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Publication date
2022

Document Version
Final published version

Published in
IABSE Symposium Prague, 2022

Citation (APA)
Yang, T., Xie, R., & Veljkovic, M. (2022). Shear performance of replaced bolt shear connectors in prefabricated composite beams. In *IABSE Symposium Prague, 2022: Challenges for Existing and Oncoming Structures - Report* International Association for Bridge and Structural Engineering (IABSE).

Shear performance of replaced bolt shear connectors in prefabricated composite beams

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Abstract

Bolt shear connectors have the advantage of efficient installation and demolition when used in prefabricated composite beams. When bolt shear connectors are damaged in the service period and replaced by new ones, the shear performance of replaced bolts is to be affected by the existing structural damage. This paper investigates the shear performance of eleven re-assembled push-out specimens of bolt connectors. The experimental results show that the replaced bolts possess a similar shear resistance to the bolts in the original tests. In contrast, the relative slips at the interfaces between the steel beams and the prefabricated concrete (PC) slabs show a bigger scattering. A calculation method of shear resistance for the replaced bolts considering the influence of the existing damage was proposed based on the experiments, and comparisons show that the calculation values agree well with the experimental results.

Keywords: replaced bolt shear connectors, prefabricated composite beams; re-assembly; shear stiffness.

1 Introduction

Steel-concrete composite beams possess superior mechanical properties compared to no composite interaction between two materials [1-2]. With the development of demountable structures in the past decades, researchers worldwide conducted much research on the shear performance of post-installed shear connectors, for example, bolt shear connectors [3-5]. Ataei et al. [6] found that increasing the clearance between the bolts and the prefabricated holes could significantly improve the ductility of the composite connections. Hosseini et al. [7] found that blind-bolt shear connections showed considerable shear ductility levels and had advantages as an alternative replacement to the traditional welded stud. Chen et al. [8] demonstrated that bolt connectors with the corrugated pipe had higher shear stiffness than normally reserved holes. It is worth noting that the "reserved hole" is the hole made in the concrete deck in which the bolt is installed [8]. Yang et al. [9] found that the average ultimate shear resistance per bolt for multi-rows of bolt shear connectors is less than that of bolt shear connectors in a single row.

Resin injected bolts have been investigated in recent years because the injectant could provide almost instantaneous shear interaction despite clearances and improve the durability of bolts. Nijgh et al. [10] presented a methodology that quantified the required hole clearance for a reusable composite floor system and illustrated it on an example of a car park building. A special type of bolted connector containing the embedded coupler in the concrete slab and an injected bolt was used in the study.

It is necessary to replace the bolt shear connectors used in demountable structures when they cannot fulfil the requirements due to local damage. By that time, the shear performance of the replaced bolt shear connectors is most likely to change as the existing structural damage produced in the previous service period. Few studies have been conducted on the shear performance of replaced bolt shear connectors. This paper conducts two rounds of push-out tests on the same batch of push-out specimens of bolt shear connectors. The performance of the replaced bolt shear connectors is compared with that obtained in the first round of tests. The methods to assess the ultimate shear resistance and shear stiffness of the replaced bolt connectors are also discussed.

2 Overview of the first round of push-out tests

2.1 Specimen design

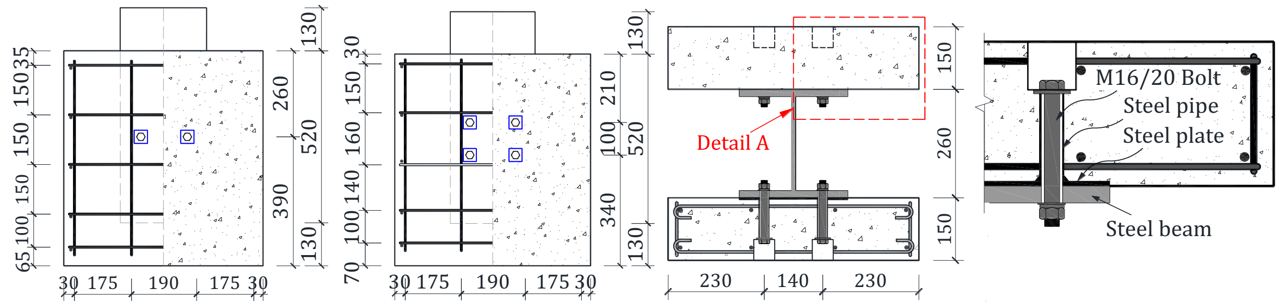
Eleven push-out specimens of bolt shear connectors with different parameters were fabricated following EN1994-1-1: 2004 [11], being appropriately adjusted due to the limitation of constructional details. The parameters such as bolt preload, bolt diameter, and bolt embedment length in the precast concrete (PC) slabs were considered, as listed in Table 1. The detailed geometries and dimensions of the specimens are shown in Fig.1, taking S1 and S9 as examples.

