Hindawi
Advances in Civil Engineering
Volume 2018, Article ID 4780759, 12 pages
https://doi.org/10.1155/2018/4780759

Research Article

Push-Out Tests on Large Diameter and High Strength Welded Stud Connectors

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Received 13 June 2018; Accepted 27 August 2018; Published 17 September 2018

Academic Editor: Libo Yan

In recent years, steel-concrete composite cable-pylon anchorages are increasingly employed in the construction of long-span cable-stayed bridges, especially in China. Welded stud connectors, typically 19 mm and 22 mm in diameter, are usually densely arranged at the interface between the steel anchorage box and the wall of the concrete pylon to transfer the huge cable force into the concrete pylon. However, dense welded stud arrangements at the interface have some disadvantages, such as the shear strength reduction of stud connectors and the difficulties in arranging reinforcements and pouring pylon concrete. Larger diameter and higher strength welded studs may be excellent alternatives since they both could increase the shear strength of a single stud connector and thus reduce the required number of welded studs. In this paper, push-out tests were implemented on four groups of welded stud connector specimens, including conventional 22 mm welded studs, 22 mm welded studs with much higher strength, 25 mm welded studs, and 30 mm welded studs. The shear strength, shear stiffness, and ductility of these welded stud connectors were investigated and compared with the predictions by the equations recommended in existing design codes. The results show that the shear strength of welded stud connectors could be conservatively determined by Eurocode 4, while AASHTO LRFD will produce a suitable estimation. The load-slip relationships proposed by Ollgaard and Buttry can be used to predict the load-slip curves of large diameter and high strength welded stud connectors.

1. Introduction

Welded studs are the most practical connectors for achieving the composite action between steel girders and concrete slabs owing to their rapid welding technique and outstanding mechanical performance. In composite bridge superstructures, the commonly used welded studs are 19 mm or 22 mm in diameter since these welded studs are adequate to realize the full shear connection between steel girders and concrete slabs. The mechanical performance of welded studs with 19 mm and 22 mm in diameter has been investigated by many researchers. Some representative research includes the following: Ollgaard et al. [1] proposed an equation to compute the shear strength of welded studs and a curve to describe the load-slip relationship. Oehlers and Coughlan [2] put forward an empirical equation to calculate the shear stiffness of welded stud connectors. Lam [3] performed 72 static push-out tests on welded studs to determine the shear strength of welded stud connectors embedded in hollow-core slabs. Pallarés and Hajjar [4, 5] reviewed a large number of push-out and pull-out tests on welded stud connectors and proposed formulas for the limit states of welded stud connectors subjected to shear force, tension force, and combined tension and shear force. Lin et al. [6] also investigated the behavior of welded stud connectors subjected to combined tension and shear loads, and an improved shear-tension interaction strength equation was recommended.

In addition to the application in steel-concrete composite girders, welded stud connectors could also be employed in steel-concrete hybrid girders [7], steel bridges with integral abutment [8], the cable-pylon anchorage zone of cable-stayed bridges, and so on. In these composite/hybrid structures, a part of welded stud connectors will bear combined tension and shear loads, and their shear force would be much larger than in the conventional steel-concrete girders. For the composite cellular beams, regular circular openings are produced in the web of the steel beam by cutting and re-welding the steel sections, or by cutting circular openings in fabricated steel sections for the distribution convenience of the circular service ducts in the building structures [9]. The openings in the composite cellular beams could not only result in much larger shear force in the steel-concrete connectors compared with the conventional composite beams [9] but also cause nonnegligible pull-out tensile force in the connectors [10]. In steel-concrete composite cable-pylon anchorage structures, welded stud connectors are commonly employed to connect the steel anchor box to the concrete pylon [11, 12], as shown in Figure 1(a). The mechanical behavior of welded stud connectors in the cable-pylon anchorage zone, as shown in Figure 1(b), has great differences compared with that at the steel-concrete interface of composite girders. The vertical component of the cable force is transferred from the steel anchor box to the wall of the concrete pylon through welded stud connectors, and this force could be extremely large in long-span cable-stayed bridges. Meanwhile, a part of welded stud connectors are under combined shear and tension loads due to the local bending moment caused by the eccentricity of the cable force at the steel-concrete interface. Therefore, welded stud connectors are commonly densely arranged at the steel-concrete interface of the cable-pylon anchorage structure to ensure the safety of the anchorage system. However, dense welded stud arrangements at the interface have some disadvantages, such as the shear strength reduction of stud connectors due to the smaller stud spacing [13, 14] and the difficulties in arranging reinforcements and pouring pylon concrete.

