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Deterioration in strength of studs based on two-parameter fatigue failure criterion

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Abstract. In the concept of two-parameter fatigue failure criterion, the material fatigue failure is determined by the damage degree and the current stress level. Based on this viewpoint, a residual strength degradation model for stud shear connectors under fatigue loads is proposed in this study. First, existing residual strength degradation models and test data are summarized. Next, three series of 11 push-out specimen tests according to the standard push-out test method in Eurocode-4 are performed: the static strength test, the fatigue endurance test and the residual strength test. By introducing the “two-parameter fatigue failure criterion,” a residual strength calculation model after cyclic loading is derived, considering the nonlinear fatigue damage and the current stress condition. The parameters are achieved by fitting the data from this study and some literature data. Finally, through verification using several literature reports, the results show that the model can better describe the strength degradation law of stud connectors.

Keywords: steel-concrete composite bridge; stud connector; fatigue; residual strength; two-parameter fatigue failure criterion

1. Introduction

In a steel-concrete composite structure, the shear connectors are the key components to assure shear transfer between the steel profile and the concrete deck (Selvi 2016, Shariati *et al*. 2013, Rodrigues and Laím 2011). Because of their economy and fast application, stud shear connectors are the most common type of shear connectors in steel-concrete composite structures (Kim *et al*. 2014, Shariati 2012). Especially in bridges, these shear studs are subjected to high-cycle fatigue loading by vehicles (Su *et al*. 2014, Zhu and Law 2016). Thus, it is a common practice for the stud connectors in composite bridge beams to be designed for both static strength and fatigue endurance.

According to current national and international standards, the calculations for the strength and endurance of the stud are normally performed independent of each other (BSI 2005b). The design rules for fatigue assume that the connector static resistance remains intact until the fatigue limit is reached. When the fatigue failure occurs, the cracks of the connector propagate quickly at the peak of the cyclic load.

However, irreversible fatigue damage of bridge members or materials is caused by every fatigue loading with greater stress or stress amplitude from the perspective of damage mechanics (Do *et al*. 2015), and it will inevitably result in changes of the mechanical properties of the entire structure. The fatigue failure process could be regarded as a process in which the structure’s strength decreases gradually under cyclic loading, ultimately leading to static failure.

Fig. 1 shows a schematic of the above two types of views. Fig. 1(a) gives the traditional design rules, that is, the strength of the structure remains the same throughout its life. Fig. 1(b) argues that the strength continues to deteriorate with the increasing number of cycle loads, and when the strength reduces to the actual stress, structure failure occurs.

For steel-concrete composite structures, many scholars have directly or indirectly found the strength degradation phenomenon with an increasing fatigue load number in their tests. Mainston and Menzies (1967) found that the static strengths of two push-out test specimens decreased by up to 50% of their original values. Roderick and Ansourian (1976) observed that one of the composite beams failed prematurely after fatigue loading in the test. Oehlers and Coughlan (1986) carried out monotonic static load tests for five push out specimens that were subjected to several fatigue loads. It was concluded that the ultimate bearing capacity was only 51%~73% of the expected strength. Xue *et al*. (2005) investigated the shear performance of stud connectors after cycle loading with C50 concrete. They found that the ultimate bearing capacities for studs with diameters of 13 mm, 16 mm and 19 mm decreased by 31.33%, 19.83% and 13.32% compared with the monotonic loading cases, respectively. Hanswille and Porsch (2014) confirmed the degradation phenomenon from a large number of fatigue push out tests and presented a composite structure design concept based on the whole life cycle.

In view of the degradation phenomenon of studs

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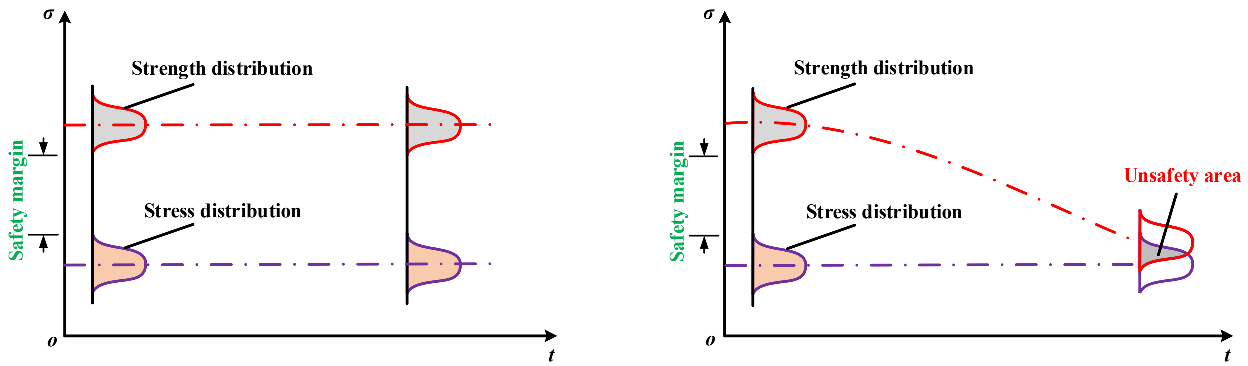


(a) Traditional norm theory (b) Actual situation of the component

Fig. 1 The trend of strength and actual stress with fatigue loading

subjected to fatigue loading, some scholars have put forward the residual strength degradation model of stud connectors, which will be discussed in the second section. It is worth noting that these regression models are all obtained by relevant test data. There is still no theoretical model available to describe the strength degradation of stud connectors under fatigue loading to date

2. Literature review

Even now, systematic reports of the residual strength of studs after fatigue loads are relatively rare. In this paper, the experimental data and models in the few studies are summarized as follows.

Oehlers was one of the earliest researchers. In 1990, Oehlers (1990) reported experimental tests showing that the monotonic strength of stud shear connectors was reduced under fatigue loads. He established a design method for the shear connection that allowed for the reduction in the monotonic strength due to fatigue loads. The studs were 12.7 mm in diameter and 75 mm in height.

The specimens were tested in three series: *S*, *F*, and *M*. In series *S*, the target was to determine the static ultimate strength of the shear connection (P_u). The mean value of the three experimental results was 54.3 kN. The specimens in series *F* were tested to obtain the endurance of the studs. The range of the cyclic load was 0.25 P_u, and the peak of the cyclic load was varied. In series *M*, the residual strength per connector (P_s) was measured after a block of cyclic loads. The range and peak of the cyclic load were held constant while the number of cycles in a block varied. The test details and results are listed in Table 1.