Table 1 Main design parameters of the specimens

ID	Number of the bolts in each slab	Bolt diameter (mm)	Contact interface processing	Bolt preload (kN)	Embedded length of bolts (mm)	Longitudinal bolt spacing (mm)	Additional constructional detail
S1	2	16	RS	72	100	–	–
S2	2	16	RS	90	100	–	–
S3	2	16	RS	54	100	–	–
S4	2	20	RS	113	100	–	–
S5	2	16	SS	72	100	–	–
S6	2	16	RS	72	100	–	WS
S7	2	16	RS	72	80	–	–
S8	2	16	RS	72	130	–	–
S9	4	16	RS	72	100	100	–
S10	4	16	RS	72	100	150	–
S11	4	16	RS	72	100	200	–

Note: RS represents the outer surfaces of the steel beam getting rusted after sandblasting; SS represents spraying the outer surfaces of the steel beam with epoxy zinc-rich paint after sandblasting; WS represents four additional 16 mm diameter and 80 mm high studs were welded on each embedded steel plate to strengthen the anchorage between the PC slabs and the embedded steel plates.

Each specimen consisted of the welded I-shaped steel beam and two PC slabs. In each slab were arranged double-layered reinforcement cages. A steel plate with welded short steel pipes was embedded in each PC slab to position the reserved bolt holes. The cross-section of the welded steel beam is 260×260×18×10 mm (flange width, web width, flange thickness, and web thickness), and the cross-section of the embedded steel plate is 520×260×6 mm (length, width, and thickness). To improve the constructional error resilience of the bolt holes, the holes in the steel beam flanges are 2 mm larger than the bolts in diameter. For 16 mm diameter bolts, the outer and inner diameters of the short steel pipes were 22 mm and 18 mm, respectively, while those for 20 mm diameter bolts were 26 mm and 22 mm, respectively. The mechanical properties of concrete and steel were tested before conducting the push-out tests. The measured compressive strength, the tensile strength, and the elasticity modulus of concrete are 53.1 MPa, 3.1 MPa, and 35.5 GPa, respectively. The yield strength, the ultimate tensile strength, and the elasticity modulus of the steel beam are 372 MPa, 525 MPa, and 206 GPa, respectively. According to the manufacturer's inspection reports, the ultimate tensile strength of the high-strength bolts M16 and M20 are 960 MPa and 950 MPa, respectively.

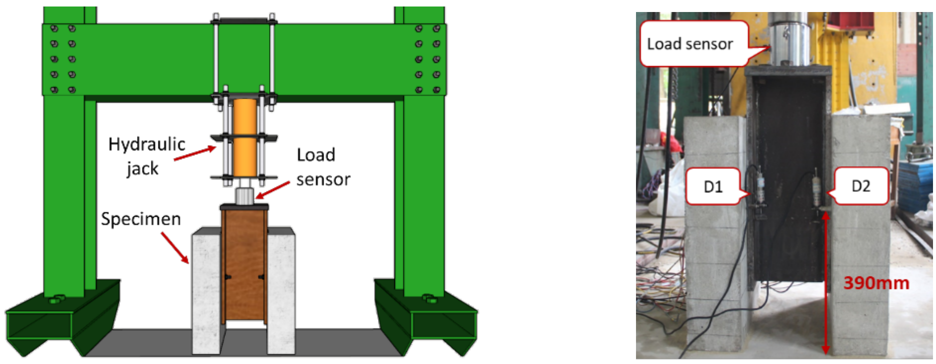


(a) S1 (b) S9 (c) Plane view (d) Detail A

Fig. 1 Specimen geometries and dimensions (unit: mm)