To ensure the adequate spacing among the welded studs at the steel-concrete interface of the cable-pylon anchorage system, much larger diameter and higher strength welded studs are two alternatives since they both could increase the shear strength of a single stud connector and thus reduce the required number of welded studs. Currently, almost all the push-out tests were conducted on welded stud connector specimens with diameter equal to or less than 22 mm. Only few research papers are available focusing on the mechanical performance of large diameter welded studs, whose diameters are greater than 22 mm. Badie et al. [15] experimentally investigated the mechanical performance of welded stud connectors with 31.8 mm in diameter. It was found that 31.8 mm stud connectors could achieve a shear capacity two times that of 22 mm stud connectors, while the slip at failure would decrease about 30%. Shim et al. [16] and Lee et al. [17] performed static and fatigue push-out tests on welded stud connector specimens with 25 mm, 27 mm, and 30 mm in diameter and approximately 430 MPa in tensile strength. The test results showed that the shear strengths predicted by EN 1994-1-1 [18] and AASHTO LRFD [19] were conservative for these large stud connectors, and it was considered that the ductility of these large stud connectors was sufficient for the application in composite bridges. Nguyen and Kim [20] conducted an extensive parametric study to investigate the effect of the stud diameter and concrete strength on the shear performance of welded stud connectors. It was also concluded that the ductility of the large stud connectors was sufficient for the practical application in composite bridges. Conclusions from each researcher lack consistency for large diameter stud connectors, especially in terms of their ductility. Given that the configurations of the push-out specimens adopted by Badie et al. [15] and Shim et al. [16] were totally different, it could be inferred that the configurations of the push-out specimens might have significant effect on the ductility of larger diameter stud connector. Besides, studies on high strength studs are scarcely reported in the literature. Therefore, it is very practical to investigate the static behaviors of large diameter and high strength stud connectors in order to expand their application in steel-concrete composite/hybrid structures.

In this paper, push-out tests are implemented on four groups of welded stud connector specimens, including conventional and high strength 22 mm stud connector specimens and 25 mm and 30 mm stud connector specimens. The shear strength of these welded stud connectors is investigated and compared with the prediction of the design equations recommended in AASHTO LRFD [19], EN 1994-1-1 [18], and GB. The shear stiffness and ductility of these welded stud connectors are also examined and compared with the estimation by the existing equations. In addition, the applicability of load-slip relationships for conventional stud connectors to large diameter and high strength stud connectors is also discussed.

2. Experimental Work

2.1. Push-Out Specimens. Four groups of push-out specimens, denoted as SN22, SH22, SN25, and SN30, are devised in this research, and the specimens in each group are totally identical. Table 1 exhibits the main variables for each group push-out tests. Groups SN22, SN25, and SN30 specimens, with 22 mm, 25 mm, and 30 mm diameter welded studs, respectively, were tested to investigate the effect of stud diameters on their shear performance. The nominal tensile strength for these three groups of welded studs is 400 MPa, which is the normal tensile strength specified in the design code. Group SH22 specimens were tested and compared with Group SN22 specimens to examine the effect of tensile strengths on their shear performance. The nominal tensile strength for this group of welded studs is 650 MPa, which is a much higher tensile strength than the conventional weld studs. The overall welded stud height is 200 mm for all the specimens.