According to the fitting result of the test data, the relationship between the load and the cycle number is shown by the following equation

$$N = N_f(1-\frac{P_s}{P_u}) \tag{1}$$

where N_f is a purely theoretical fatigue life, and N *is* taken as a block of cyclic loads causing the monotonic strength to reduce from P_u to P_s.

In 2004, Bro and Westberg (2004) performed some tests on the EC4 standard push-out specimens. The studs were 22 mm in diameter and 125 mm in height. The compositions of the push-out tests were three static tests, one endurance test and four residual strength tests. The test results are shown in Table 2. For the residual strength, Bro and Westberg put forward a linear equation similar to Eq. (1) established by Oehlers, as shown in Eq. (2)

$$P_s = P_u(1-\frac{N}{N_f}) \tag{2}$$

Table 1 Test results by Oehlers

Specimens	f_c MPa	f_y MPa	f_u MPa	d mm	P_{max} kN	P_{min} kN	ΔP kN	N ($\times10^3$)	P_s kN	P_u kN
S1	60	-	458	12.7	-	-	-	-	-	52.3
S2	60	-	458	12.7	-	-	-	-	-	56.0
S3	60	-	458	12.7	-	-	-	-	-	54.7
F5	60	-	458	12.7	15.6	2.0	13.6	1251.0	-	-
F6	60	-	458	12.7	15.6	2.0	13.6	1507.0	-	-
M1	60	-	458	12.7	15.6	2.0	13.6	250.0	46.1	-
M2	60	-	458	12.7	15.6	2.0	13.6	500.0	43.6	-
M3	60	-	458	12.7	15.6	2.0	13.6	750.0	40.1	-
M4	60	-	458	12.7	15.6	2.0	13.6	1026.0	30.0	-
M5	60	-	458	12.7	15.6	2.0	13.6	1250.0	26.5	-

Table 2 Test results by Bro and Westberg

Specimens	f_c MPa	f_y MPa	f_u MPa	d mm	$P_{\max}$ kN	$P_{\min}$ kN	ΔP kN	N ($\times10^3$)	P_s kN	P_u kN
1	30	350	450	22	-	-	-	-	-	178.8
2	30	350	450	22						180.6
3	30	350	450	22						176.9
4	30	350	450	22	107.5	71.3	36.2	4900	-	-
5	30	350	450	22	107.5	71.3	36.2	400	166.1	-
6	30	350	450	22	107.5	71.3	36.2	1000	161.9	-
7	30	350	450	22	107.5	71.3	36.2	1200	159.6	-
8	30	350	450	22	107.5	71.3	36.2	2000	164.1	-

Table 3 Test results by Ahn

Specimens	f_c MPa	f_y MPa	f_u MPa	d mm	$P_{\max}$ kN	$P_{\min}$ kN	ΔP kN	N ($\times10^3$)	P_s kN	P_u kN
ST-S-A1	30	351	422	16	-	-	-	-	-	97.4
ST-S-A2	30	351	422	16	-	-	-	-	-	100.6
ST-S-A3	30	351	422	16	-	-	-	-	-	97.9
ST-F-A1	30	351	422	16	24.6	4.9	19.7	2120	-	-
ST-F-A2	30	351	422	16	24.6	4.9	19.7	2535	-	-
ST-F-A3	30	351	422	16	24.6	4.9	19.7	2829	-	-
ST-R-A1	30	351	422	16	24.6	4.9	19.7	500	89.6	-
ST-R-A2	30	351	422	16	24.6	4.9	19.7	1000	86.3	-
ST-R-A3	30	351	422	16	24.6	4.9	19.7	1500	77.6	-

In 2007, Ahn *et al*. (2007) also presented push-out tests of stud connections to examine their static and fatigue performance for developing a new bridge deck system. According to the standard push-out specimens in Eurocode-4, the concrete slab in the specimens was 700 mm long, 600 mm wide and 211 mm thick. The concrete slabs were connected to 9 mm steel plates by four welded studs with diameters of 16 mm and heights of 125 mm. Table 3 gives only the experimental data related to the residual strength. As shown in Table 3, the test series contain three static tests, three endurance tests and three residual strength tests.

The test results show that in the cases of 5×10^5 cycles, 1.0×10^6 cycles and 1.5×10^6 cycles, there were approximately 9.2%, 12.5% and 21.3% decreases in strength, respectively. In their study, the authors drew a straight line in the related figure to describe the relationship between the residual strength and the number of fatigue loads.

In 2007, Hanswille *et al*. (2007a) carried out a series of experimental work with standard EC4 push-out specimens to determine the fatigue life and residual strength of headed studs subjected to unidirectional cyclic loading. The specimen used in the push-out test consisted of a 650 mm long HEB260 profile and two 650 mm long, 600 mm wide and 150 mm thick concrete slabs. The slabs were connected to the steel beam by four studs with diameters of 22 mm and heights of 125 mm welded on each side of the beam. Standard bent bars with diameters of 10 mm and 12 mm were used in the concrete slabs as reinforcement.

The specimens were prepared and grouped into six series. Among them, S1~S4 and S5E were subjected to constant amplitude tests to determine the residual strength. In each series, three static and cyclic tests were performed to obtain the mean values of the ultimate load and the fatigue life, respectively. Detailed information is shown in Table 4. The test results indicate that the peak value of cyclic load has a significant effect on the way cracks are formed at the stud foot. Additionally, analytical expressions to predict the residual strength were derived based on the improved Palmgren-Miner damage accumulation rule.

Hanswille established the following equation taking into account the limitations to predict the reduced static strength at a given number of loading cycles 0.54

$$\frac{P_s}{P_u} = 0.74 \cdot \frac{P_{\max}}{P_u} \cdot \left(1 - \frac{\Delta P}{P_{\max}}\right) + 0.54 - 0.04 \cdot \ln\left(\frac{\dfrac{N}{1 - \dfrac{P_{\max}}{P_u}}}{10^{0.1267 - 0.1344 \cdot \frac{P_{\max}}{P_u} \cdot \left(1 - \frac{\Delta P}{2P_{\max}}\right)} - N}\right) \tag{3}$$

where $P_s \in [P_{\max}, P_u]$.