2.2 Loading scheme and instrumentations

The hydraulic jack was used during the push-out tests, and Fig. 2(a) illustrates the assembly of the specimen. Fine sand was paved under the bottom of the PC slabs to eliminate the potential gaps between the PC slabs and the testbed. A steel plate was placed on the top of the steel beam to ensure uniform load transmission during the tests. Before the formal loading test began, loading and unloading cycles, with loads varying between 5%-40% of the ultimate shear resistance, were repeated 25 times, referring to the standard loading procedure specified by EN1994-1-1: 2004 [11]. Then, the monotonic static load was applied up to the load-bearing capacity of the specimens dropped to 80% of the peak load. Shear forces and relative slips were monitored. The measuring instrumentations are shown in Fig. 2(b), where D1 and D2 are displacement meters to measure the relative slip.



(a) Assembly of the specimen *(b) Measuring instrumentation*

Fig. 2. Test setup of the push-out tests

2.3 Failure modes

Fracture of bolt connectors occurred at the final failure for all the push-out specimens in the first round tests and the failed cross-sections of the bolts located at the interface between the PC slab and the steel beam. Several cracks appeared on the PC slab, propagating towards the closer edges of the slab. Fig. 3(a)-(d) shows the typical crack distribution on the PC slab, taking S4 and S11 as examples. Note that the bolt diameter for S4 is 20 mm and that for S11 is 16 mm. The comparison shows that more cracks appeared on the PC slab with the increase in bolt number.

Meanwhile, the holes in the embedded steel plate and the steel beam were locally deformed, as shown in Fig. 3(e)-(f). Fig. 3(g) shows the typical failure modes of the bolt shear connectors. It can be seen that the bolt cross-sections deformed slightly, and nearly no bending deformation was observed.

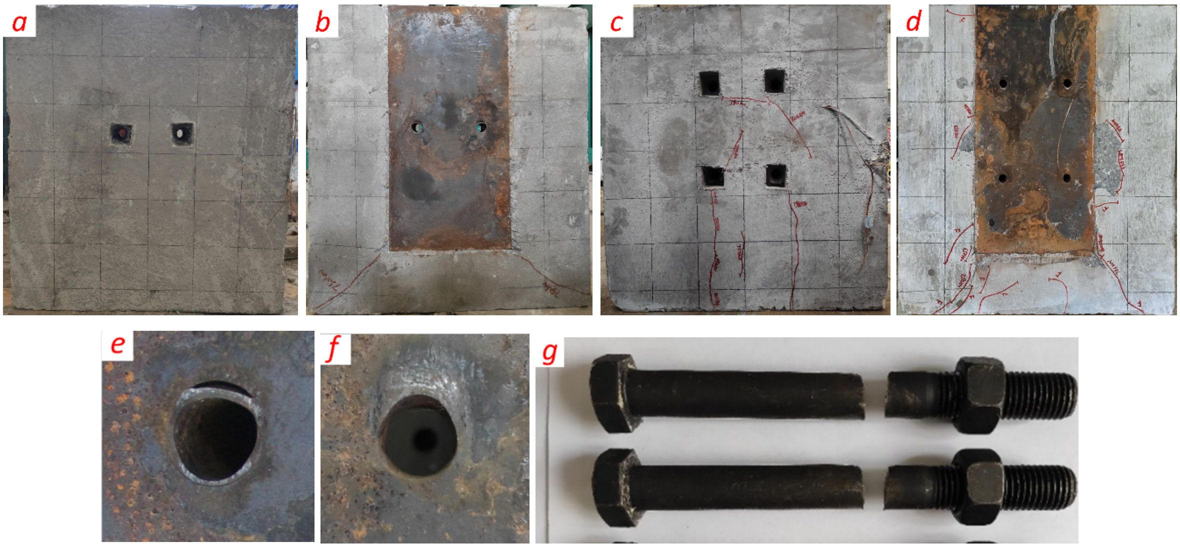


Fig. 3. Typical failure modes of the specimens in the first round of tests:(a) Outer slab surface (S4); (b) Inner slab surface (S4); (c) Outer slab surface (S11); (d) Inner slab surface (S11); (e) Hole in the embedded steel plate; (f) Hole in the beam flange; (g) Fracture of the bolts