Figure 2 demonstrates the configuration of the push-out specimen in this research. Each specimen consists of an assembled push-out steel member, two concrete blocks, and four welded stud connectors. The transverse spacing for welded studs is 150 mm, and the spacing from the welded studs to the top and bottom of the concrete block both is 230 mm. The concrete slab thickness in the specimen for standard push-out tests recommended in EN 1994-1-1 [18] is only 150 mm, and the thinner concrete slab may result in a splitting failure mode of the concrete slab rather than the fracture of stud connectors due to the higher shear strength of large diameter and high strength stud connectors. Besides, the thickness of the concrete pylon wall, the integral concrete

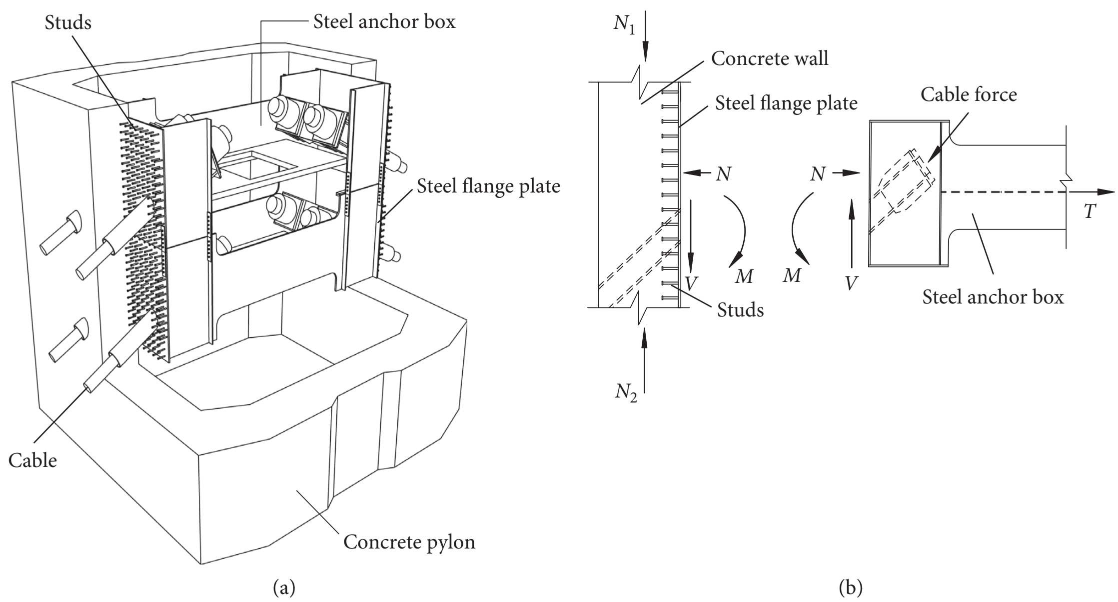


Figure 1: Steel-concrete composite cable-pylon anchorage. (a) Cable-pylon anchorage with steel anchor box. (b) Mechanical model of the stud connection.

Table 1: Program of push-out tests.

Group	Specimen number	Stud diameter (mm)	Stud tensile strength (MPa)	Stud height (mm)
SN22	4	22	400	200
SN25		25		
SN30	3	30		
SH22		22	650	

abutment, and the concrete deck slabs in the haunch part could be at least 400 mm. Therefore, the thickness of the concrete blocks in the push-out specimens is set to be 400 mm, while the width and height of the concrete blocks in the push-out specimens are 460 mm.

The steel plate thickness of the push-out steel member is 20 mm. The height of the push-out steel member is 460 mm, and the width of the flange and web is 350 mm and 220 mm, respectively. Given that the maximum slip at the steel-concrete interface would not exceed 20 mm, the distance from the bottom of the steel member to that of the concrete block is set to be 50 mm. The concrete blocks were casted in the horizontal position and air-cured. Bond and friction at the interface between the steel flanges and the concrete slabs were avoided and reduced by greasing the flanges. The reinforcement diameters are 20 mm, 12 mm, and 20 mm for N1, N2, and N3, respectively. The fabrication process of the push-out specimens is shown in Figure 3.