In conclusion, there are two different views regarding the calculation models for the residual static strength of studs after a certain number of cycling loadings. One opinion is that the residual strength has a linear relationship

Table 4 Test results by Hanswille

Specimens	f_c MPa	f_y MPa	f_u MPa	d mm	$P_{\max}$ kN	$P_{\min}$ kN	ΔP kN	N ($\times 10^3$)	P_s kN	P_u kN
S1	44-52	337	448	22	-	-	-	-	-	205
		337	448	22	90.2	49.2	41.0	6200	-	-
		337	448	22	90.2	49.2	41.0	1984	154	-
		337	448	22	90.2	49.2	41.0	5580	129	-
S2	42-45	337	448	22	-	-	-	-	-	184
		337	448	22	130.6	84.6	46.0	1200	-	-
		337	448	22	130.6	84.6	46.0	384	174	-
		337	448	22	130.6	84.6	46.0	840	154	-
S3	53-56	337	448	22	-	-	-	-	-	201
		337	448	22	88.4	38.1	50.3	5100	-	-
		337	448	22	88.4	38.1	50.3	1224	133	-
		337	448	22	88.4	38.1	50.3	3519	123	-
S4	43	337	448	22	-	-	-	-	-	181
		337	448	22	128.5	92.3	36.2	3500	-	-
		337	448	22	128.5	92.3	36.2	1015	181	-
		337	448	22	128.5	92.3	36.2	2520	156	-
S5E	43	337	448	22	-	-	-	-	-	189
		337	448	22	56.7	9.4	47.3	6400	-	-
		337	448	22	56.7	9.4	47.3	3776	111	-
		337	448	22	56.7	9.4	47.3	4672	114	-

with the fatigue life, as shown by the expression derived by Oehlers, Bro and Westberg and Ahn in Eq. (2). Another is that the relationship between the residual strength and the fatigue life is nonlinear, as presented in Eq. (3) derived by Hanswille.

Previous experimental results show that the degradation rate of residual bearing capacity for stud connectors under fatigue loading is slow in the early stage but fast in the later period, which forms a nonlinear process. Therefore, Eq. (2) is not very reasonable. Eq. (3) is nonlinear, but the form is too complicated for engineering applications.

In this paper, stud connectors were selected as the research object. Three series of 11 push-out tests (static tests, fatigue tests and residual tests) were performed. By introducing the concept of "two-parameter fatigue failure criterion," the nonlinear degradation rule of the static strength for the cyclic loaded studs was derived. Furthermore, we established a degradation model to describe the variation of residual strength. The parameters in the model were determined by fitting the test data in this study and previous studies. Finally, the correctness of the model was verified by the experimental data in some literature works.

3. Experimental study

3.1 Test specimens

The specimen dimension and reinforcement of the push test mainly followed Eurocode 4 (BSI 2005a). The concrete grade was C50. The ordinary reinforced bar was HPB 300 grade hot-rolled steel bars with diameter of 10 mm. The steel plate was Q345 which is commonly used in bridges. The I-shaped steel beam was welded using a 14 mm thick steel plate, and its size was HW250(H) × 250(B) × 14(t1) × 14(t2). The beam was 500 mm long. On both sides of the beam, two studs with diameters of 13 mm and lengths of 70 mm were welded. The material of the stud was ML-15. The measured diameter was 12.96 mm, and the measured length was 69.86 mm. The concrete flange plate was composed of 2 blocks of C50 concrete with sizes of 450 mm × 500 mm × 150 mm, where two layers of reinforced networks were configured. The specific dimension is shown in Fig. 2. Fig. 3 shows the pouring process and the completion of the specimens. Furthermore, when the concrete was poured, oil needed to be brushed on the surface between the steel beam and the concrete to avoid a bonding effect on the test results.

3.2 Material test

Before the push-out tests, the mechanical properties of materials related to the specimens were tested first. For the concrete, a standard cube block of 150 mm × 150 mm × 150 mm in size was selected from the same concrete materials and cured under a condition similar to that of the specimens. The material measurement was performed on the day of testing. The same condition was still available for the studs and the steel. The material test process is shown in Fig. 4. The results are listed in Table 5.

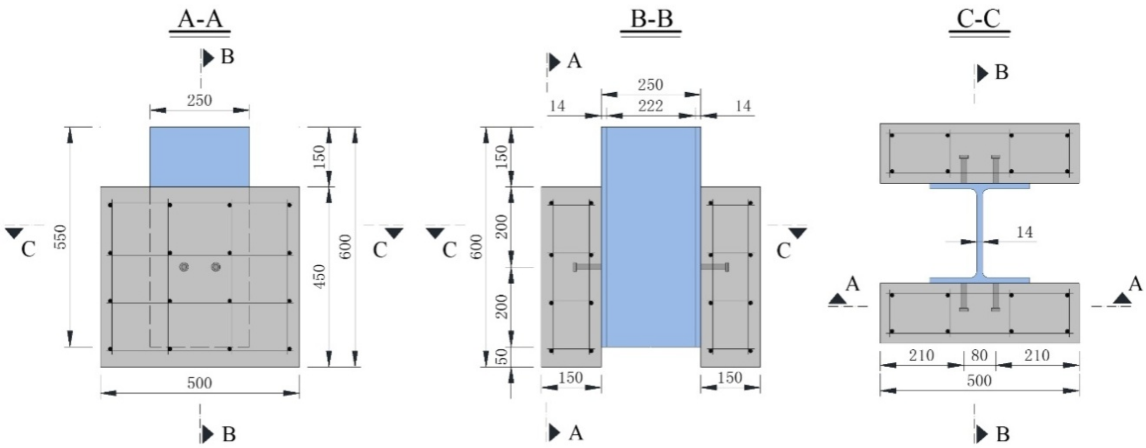


Fig. 2 The dimension of the push-out specimen (Unit: mm)