2.4 Primary experimental results

Fig. 4 illustrates the representative load-slip curves in the first round of tests. The load-slip curves of the specimens are globally similar and present a three-stage developing tendency:

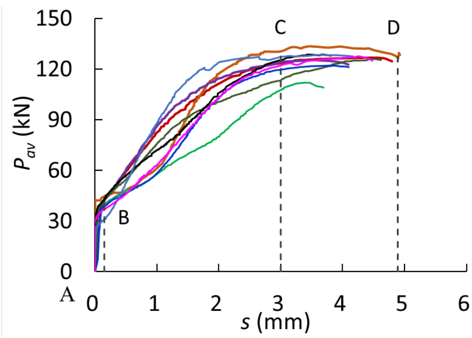


Fig. 4. Shear force versus interface slip curves in the first round of tests

(1) Stage AB. As the applied load was less than the friction between concrete and steel, there was almost no slip produced. At this stage, the bolt connectors show significant shear stiffness. As for S1-S3, the load to overcome the initial interfacial friction depends on the preload applied to the bolt.

(2) Stage BC. Once the applied load exceeded the static friction, the relative slip between the steel beam and the PC slab increased fast until the ultimate shear load was reached.

(3) Stage CD. The shear bearing capacity kept steady or slightly reduced until the bolt shear connectors suddenly fractured.

Table 2 shows the main experimental results of the push-out tests, where P_u is the ultimate shear load of the specimen; P_{av} is the average shear load of each bolt; $P_{u,av}$ and s_u are the average ultimate shear load of each bolt and the interface slip corresponding to $P_{u,av}$; P_i is the shear force corresponding to the inflection point from static friction to sliding friction; R1 and R2 represent the first and second rounds of tests.

It can be seen from Table 2 that P_i in the first round of tests is about $0.283P_{u,av}$. The ultimate shear resistance of the bolts is strongly related to the bolt preload, the bolt diameter, and the bolt diameters. Based on the experimental results, the ultimate shear resistance of the bolt shear connectors can be expressed as:

$$Q_{u1} = 0.5 A_{sc} f_{u,b} + \lambda \mu T \quad (1)$$

Where A_{sc} is the cross-sectional area of the bolt, $f_{u,b}$ is the ultimate tensile strength of the bolt, $\lambda=0.85$ is the reduction factor of the bearing capacity considering the adverse influence of unevenly applied preload, μ is the friction coefficient between the contact surfaces, and T is the bolt preload.

Table 2 The Push-out test results

ID	P_i (kN)		P_u (kN)		$P_{u,av}$ (kN)		s_u (mm)		Q_{u1} (kN)
	R1	**R2**	**R1**	**R2**	**R1**	**R2**	**R1**	**R2**	
S1	37.5	48.6	507.3	513.3	126.8	128.3	4.48	4.51	127.1
S2	42.3	40.0	533.8	538.4	133.5	134.6	3.46	3.03	134.8
S3	38.8	39.9	448.0	431.3	112.0	107.8	3.40	4.01	119.5
S4	41.1	40.0	750.6	738.0	187.7	184.5	4.69	4.31	197.0
S5	37.1	38.0	501.3	526.0	125.3	131.5	3.51	2.57	127.1
S6	37.7	35.9	504.1	540.8	126.0	135.2	4.50	4.84	127.1
S7	38.9	39.1	514.8	506.7	128.7	126.7	3.67	5.53	127.1
S8	37.7	36.0	488.2	561.2	122.1	140.3	3.77	3.18	127.1
S9	29.1	26.0	1026.8	1010.6	128.3	126.3	3.62	3.64	127.1
S10	30.4	30.9	1061.8	947.5	132.7	118.4	5.04	5.12	127.1
S11	35.0	33.3	1020.7	1002.7	127.6	125.3	4.24	5.23	127.1