2.2. Material Properties. Six concrete cubic specimens (150 mm × 150 mm × 150 mm) were cast at the same time as pouring the concrete blocks to determine material properties of the concrete. All the cubic specimens were air-cured alongside the push-out specimens. Three specimens were tested at 28th day, and the other three were tested at the test day. Table 2 summarizes the measured material properties of concrete. The nominal cubic compressive strength of the concrete is 60 MPa.

The headed studs used in Groups SN22 and SN25 were fabricated according to GB/T 10433-2002 [21]. The diameter of headed studs used in Group SN30 and the specified tensile strength used in Group SH22 exceed the limitations of the specification. Therefore, these two groups of headed studs were specially fabricated. The specified yield strengths of the stud material are 320 MPa for Groups SN22, SN25, and SN30 and 550 MPa for Group SH22, respectively. The actual yield strengths and ultimate strengths of the stud material were provided by the supplier. HRB335 reinforcements (with 335 MPa nominal yield strength) and Q345C steel plates (with 345 MPa nominal yield strength) were used in the fabrication of the push-out specimens. Three 10 mm × 10 mm × 50 mm samples of steel plate were tested under direct tension to obtain the mechanical properties, and coupon tests were performed to get the properties of the reinforcements. The mean values of the measured material properties of the studs, reinforcements, and steel plates are summarized in Table 3.

2.3. Loading Procedure and Measurement. As shown in Figure 4, the push-out specimens were tested using a hydraulic testing machine. In order to apply the load uniformly, a stiff steel transfer component was placed on the top of the push-out steel member. Fine sands were spread on the surface of the base to avoid uneven loading. Force control was adopted before 70% of the predicted failure load, and the loading rate is 6 kN/min. Then, displacement control was used until the load dropped to 80% of the maximum load or failure of the specimen was observed with the loading rate equal to 0.5 mm/min. Four displacement sensors were installed at the same elevation as the welded stud connector to

