal to the sum of these components because they are not obtained at the same slip in shear (Nakano and Matsuzaki, 2001). The adherence would be quickly lost even with very small slip at the interface while the dowel action is not likely to take effect until some considerable slip occurs. In the whole process of shearing, the friction may always exist without much displacement. Because the adherence strength of the concrete interface may highly depend on the condition, such as the roughness, of the surface and it will be quickly lost with any considerable slip, it is preferred not to take it into account in the design of the connection. Instead, it is regarded as safety factor. Thus, the other two components, i.e., the contribution of the dowel action and that of the friction, may be readily summed up for the overall shear strength of the interface because the friction exists all the time. This is to be proved by the experiment in this study.

In the evaluating method recommended by JBDPA (2002), the shear strength of the prestressing rods is evaluated as the friction force, Q_{fr}, calculated by Eq. 3.1, while that of the post-installed anchor is evaluated as a dowel action, ${}_aQ_a$, calculated by Eq. 3.2 ${}_aQ_a$, is the smaller of ${}_aQ_{a1}$ and ${}_aQ_{a2}$ while ${}_aQ_{a1}$ in Eq. 3.2a is the strength when the anchor fractures in shear at the concrete interface and ${}_aQ_{a2}$ in Eq. 3.2b is the strength when the concrete surrounding the anchor is crushed.

$$Q_{fr} = \mu \cdot {}_p n \cdot \eta P_0 \tag{3.1}$$

where ${}_pn$ is the number of prestressing rod, μ is the coefficient of friction, η is the effectiveness ratio of prestressing, P_0 is the initial prestressing force per a rod. The effective ratio of prestress, η, in those equations should represent the prestress loss due to the creep and drying shrinkage of concrete. In case of this experiment, the test was carried out within three months after the specimens were cast before any considerable loss of the prestress could take place due to the creep or drying shrinkage of the concrete. So it is believed appropriate to make η equal to 1.0, rather than the recommended value of 0.85 in JBDPA (2002) and AIJ (1998).

$${}_aQ_a = \phi_s \times {}_a n \times \min\left({}_aQ_{a1}, {}_aQ_{a2}\right) \tag{3.2}$$

$${}_aQ_{a1} = 0.7 \times {}_a a \times {}_a \sigma_y \tag{3.2a}$$

$${}_aQ_{a2} = 0.4\sqrt{E_c \cdot \sigma_B} \times {}_a a \tag{3.2b}$$

where ${}_an$ is the number of post-installed anchor, ϕ_S is a reduction factor, ${}_aa$ is the sectional area of a post-installed anchor, ${}_a\sigma_y$ is the yield strength of post-installed anchor, E_c is the Young's modulus of concrete, σ_B is the compressive strength of concrete. The reduction factor ϕ_S represents ratio for the ultimate shear strength of the post-installed anchor as the dowel action. As recommended by JBDPA (2002), the reduction factor ϕ_S is taken as 0.7 for shear slip 2mm.

It is assumed that the ultimate strength of post installed anchor steel is 1.2 times its yield strength and hence $0.7\sigma_y$ in the right hand side of Eq. 3.2a represents the ultimate shear strength of anchor steel. Eq. 3.2b refers to J. G. Ollgaard et al (1971)

3.2. Proposal Evaluating Method

As recommended by JBDPA (2002), either Eq. 3.1 or Eq. 3.2 should be used to evaluate the interface shear strength when post-installed anchors or prestressing rods are used for the connection. No method is provided in JBDPA (2002) to estimate the interface shear strength with both the prestressing rods and the post-installed anchors. In addition, the shear strength of the prestressing rods is evaluated as the friction only. Since no recommendation is provided in JBDPA (2002) to estimate the interface shear strength with both the prestressing rods and the post-installed anchors, and the shear strength of prestressing rods as dowel action, an equation of doing this is proposed Eq. 3.3 and Eq. 3.4.

$${}_{add}Q = Q_{fr} + {}_aQ_a + {}_pQ_a \tag{3.3}$$

$${}_pQ_a = \phi_s \times {}_p n \times \min({}_pQ_{a1}, {}_pQ_{a2}) \tag{3.4}$$

$${}_pQ_{a1} = 0.7 \times \sqrt{1 - \left(\frac{\eta P_0}{1.2 P_y}\right)^2} \times P_y \tag{3.4a}$$

$$ {}_{p}Q_{a2} = 0.4\sqrt{E_c \sigma_B} \times \sqrt{1 - \left(\frac{\eta P_0}{1.2 P_y}\right)^2} \times {}_{p}a \tag{3.4b} $$

where ${}_{p}a$ is the sectional area of a single prestressing rod.