3 The second round of push-out tests

After the first round of tests, local deformation and several cracks formed in the vicinity of the holes of the PC slabs, see Fig. 3. It can be seen that damage on the PC slabs and the steel beam is limited. After the tests, the components of each specimen were re-assembled using the same specification of bolt shear connectors. Meanwhile, the same loading scheme presented in section 2.2 was used for the second round of tests.

3.1 Experimental observations

In the second round of tests, the interfacial slips between the steel beams and the PC slabs were about 1 mm on average after 25 times of cyclic loading. The failure modes of the second round of tests are similar to those in the first round. The bolts were sheared off, and new cracks appeared on the PC slabs, as shown in Fig. 5. Cracks in the R1 and R2 loading process are marked red and blue, respectively

3.2 Comparison of the shear force versus relative slip curves

The load-slip curves of the re-assembled push-out specimens for the two rounds of tests are depicted in Fig. 6. The dotted lines represent the curves obtained from the second round of tests, and it can be seen that the slips are usually greater than those in the first round of tests under the same shear force in most cases. Therefore, the existing structural damage causes a decrease in the shear stiffness of bolt connectors. The shear force versus slip curves still present a three-stage relationship. The ultimate statistical slip s_{u2} of the bolts was about 0.264 d, being about 1.07 times the ultimate slip s_{u1} (0.247 d) in the first round of tests.

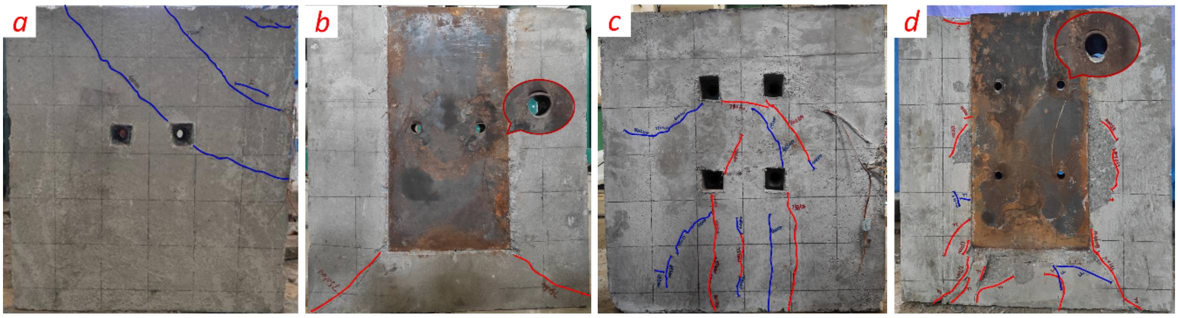


Fig. 5. Typical failure modes of the specimens in the second round of tests:(a) Outer slab surface (S4); (b) Inner slab surface (S4); (c) Outer slab surface (S11); (d) Inner slab surface (S11)