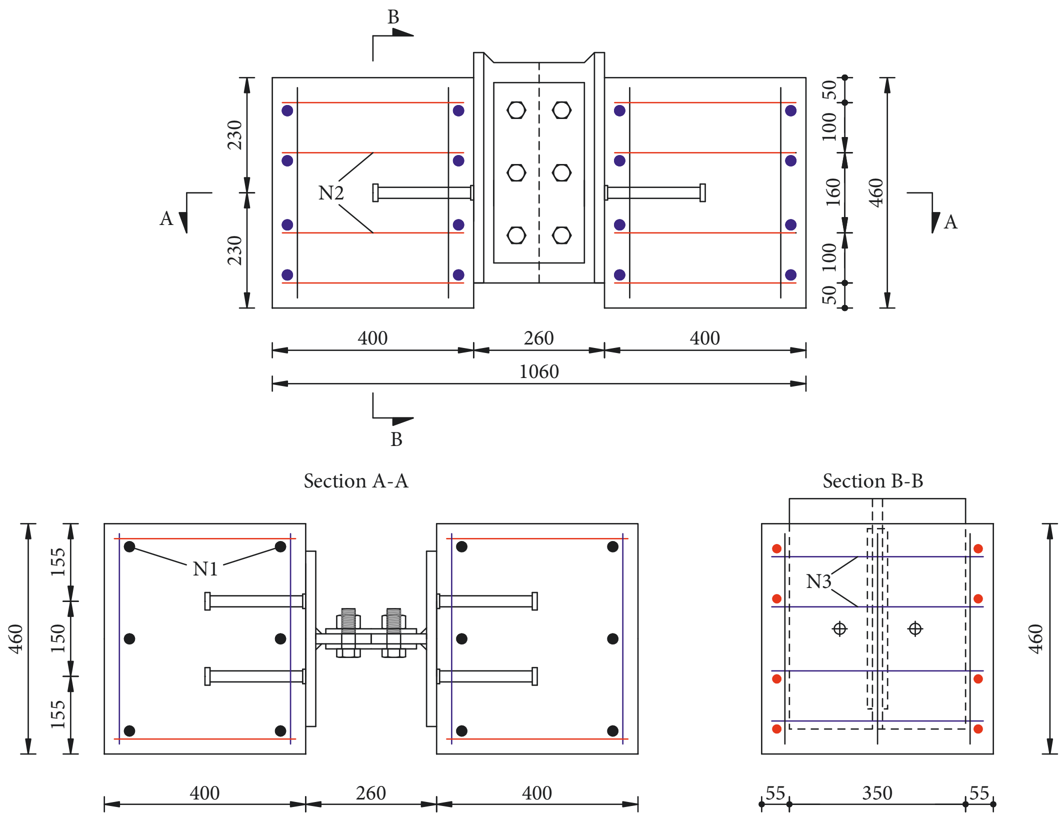
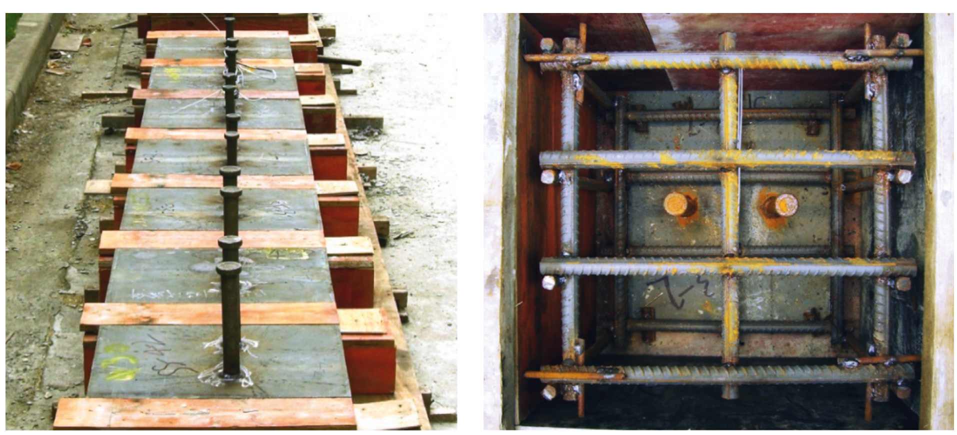


Figure 2: Details of push-out specimens.



(a) (b)

Figure 3: Fabrication of push-out specimens.

Table 2: Compressive strength of concrete (MPa).

Curing period	1	2	3	Mean
28th day	65.6	64.9	68.2	66.2
Test day	68.4	71.6	70.9	70.3

measure the relative slip between the concrete slab and the steel beam. In the following analysis of the slip data, the average value of the four vertical displacements was taken as the representative slip for each specimen.

3. Push-Out Test Results

3.1. Failure Modes. It can be seen from Figure 5 that the failure modes for four groups of welded stud connector specimens are almost identical, which is the stud shank fracture with apparent local concrete crushing under the shank root of welded studs. The stud welding failure in all the push-out tests was not observed as well as the concrete splitting phenomenon since the concrete blocks had adequate strength and thickness. It is concluded that ductile failure modes can also be achieved for large diameter and

Table 3: Properties of steel materials (MPa).