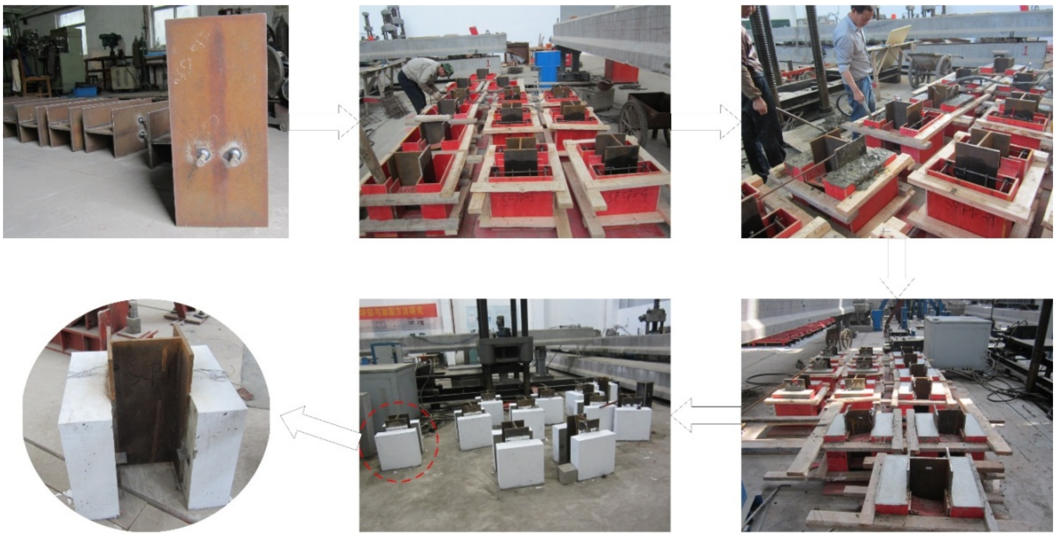


Fig. 3 The pouring process and completion of the specimens

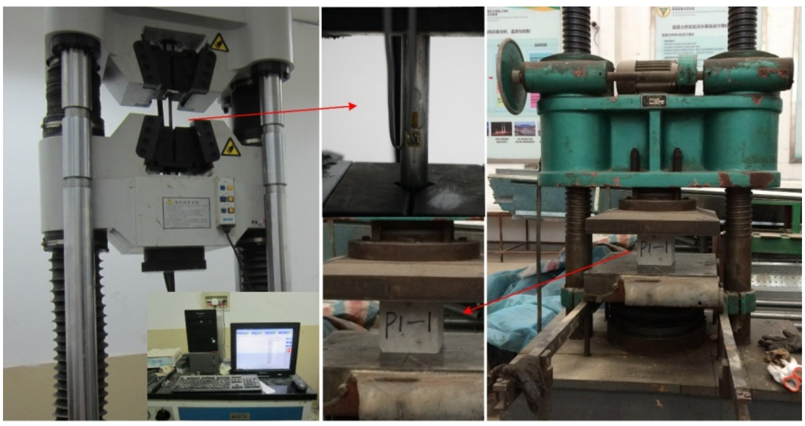


Fig. 4 The material test process

3.3 Test program

The test was performed on a multifunctional structural test system equipped with a loading actuator with 500 kN capacity for static and dynamic loading. To guarantee that the specimen loading surface was horizontal, an adjustable base was specially designed at the bottom of the specimen (Zou 2016). The tests were divided into three series: static tests, fatigue tests and residual tests.

(1) Static strength test

First, pre loading was conducted before the formal

Table 5 Mechanical properties of the specimen materials

Material type	Series No.	Average value of cube strength	Average elastic modulus	Material type	Series No.	Average elastic modulus	Average yield strength	Average ultimate strength
		N/mm^2	N/mm^2			N/mm^2	N/mm^2	N/mm^2
Concrete	S-1	59.7	35900	Stud	M-1	2.0×10^5	442	525
	S-2	60.4	36000	I-beam	G-1	2.1×10^5	352	495
	S-3	59.1	35800					

(a) The loading device (b) The specimen loading process

Fig. 5 The loading device and the specimen loading process

Table 6 Fatigue loading parameters for stud push-out specimens

Test	Specimens	P_{max}/P_u	P_{min}/P_u	$\Delta P/P_u$	N ($\times10^3$)
(1) Static	SCP-1	-	-	-	-
	SCP-2	-	-	-	-
	SCP-3	-	-	-	-
(2) Fatigue (Endurance)	FCP-1	0.60	0.35	0.25	-
	FCP-2	0.60	0.35	0.25	-
	FCP-3	0.60	0.35	0.25	-
(3) Fatigue (Residual strength)	SFCP-1	0.60	0.35	0.25	500
	SFCP-2	0.60	0.35	0.25	1000
	SFCP-3	0.60	0.35	0.25	1500
	SFCP-4	0.60	0.35	0.25	2000
	SFCP-5	0.60	0.35	0.25	2500

loading. The loading was applied to 0.4 times the elastic limit load (approximately 50 kN) and then unloaded. Next, the formal loading was conducted using the multi-stage method. Each load increment was 20 kN, and the loading rate was 10 kN/min. When the load reached 60% of the ultimate load, it was changed to a displacement control, which was applied until the structure failed at a speed of 0.5 mm/min. This series included three specimens, the numbers of which were SCP-1 to SCP-3.

(2) Fatigue endurance test

The preloading method was the same as the static loading method. Fatigue loading was applied by a sine wave with a loading frequency of 4 Hz. This series consisted of three specimens, i.e., FCP-1 to FCP-3. The upper and lower limit values and the loading amplitude are presented in Table 6. Static tests were conducted at cyclic numbers of 0, 10000, 30000, 50000, 100000, 250000, 500000, 1000000, 1500000, 1750000, 2000000, 2250000 and 2500000. The load was taken as the upper fatigue limit value.

(3) Residual strength test

The residual test is a static failure test after a block of fatigue loading. The loading mode is cyclic loading first and then monotonic loading, which is similar to that above, respectively. The series contained five specimens, and the numbers were SFCP-1~SFCP-5. The test parameters are shown in Table 6.

Fig. 5 shows the loading device and the specimen loading process. A displacement sensor and a force sensor were fixed on the actuator to capture the displacement and load at the loading end during the test. In the stud locations, a displacement gauge was arranged to collect the relative slip between the concrete slab and the steel plate.

3.4 Test results

For the push-out tests, the failure modes of the specimens are generally divided into two categories due to the different relative strength grades between the concrete and the studs: one is the shear failure of the studs; the other is the local crushing or splitting of the concrete. In this experiment, the materials were all high-strength concrete of C50 grade. The results of 3 series of push out tests all showed stud shear failure. The concrete flange plate was intact without obvious cracks in addition to local crushing at the root of the studs. Fig. 6 demonstrates the static failure modes.