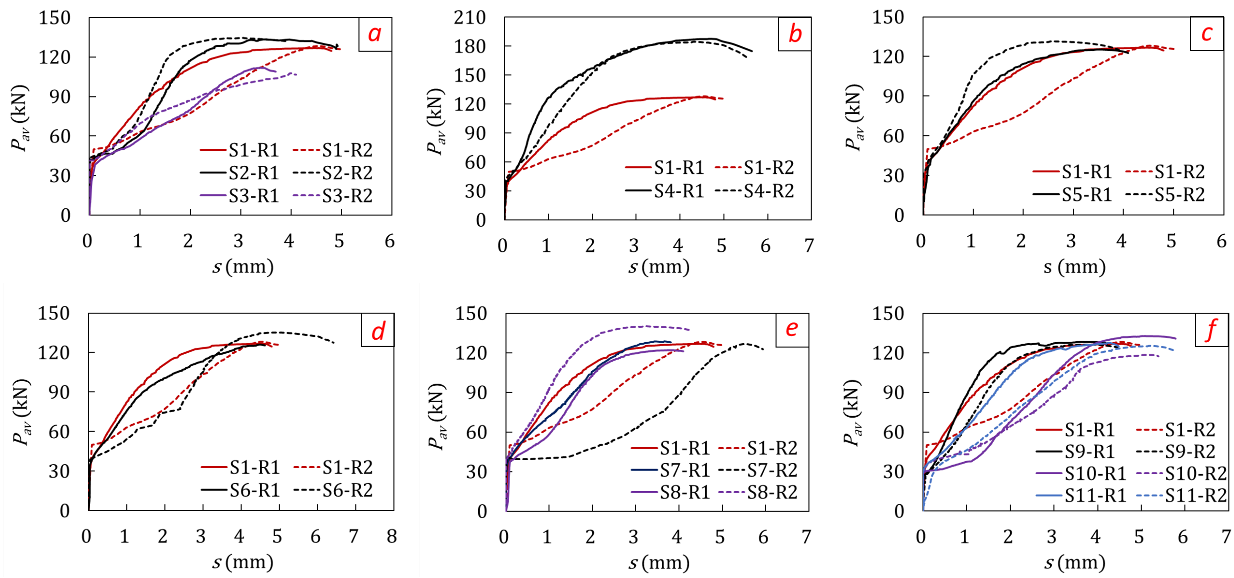


Fig. 6. Comparisons of the Shear force versus relative slip curves: (a) Effect of bolt preload; (b) Effect of bolt diameter; (c) Effect of handling method of the contact surfaces; (d) Effect of constructional measure; (e) Effect of bolt length embedded in the PC slabs; (f) Effect of row spacing of bolts

3.3 Comparison of the shear resistance of bolt shear connectors

Fig. 7 shows the comparison of the shear bearing resistance of the bolts in both rounds of tests. It can be seen that the shear resistance of the replaced bolts shows a bigger scattering compared with those in the first round of testing. Six specimens present higher shear resistance, and the other five give lower shear resistance. Therefore, the existing structural damage generated in the first round of tests does not necessarily cause a decrease in the shear resistance of the replaced bolts. In addition, the influence of the same parameter on the ultimate shear resistance of the replaced bolts shows the same trend as in the first round of tests.

4 Shear performance of the replaced bolt shear connectors

4.1 Ultimate shear resistance

Based on the second round of tests, a reduction coefficient α is introduced to consider the influence of structural damage formed in the first round of tests on the ultimate shear resistance Q_u.

Then, Q_u of the replaced bolt shear connectors with 95% confidence can be obtained:

$$Q_u = \alpha(0.5A_{sc}f_{u,b} + \lambda\mu T) \quad (2)$$

Where α should be taken as 0.9 for the second round of tests.

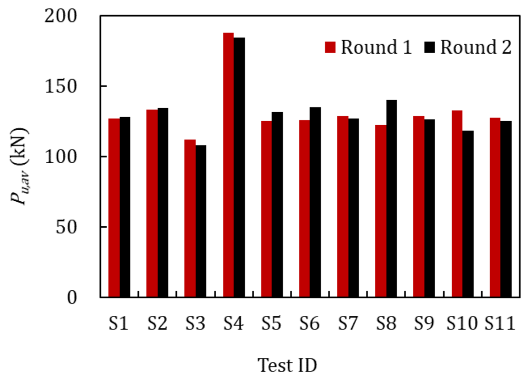


Fig. 7. Comparison of the ultimate shear resistance of the bolts in the two rounds of tests

Table 4 shows different prediction methods for the shear resistance of bolt shear connectors. The statistical indicators of the ratios of the calculation results to $P_{u,av}$ obtained in the second round of tests are given in Table 5. It can be seen that the calculation values by Eq. (2) are most consistent with the experimental results. The calculation values and the test results are plotted in Fig. 8. In summary, Eq. (2) is most suitable to assess the shear resistance of the replaced bolt shear connectors.