The shear strength, ${}_{add}Q$ in Eq. 3.3 is the sum of Q_{fr} in Eq. 3.1, ${}_{a}Q_a$ in Eq. 3.2 and an extra term, ${}_{p}Q_a$, representing the dowel action of the prestressing rods. ${}_{p}Q_a$ is evaluated as the smaller of ${}_{p}Q_{a1}$ and ${}_{p}Q_{a2}$, the same manner as evaluating the shear strength of the post-installed anchor in Eq. 3.2. For post-installed anchor steel, a pure shear stress state can be readily assumed to estimate its shear strength from its direct tensile yield strength. For prestressing rod steel, however, a combined tensile and shear stress state has to be considered when calculating its shear strength because the prestressing rods are usually carrying considerable initial axial force when it is subject to shear force. In the light of this, the middle term on the right hand side of Eq. 3.4a is adopted to take into account the influence of the prestress on the shear strength of the prestressing rods. It is derived by assuming that the yield surface of prestressing rod steel follows the von Mises criterion. The reduction factor ϕ_S is same value with post-installed anchors.

4. TEST RESULTS AND DISCUSSIONS

4.1. Shear Force-Slip Displacement Relation

Shear force versus slip relationship is shown in Fig. 4.1. The data of Specimen No.2 has two figures because the test of No.2 was stopped immediately a first loading after the failure of the bond of the interface. Characteristic loads, namely the maximum shear force, Q_{max}, and the shear force at ±2mm and ±10mm slip, Q_{+2mm}, Q_{-2mm}, Q_{+10mm}, and Q_{-10mm}, are also marked in Fig. 4.1 and listed in Table 4.1.

The stiffness of all specimens is high before the failure of the interface bond (Fig. 4.1). When the bond failed, the strength of most specimens underwent a sudden loss and a considerable slip occurred. The shear strength of all specimens declined with cyclic loading.

As can be seen in Table 4.1., Q_{-2mm} of Specimen No. 3 and 4 is much lower than their Q_{+2mm}. It may be caused by the crushing of the concrete around the post installed anchors when the slip suddenly increased due to the brittle failure of the bond. Other specimens also exhibited different Q_{-2mm} and Q_{+2mm}. In the following discussion, the average value of the two, denoted as ${}_{ave}Q_{2mm}$, is used.

It is also worth noting that the shear strength is not so much different between specimens with σ_B = 18MPa and 10MPa concrete strength for the RC beams. From the result of this experiment, it seems that the concrete strength of the existing RC beam has little effect on the interface shear strength.

4.2. Shear Strength Evaluated

Shear strength predicted by the design method in Eqs. 3.1 and 3.2 are compared with the test results in Table 4.1. Test strength of the steel and the concrete, as listed in Table 2.2, are used in the calculation. Only prestressing rods were used in Specimen 10-0.5P and 18-0.5P to strengthen the connecting interface. It would be appropriate to use Eq. 3.1 to estimate their shear strength after the interfacial bond fails. As mentioned above, the test of Specimen No. 2 was prematurely stopped and thus the post-peak strength was not obtained. For Specimen No.1, however, it is obvious that Eq. 3.1 may over underestimate its shear strength. The friction force calculated by Eq. 3.1 is less than half of the test result. Though it may be partly due to an underestimate of the coefficient of friction of the interface, it is not likely the sole reason of the underestimation because a coefficient of friction of greater than 1.0 would be required to match the test result. Actually, note that the prestressing rods were bonded by grout in this specimen. The prestressing rods themselves were very likely to be able to provide some shear capacity by dowel actions.

For Specimen 10-A and 18-A, only the post-installed anchors were employed in the interface. Their post-peak shear strength may come primarily from the dowel action of the anchors. Although some friction may also have existed in the interface due to the weight of the concrete block and steel jigs above the interface, it was only marginal. By comparing with the test result, it is seen that the shear strength calculated by Eq. 3.2 agrees quite well with the test results, ${}_{ave}Q_{2mm}$, of Specimen 10-A and 18-A.