The results of three series of push tests are listed in Table 7. As shown in Table 7, the ultimate bearing capacities of 3 studs in the first series were almost the same, with less than 4% relative error. The mean ultimate strength was 70.2 kN. In the fatigue test series, the results had a relatively large dispersion. Among them, the FCP-2 test specimen was deflected in the loading process, leading to failure in advance, which was not adopted. The mean fatigue life of the studs from the FCP-1 and FCP-3 test results was 2.68×10^6. In the third series, the results showed that the ultimate strength of the studs continued to decrease with the increase of the cyclic number, and the degradation rate was initially slow and then fast. The residual strength was only 68.8 kN at the cyclic number of 500000. The residual strength was 44.9 kN at the cyclic number of 2500000, which was 64% of the ultimate value. It showed an obvious decrease in strength.

Table 7 Average test results per stud

Specimens	f_c MPa	f_y MPa	f_u MPa	d mm	P_{max} kN	P_{min} kN	ΔP kN	N ($\times 10^3$)	P_s kN	P_u kN
SCP-1	59.7	442	525	12.96	-	-	-	-	-	68.6
SCP-2	59.7	442	525	12.96	-	-	-	-	-	70.7
SCP-3	59.7	442	525	12.96	-	-	-	-	-	71.3
FCP-1	59.7	442	525	12.96	168	98	70	2742	-	-
*FCP-2	59.7	442	525	12.96	168	98	70	1753	-	-
FCP-3	59.7	442	525	12.96	168	98	70	2618	-	-
SFCP-1	59.7	442	525	12.96	168	98	70	500	68.8	-
SFCP-2	59.7	442	525	12.96	168	98	70	1000	63.9	-
SFCP-3	59.7	442	525	12.96	168	98	70	1500	58.3	-
SFCP-4	59.7	442	525	12.96	168	98	70	2000	54.1	-
SFCP-5	59.7	442	525	12.96	168	98	70	2500	44.9	

*Note: FCP-2 test specimen was deflected in the loading process, leading to failure in advance

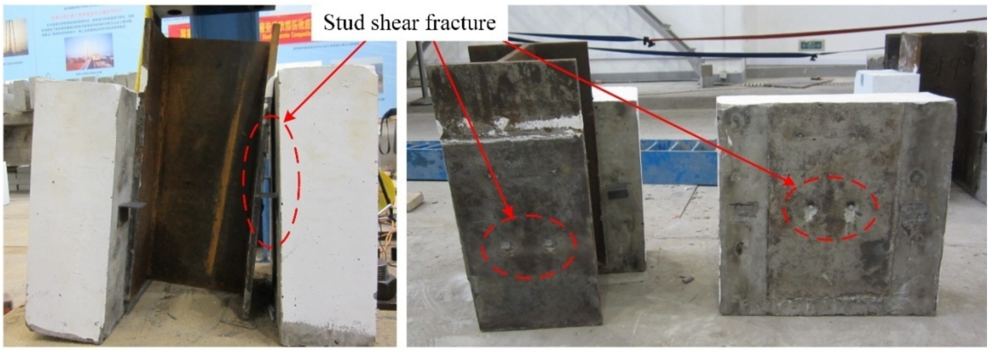


Fig. 6 Failure modes of the specimens

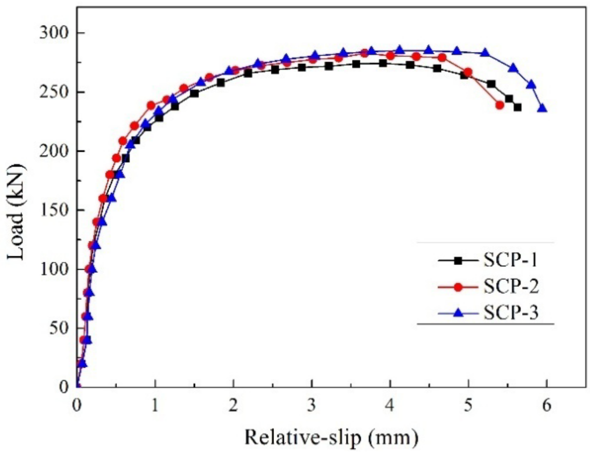
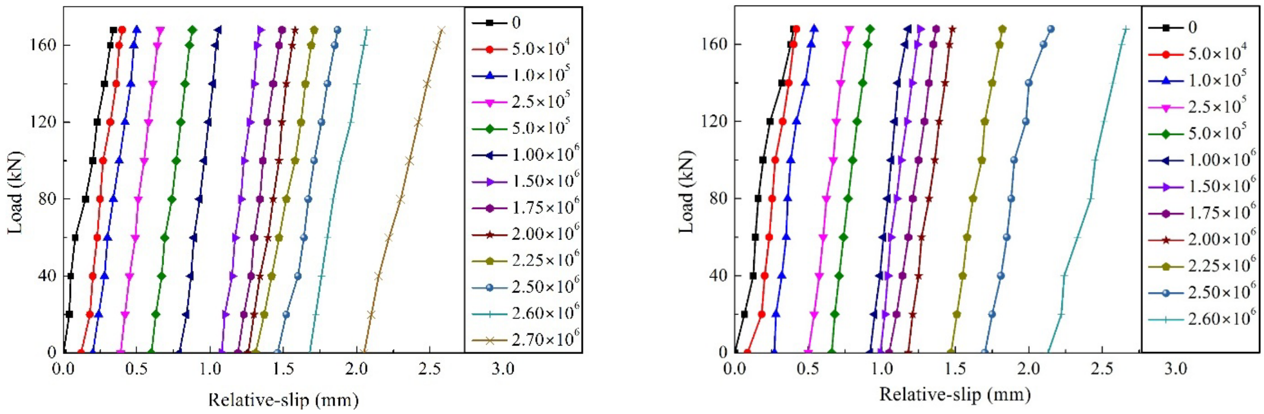


Fig. 7 Load-relative slip curves of stud push-out specimens from static tests

The load-relative slip curves for shear connectors are used to extract the mechanical properties of the connector. Fig. 7 presents the load-relative slip curves of stud push-out specimens from static tests. It can be observed that three load-relative slip curves under monotonic loading all exhibited similar characteristics and three significant stages the elastic stage, the elastoplastic stage and the plastic stage.