Table 4 Calculation methods for the shear resistance of bolt shear connectors

Ref.	Calculation equations
EC 4 [11]	$Q_u = 0.6A_{sc}f_u$
CAN/CSA S16-01 [12]	$Q_u = 0.8A_{sc}f_u$
ANSI/AISC 360-10 [13]	$Q_u = 0.75A_{sc}f_u$
Kwon [14]	$Q_u = 0.5A_{sc}f_u$

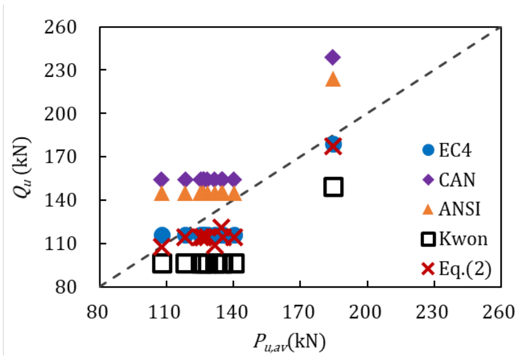


Fig. 8 Comparison of the experimental and calculation results

Table 5 Error analysis of different calculation methods for the second round of tests

Items	$\frac{Q_{EC4}}{P_{u,av}}$	$\frac{Q_{CAN}}{P_{u,av}}$	$\frac{Q_{ANSI}}{P_{u,av}}$	$\frac{Q_{Kwon}}{P_{u,av}}$	$\frac{Q_{Eq.(2)}}{P_{u,av}}$
Average	0.92	1.22	1.15	0.77	0.91
Variance	0.0048	0.0086	0.0075	0.0033	0.0028
Standard deviation	0.0694	0.0925	0.0867	0.0578	0.0533
Variation coefficient	0.0755	0.0755	0.0755	0.0755	0.0588

4.2 Shear stiffness of the replaced bolt shear connectors

According to Fig.6, the shear force-slip curves can be simplified by a bilinear model, as shown in Fig. 9. The red and black solid lines are the simplified models for the first and second rounds of tests, respectively. The mathematical model can be expressed in Eq. (3), and the secant shear stiffness k_2 of the replaced bolt shear connectors is given in Eq. (4).

$$P = \frac{(Q_u - P_i)s}{s_u} + P_i \text{ , } 0 \le s \le s_u \tag{3}$$

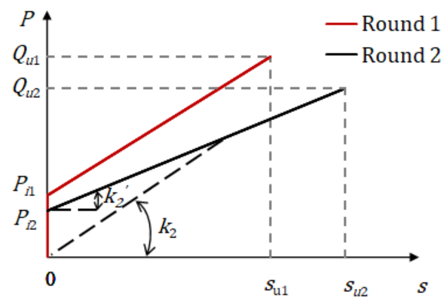


Fig. 9 The simplified model of P - s relationship for bolt shear connectors.

$$k_2 = \frac{P}{s} = \frac{Q_u - P_i}{s_u} + \frac{P_i}{s} = k_2' + \frac{P_i}{s}, 0 < s \le s_u \tag{4}$$

Where $Q_u = 0.9Q_{u1}$ is the ultimate shear resistance of the replaced blot shear connectors; s_u is the slip corresponding to Q_u.

5 Conclusions

Two rounds of push-out tests were conducted on the same batch of push-out specimens with bolt connectors. The following conclusions can be drawn:

(1) The damage caused by the first round of tests had no significant effect on the failure modes of the re-assembled push-out specimens. The typical failure modes for the two rounds of tests are bolt fractures due to shear force. The same design parameters had the same influence on the shear resistance of bolt shear connectors in both rounds of tests.

(2) The existing structural damage generated in the first round of tests does not necessarily cause a decrease in the shear resistance of the replaced bolts. Meanwhile, the interfacial slip tends to increase under the same load in most cases because of the change in the status of contact surfaces and the damage near the holes generated in the first round of tests.

(3) A calculation method was proposed to assess the shear resistance of the replaced bolt shear connectors based on the experimental results. The calculated values were in good agreement with the experimental results. In addition, the simplified shear force-slip model for the replaced bolt connectors was developed.

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