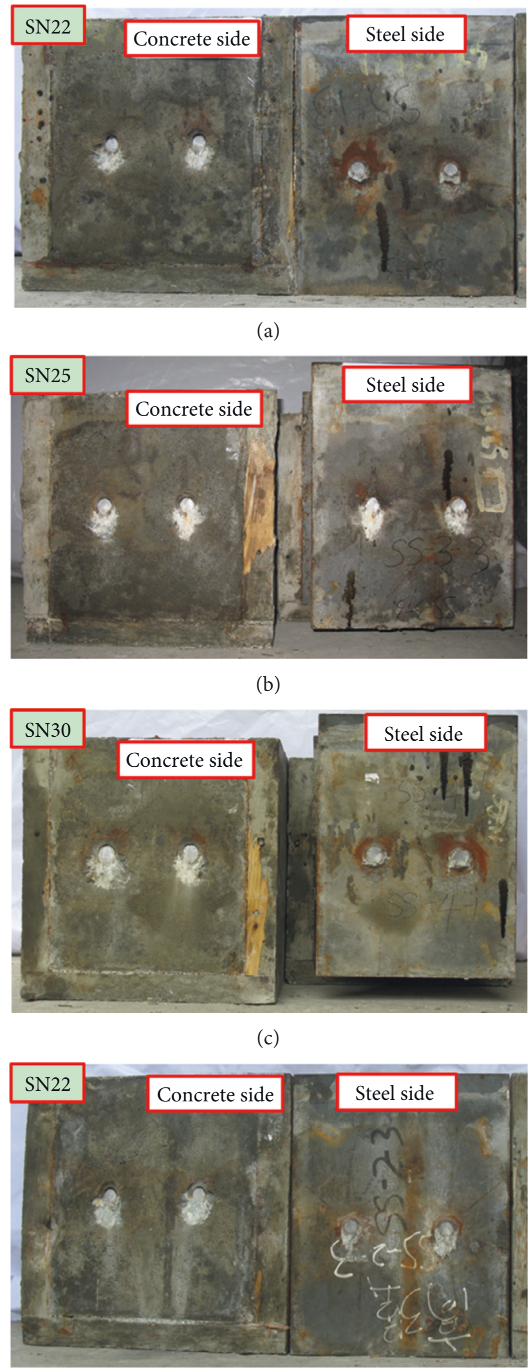
Component		Yield strength		Tensile strength	
		Nominal	Test	Nominal	Test
Stud	SN22		385		530
	SN25	320	380	400	485
	SN30		375		430
	SH22	550	630	650	675
Reinforcement		335	374	—	578
Steel plate		345	410	—	545

Figure 4: Loading setup of push-out tests.

high strength welded stud connectors when the same welding technique and concrete strength used for conventional stud connectors are employed.

3.2. Test Results. Figure 6 shows the load-slip curves for four groups of welded stud connector push-out specimens. There are three stages for the load-slip curves of welded stud connectors including initial linear stage, nonlinear or strengthening stage, and descending stage. All the welded stud connectors exhibit certain shear ductility in the nonlinear or strengthening stage since there are obvious slips at the steel-concrete interface. After reaching the ultimate shear bearing capacity, the loading force would slightly decrease under the displacement-controlled loading regime and the welded studs would fracture one by one eventually. Accordingly, the descending stage in the load-slip curves of each push-out specimen could be neglected in the further analysis of these load-slip curves.

Table 4 exhibits the final test results for each push-out specimen including their shear strength, shear stiffness, and peak slip. In this paper, shear strength refers to the ultimate shear capacity of each push-out specimen. Shear stiffness is defined as the secant modulus at the point where the applied load is half of the ultimate load on the load-slip curve for normal welded studs [2, 22] since load-slip curves are almost linear until that load and shear forces in welded stud connectors usually are less than half of the ultimate load at the serviceability limit state. The peak slip of the load-slip curve corresponds to the slip at the



(a) (b) (c) (d)

Figure 5: Failure modes of push-out specimens.

ultimate shear capacity. The ultimate slip of the load-slip curve may be meaningless for discussion due to the extremely short descending branch of each load-slip curve. In the following, the shear bearing capacity, peak slip, shear stiffness, and load-slip curve of four groups of welded stud push-out specimens will be discussed in detail.