Fig. 8 shows the load-relative slip curves of FCP-1 and FCP-3 specimens under different loading cycles. It indicates that there is a large difference between the load-relative slip curves in the initial loading stage and the later loading stage. In the initial stage of fatigue loading, the load-relative slip curves fluctuate and are nonlinear. This is mainly because the contact between the stud and the surrounding concrete is not close in the early fatigue loading stage. With the increase of fatigue loading times, the void between the stud and the surrounding concrete is eliminated, and the load-relative slip curves have a relatively stable slope. In the later loading stage, the stud stiffness gradually deteriorates due to the fatigue damage, which makes the load-relative slip curves nonlinear again.

Fig. 9 presents the load-relative slip curves of the third series of specimens after different numbers of fatigue loading. As shown in Fig. 9, apart from the ultimate bearing capacities of the specimens decreasing with the increasing number of cyclic loading, the stiffness in the elastic stage and the ultimate slip from the load-relative slip curves decrease with the increase of cycle number. This indicates that the ductility of stud connectors decreases gradually, and the failure mode tends to change from ductile failure to brittle fracture with the increase of the number of fatigue loading.



(a) FCB-1 (b) FCB-3

Fig. 8 The load-relative slip curves at different load cycle numbers

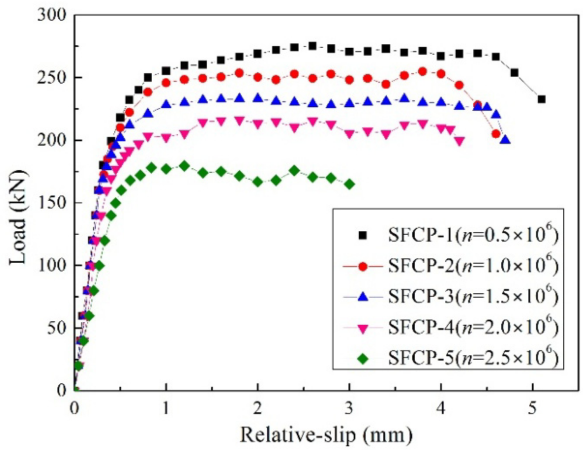


Fig. 9 Load-relative slip curves of stud push-out specimens under different fatigue loading numbers

4. Strength degradation model of stud connectors based on the two-parameter fatigue failure criterion

4.1 Two-parameter fatigue failure criterion

Fatigue failure is a phenomenon where the damage of the material is accumulated under cyclic loading, and the strength is decreased until the material cannot resist the external load. The destruction is provided with a dynamic behaviour, and the damage amount only indicates the state of the material and is not related to the fracture. If no force continues to be applied, the damage will no longer occur. Therefore, destruction is the result from the action of force under a certain damage state. A certain damage state has a unique corresponding critical stress. In the case of a certain damage state, the material failure depends on the stress condition. If the damage is small, it needs a large stress to cause failure, and vice versa.

In fracture mechanics (Chakherlou *et al*. 2012), the stress intensity factor $K(\sigma, a)$ is used as the strength parameter, in which there are two parameters, i.e., stress σ and crack length a. The criterion is that when K reaches its critical value K_c, the material fails. Obviously, the fracture is determined by two variables (σ and a). Stress σ represents the external factor, and crack length a represents an internal factor. Hence, we call this failure criterion the "two-parameter criterion."

In this study, the relationship between the residual strength and fatigue damage was analysed based on the "two-parameter fatigue failure criterion". Furthermore, a nonlinear strength degradation model was established to describe the variation of the residual strength.

4.2 Nonlinear fatigue damage and residual strength of materials

Most of the existing fatigue theories consider that the damage is absolutely equivalent and objective (Richart and Newmark 2015). The Miner rule suggests that the damage is uniform. However, the amount of actual damage is nonlinear with the number of cycles. The definition of nonlinearity was first proposed by Marco and Starkey (1954). Later, Manson and Halford (1981) developed a different damage curve method. In the Manson model, the damage caused within one cycle is defined by the current damage degree of the material and current stress level, as shown in Eq. (4).

$$\begin{cases} D = \left(\dfrac{1}{N_i}\right)^{q_i} \\ q_i = BN_i^{\mu} \end{cases} \tag{4}$$

where N_i denotes the fatigue life under the ith level stress action. B and μ are material constants.

Fatigue cumulative damage $D_{(A)}$ refers to the total amount of damage before N number of cycles. According to the reversibility and randomness of damage (Zhu *et al*. 2013), its curve is usually measured through the test. However, the measurement in actual engineering is difficult. We can assume the function form as follows

$$D_{(A)} = (n / N)^c \tag{5}$$

where c is a damage index.

The residual strength of the material $\sigma_R(n)$ refers to the ability to resist the external load under a certain number of loading cycles. Based on the two-parameter fatigue failure criterion, it is related to the damage degree (generally characterized by number of cycles n) and stress level σ. That is

$$\sigma_R(n) = f(n, \sigma) \tag{6}$$

For metallic materials, in the early stage of fatigue loading, the defects caused by fatigue loading (such as dislocation, slip, voids, etc.) have little influence on the strength of the materials, and the strength degradation rate is very slow. However, in the later period, especially when the fatigue cycle number ratio is close to 1, internal continuous initiation and propagation of cracks lead to the reduction of the effective bearing area. This further results in a rapid decrease of the residual strength and failure occurs finally.

In general, the residual strength degradation curve has the following characteristics.

(1) $\sigma_R(0) = \sigma_f$ This means the initial residual strength $\sigma_R(0)$ is equal to the static ultimate strength σ_f.
(2) $\sigma_R(N) = \sigma_{\max}$ When the cyclic number reaches fatigue life N, the residual strength $\sigma_R(N)$ is equal to the peak fatigue load $\sigma_{\max}$.
(3) $\frac{d\sigma_R(n)}{dn}|_{n\to 0} \to 0$ At the beginning of the fatigue loading, the degradation rate is zero.
(4) When n approaches N, the material has the characteristics of "sudden death."

To satisfy the characteristics of residual strength, the nonlinear strength degradation model can be written as follows (Khoramishad and Crocombe 2011)

$$\sigma_R(n) = \sigma_R(0) - [\sigma_R(0) - \sigma_{\max}](n/N)^c \tag{7}$$

where $\sigma_R(n)$ is the residual strength under the nth number of cycle. $\sigma_R(0)$ is the static strength of the intact material. $\sigma_{\max}$ is the maximum cyclic stress.

c mainly depends on the internal damage development of the material and $c > 1$ according to the fourth characteristic. Therefore, the function of c is defined as follows

$$c = \exp[\gamma (n/N)^\alpha] + 1 \tag{8}$$

where γ is a material coefficient related to function $\sigma_R(n)$. α is the stress level coefficient, and $\alpha = \sigma_{\max} / \sigma_f$.