The shear strength predicted by Eq. 3.3, ${}_{add}Q$, is compared in Table 4.1. as well as in Fig. 4.2. with ${}_{ave}Q_{2mm}$ and ${}_{ave}Q_{10mm}$ from the test results. The shear strength at 10mm, ${}_{ave}Q_{10mm}$, is considered as the ultimate strength. Therefore, the reduction factor ϕ_S=0.7 is employed for the shear strength at slip 2mm, ${}_{ave}Q_{2mm}$, in other hand, ϕ_S=1.0 is employed for the shear strength at slip 10mm, ${}_{ave}Q_{10mm}$.

The estimated strength of the shear strength at slip 10mm generally agrees well with the test results as can be seen in Fig. 4.2. It indicates that it is appropriate to add the ultimate shear strength contributions of the post installed anchor and the prestressing rods. The estimated strength of the shear strength at slip 2mm has safety margin. It is also worth mentioning that adequate safety margin should be guaranteed when estimating the interface shear strength by Eq. 3.3. in design practice because large variation is observed in the test results.

Table4.1. Experimental and Predicted Shear Strength of Interface

Specimen		10-0.5P	18-0.5P	10-A	18-A	10-0PA	18-0PA	10-0.5PA	18-0.5PA	10-0.9PA	18-0.9PA
Result of Experiment	Maximum shear force Q_{max} (kN)	888	922	772	719	821	983	1000	1390	1200	1372
	Shear force at +2mm Q_{+2mm} (kN)	211	136	253	241	537	254	573	594	852	998
	Shear force at -2mm Q_{-2mm} (kN)	282	54	17	30	291	494	704	294	729	698
	${}_{ave}Q_{2mm}$(kN)	246	95	135	136	414	374	638	444	790	848
	Shear force at +10mm Q_{+10mm} (kN)	375	289	312	365	687	674	697	688	587	793
	Shear force at -10mm Q_{-10mm} (kN)	334	181	228	194	-	545	602	648	603	636
	${}_{ave}Q_{10mm}$(kN)	354	235	270	280	687	610	649	668	595	714
JBDPA (2002)	Q_{fr}(kN)	119	119	-	-	0	0	119	119	203	203
	${}_{a}Q_{a}$(kN)	-	-	121	168	125	168	126	168	131	168
Proposed	${}_{add}Q$(kN) (ϕ_S=0.7)	170	199	134	203	195	294	299	394	376	463
	${}_{add}Q$(kN) (ϕ_S=1.0)	191	233	185	284	273	414	376	512	451	574