3.3. Shear Strength. Figure 7 shows the shear strength distribution of four groups of welded stud connectors, in which

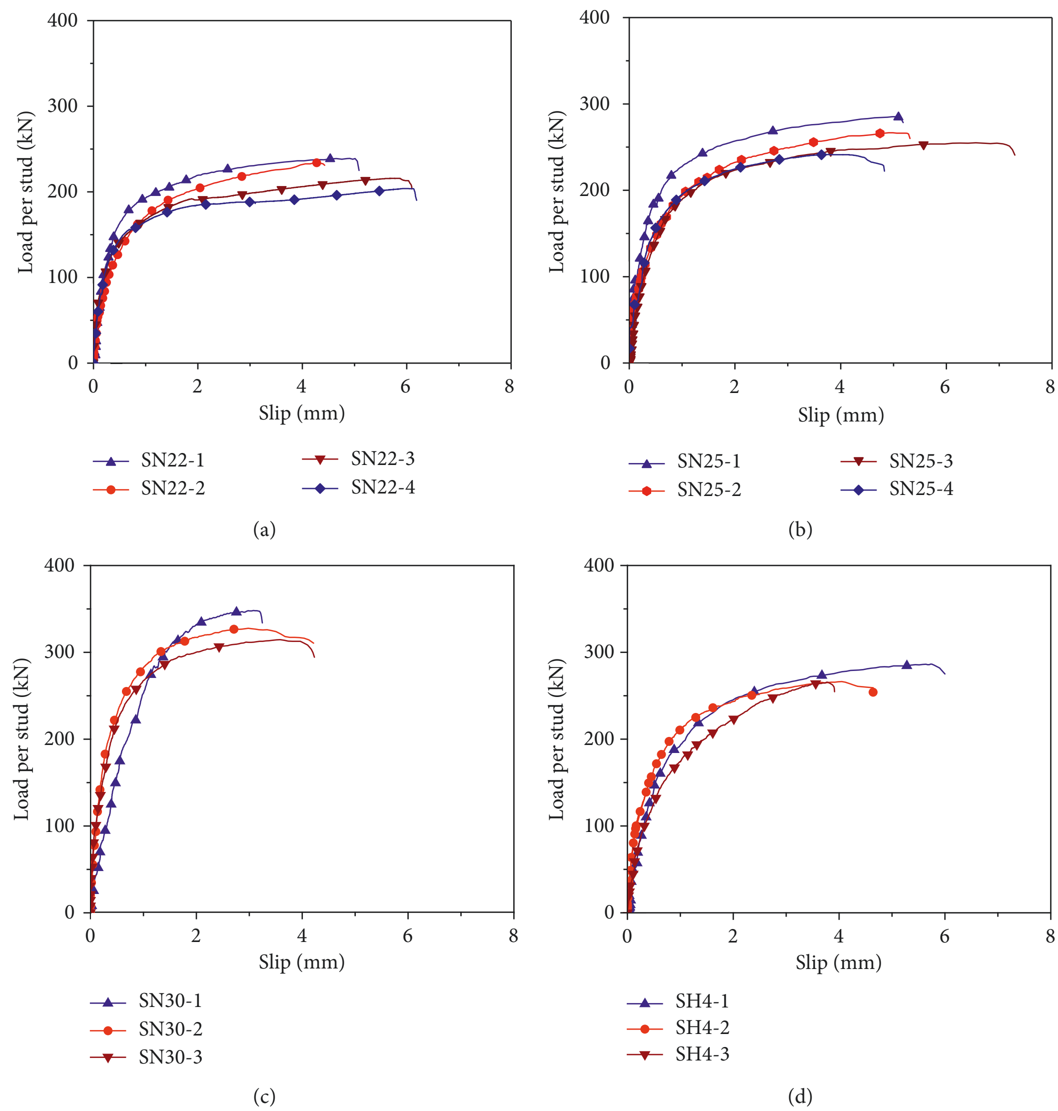


Figure 6: Load-slip curves for four groups of push-out specimens.