Combining Eqs. (7) and (8), the nonlinear residual strength equation can be obtained.

$$\sigma_R(n) = \sigma_R(0) - [\sigma_R(0) - \sigma_{\max}](n/N)^{\exp[\gamma (n/N)^\alpha]+1} \tag{9}$$

According to the view of thermodynamics, the damage is irreversible. Hence, the residual strength degradation model should be a monotonically decreasing function. The first order derivation of Eq. (9) is less than zero, which proves that the strength degradation model can meet the irreversible conditions.

At the same time, according to the physical condition of fatigue damage, $\sigma_R(n)$ needs to satisfy the degradation law of "slow first and fast later." Therefore, it requires the second order derivation of Eq. (9) to be less than zero. The calculation result proves this.

4.3 The degradation model of residual bearing capacity for stud

For stud connectors, the bearing capacity is proportional to the material strength. According to the residual strength degradation model (see Eq. (4)), the model for the residual bearing capacity degradation for the stud is given in Eq. (10).

$$P_s(n) = k\left(\sigma_R(0) - [\sigma_R(0) - \sigma_{\max}](n/N)^c\right) \tag{10}$$

Considering the boundary conditions, when $n = 0$, $P_s(0) = P_u = k\sigma_R(0)$, while if $n = N$, $P_s(n) = P_{\max} = k\sigma_R(0) - [k\sigma_R(0) - k\sigma_{\max}]$. Here, Eq. (10) can be changed as follows

$$P_s(n) = P_u - [P_u - P_{\max}](n/N)^c \tag{11}$$

To facilitate the analysis and fitting of the subsequent experimental data, the non-dimensional treatment of formula (11) is performed. P_s/P_u is marked as λ. n/N is taken by β. Then, the model can be written as

$$\lambda(n) = 1 - [1-\alpha]\beta^{\exp[\gamma\beta^\alpha]+1} \tag{12}$$

To determine the parameters of the formula, data from Oehlers and Hanswille's tests and data in this study were selected, which is listed in Table 2. The related parameters of the studs were normalized. Finally, we can obtain several groups of degradation strength data for stud connectors with variables α and β.

According to the physical meaning of the residual strength model, the data with different α satisfied such conditions: $\beta = 0$, $\lambda = 1$ and $\beta = 1$, $\lambda = \alpha$. The variable region of α is 0 to 1. The range of λ is α to 1.

MATLAB software was used to fit the equation with two variables to determine the parameter γ. The determination coefficient of the result was $R^2 = 0.958$, which indicated that the equation fitted well, as shown in Fig. 10. $\gamma = -1.228$. Then, the final equation was obtained.

$$\lambda(n) = 1 - [1-\alpha]\beta^{\exp[-1.228\beta^\alpha]+1} \tag{13}$$

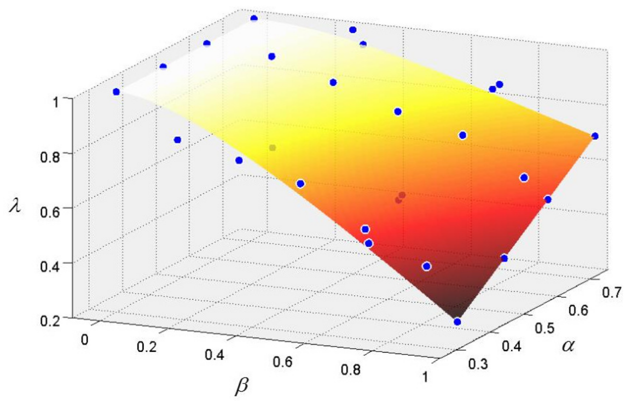


Fig. 10 Surface fitting results

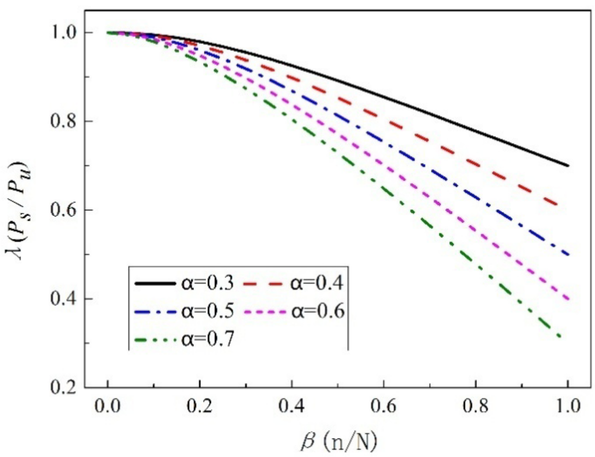


Fig. 11 The degradation curve of the stud with different α

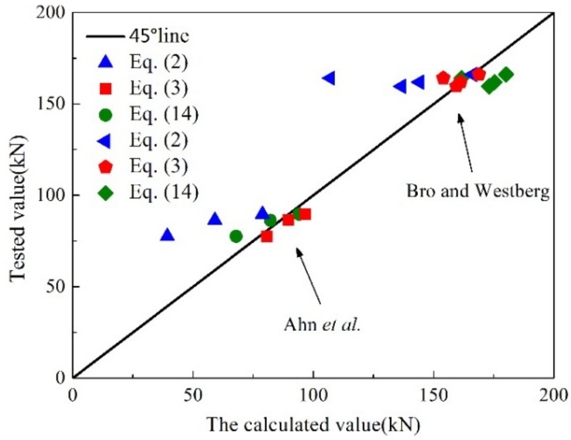


Fig. 12 Comparison of the calculated values and the test data of the residual capacity

$$P_s(n) = \lambda(n) \cdot P_u \quad (14)$$

Fig. 11 shows the degradation curve of the stud which was calculated by Eq. (12) for different α. As shown in Fig. 11, the bearing capacity degradation curve of the stud shows the characteristics of slow first and fast later. Taking the curve of α = 0.3 as an example, during the first 20% of the fatigue life, the capacity was reduced only by 6.6%. However, in the final 20% of the fatigue life, the reduction was approximately 17.8%, which was 2.7 times of the former value.