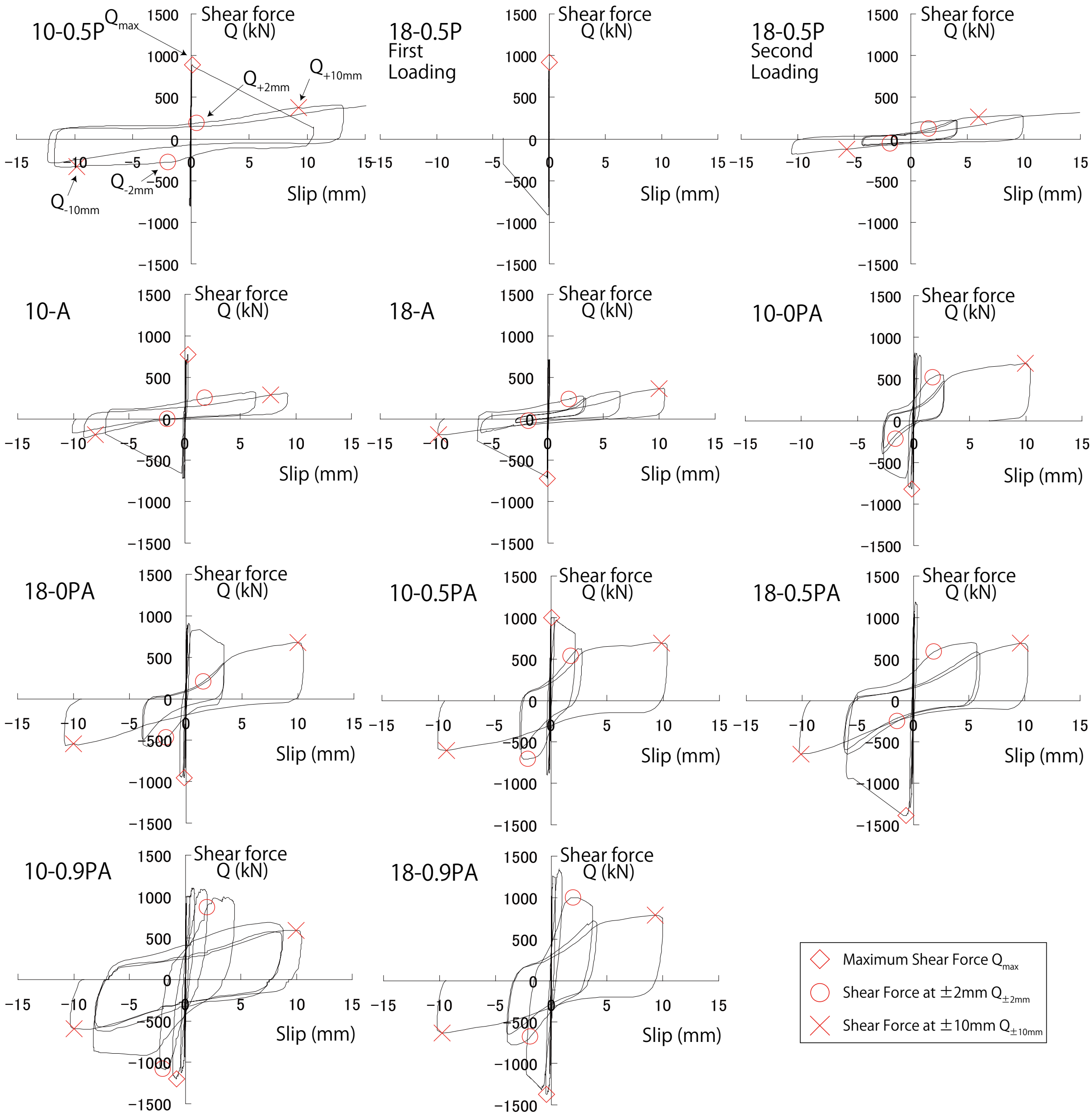


Figure 4.1. Shear Force versus Slip Relation

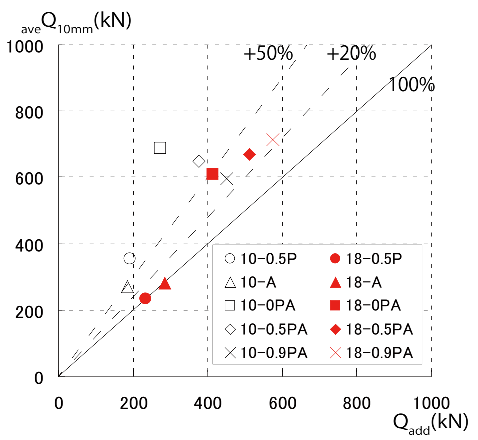


Figure 4.2.
Comparison between $_{ave}Q_{10mm}$ and $_{add}Q$

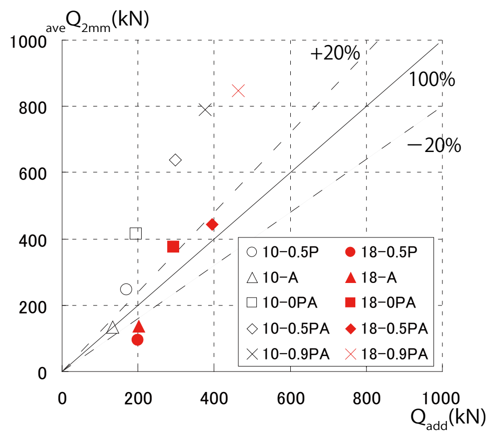


Figure 4.3.
Comparison between $_{ave}Q_{2mm}$ and $_{add}Q$

5. CONCLUDING REMARKS

The following findings are obtained through the experiment and the above discussions.

(1) Shear force versus slip relationship of the interface was observed in the experiment. Although the bond of the interface exhibited very brittle failure, the strength provided by the dowel action of the interface reinforcement and the friction force is quite stable.

(2) The proposal equation works well in predicting the interface ultimate shear strength with either post-installed anchors or prestressing rods or both. It also indicates that the shear strength of the connecting slab at 10mm interface slip can be appropriately estimated by the sum of the contribution of the post installed anchors and the prestressing rods.

(3) The proposal equation has safety margin in predicting the interface shear strength at 2mm slip.

(4) In this experiment, the compressive strength of RC beam representing the existing RC building has little effect on the interface shear strength.

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