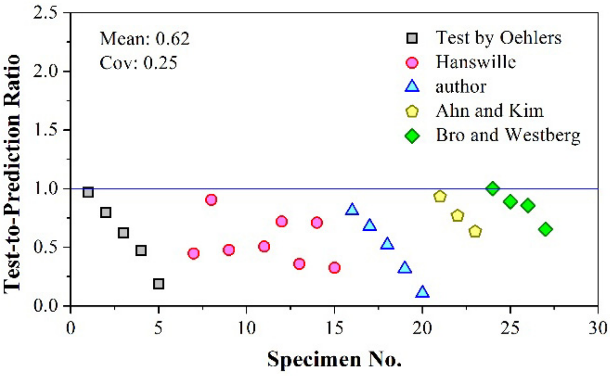
5. Model verification

To verify the correctness and universality of the exponential regression model proposed in this paper, the results from the proposed equation were compared with those from Eqs. (2)-(3). The data are shown in Table 2 and Table 3. The comparison result is shown in Fig. 12.

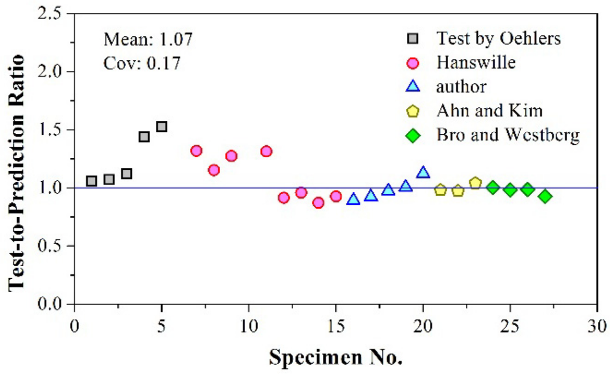
As shown in Fig. 12, the calculated values from the proposed equation in this study and Eq. (3) are in good agreement with the test data, while the result from Eq. (2) deviates relatively from the test value.

Table 8 Summary of test data for the stud connectors

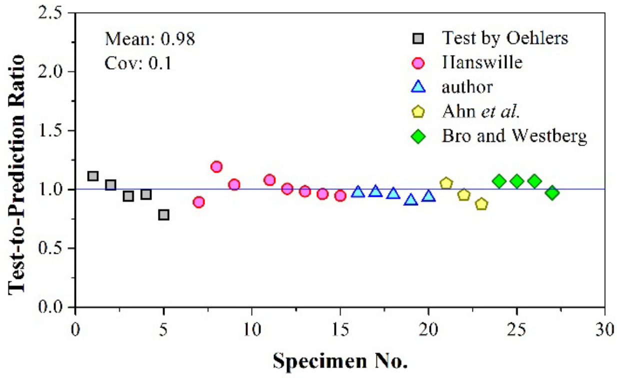
Data sources	Ultimate capacity P_u/kN	Fatigue life $N/10^3$	Fatigue loading parameters		Cyclic number	Residual capacity	Normalization	
			$\Delta P/P_u$	P_{max}/P_u	$n/10^3$	P_s/kN	n/N	P_s/P_u
Oehlers's data	54.3	1379	0.25	0.30	250	46.1	0.18	0.85
	54.3	1379	0.25	0.30	500	43.6	0.36	0.8
	54.3	1379	0.25	0.30	750	40.1	0.54	0.74
	54.3	1379	0.25	0.30	1026	30.0	0.74	0.55
	54.3	1379	0.25	0.30	1250	26.5	0.91	0.49
Hanswille's data	189	6400	0.25	0.30	1216	111	0.19	0.59
	189	6400	0.25	0.30	4672	114	0.73	0.6
	205	6200	0.2	0.44	1984	154	0.32	0.75
	205	6200	0.2	0.44	4340	129	0.70	0.63
	201	5100	0.25	0.44	1224	133	0.24	0.66
	201	5100	0.25	0.44	3519	123	0.69	0.61
	184	1200	0.25	0.71	384	174	0.32	0.95
	184	1200	0.25	0.71	840	154	0.7	0.84
	181	3500	0.2	0.71	1015	181	0.29	1.00
	181	3500	0.2	0.71	2520	156	0.72	0.86
Data in this study	70.2	2705	0.25	0.60	500	68.8	0.19	0.98
	70.2	2705	0.25	0.60	1000	63.9	0.37	0.91
	70.2	2705	0.25	0.60	1500	58.3	0.56	0.83
	70.2	2705	0.25	0.60	2000	54.1	0.75	0.77
	70.2	2705	0.25	0.60	2500	44.9	0.93	0.64



(a) Predictions by Eq. (2)



(b) Predictions by Eq. (3)



(c) Predictions by Eq. (14)

Fig. 13 Test-to-prediction ratio obtained from three formulas

For further validation of the three formulas, the results of all experiments in this paper were compared with the calculation results of the formulas. Fig. 13 shows the test-to-prediction ratio obtained from the three formulas. It was found that the deviation of Eq. (2) was the biggest, and the mean test-to-prediction ratio was only 0.62. In Fig. 13(b), the mean test-to-prediction ratio was larger than one, which showed that the calculation result of the formula was not safe. In Fig. 13(c), the test data were very close to the prediction values, and the stability was good. It can be observed that the proposed degradation model based on the two-parameter fatigue failure criterion can well describe the variation of the residual strength for the studs with the number of fatigue loads.

6. Conclusions

In this paper, the residual strength and the strength degradation model of stud connectors were investigated. Three series of standard push-out test in Eurocode-4 (static tests, fatigue tests and residual tests) were performed. The existing data of the stud connection were analysed and summarized. The failure modes of the specimens were all stud shear failure. The following conclusions are obtained:

- The ultimate strength of the stud decreased with the increase of the cyclic number, and the degradation rate was slow first and then fast. Remarkably, the residual strength was only 64% of the ultimate value at the cyclic number of 2500000. More attention should be given in the actual bridge design.
- To make the residual strength test data under different loading conditions uniform and comparable, the residual capacity and the fatigue life were normalized and fitted with double variables.
- A strength degradation model was derived based on the two-parameter fatigue failure criterion in this paper. The form was simple and reasonable with strong applicability and stability. The model can better describe the variation of the residual strength for stud shear connectors under fatigue loads. A number of experimental data from the literature verified the correctness of the model.

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