Table 4: Test results for each push-out specimen.

Specimen		Shear strength (kN)		Shear stiffness (kN/mm)		Peak slip (mm)	
		Test	Mean	Test	Mean	Test	Mean
SN22	1	239	223	448	422	4.9	5.2
	2	234		300		4.2	
	3	216		447		5.8	
	4	204		494		5.9	
SN25	1	285	262	526	394	5.0	5.1
	2	267		327		4.9	
	3	255		309		6.6	
	4	242		415		3.8	
SN30	1	348	330	316	554	3.1	3.2
	2	328		727		3.0	
	3	314		618		3.6	
SH22	1	286	273	284	313	5.7	4.5
	2	266		409		4.1	
	3	266		246		3.8	

the blue symbols represent the test shear strength value and the red lines stand for the mean value. The vertical axis in Figure 7(b) indicates the shear strength ratio of these four groups of welded stud connectors to the mean shear strength of group SN22, which is 223 kN as listed in Table 4. The average shear strengths of Group SN25, SN30, and SH22 are about 17%, 48%, and 22% larger than that of Group SN22, respectively, which illustrates that using large diameter or high strength welded stud connectors is able to increase the shear strength of welded stud connectors. However, it should be noted that the shank cross-sectional area of welded studs in Groups SN25 and SN30 is about 29% and 86% larger than that of Group SN22, respectively, while the tensile strength of welded studs in Group SH22 is 27% larger than that of Group SN22.

3.4. Shear Stiffness and Peak Slip. Figure 8 shows the shear stiffness and peak slip distribution for four groups of welded

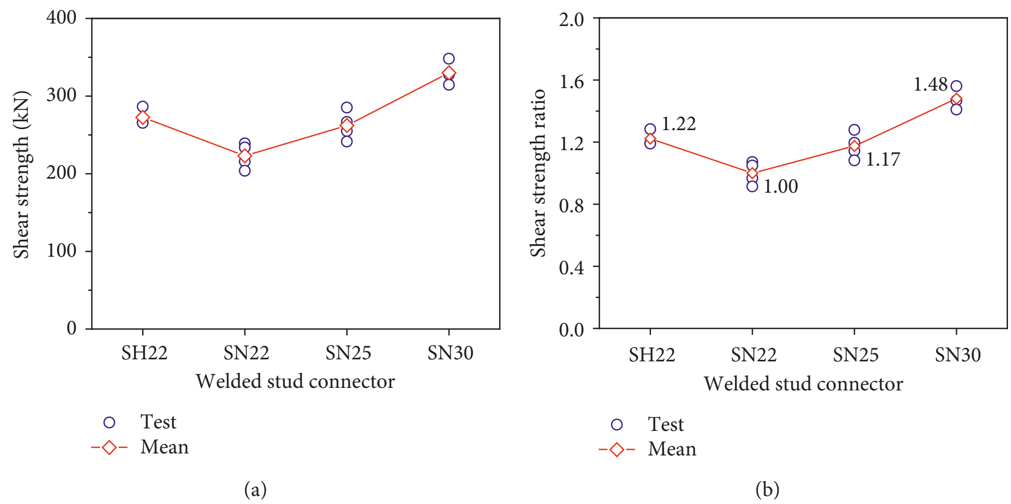


Figure 7: Shear strength distribution for four groups of push-out specimens.

stud connectors. It can be obviously seen that the test values of shear stiffness and peak slip are much more discrete than the corresponding shear strength values. Some conclusions still could be obtained from the comparison of the mean test values. It can be observed from Figure 8(a) that the average shear stiffness of Group SN30 welded stud connectors is much larger than that of other three groups, which may be produced by the larger diameter of Group SN30 welded studs. On the contrary, the average peak slip of Group SN30 welded stud connectors is less than that of other three groups as shown in Figure 8(b).

4. Evaluation of the Push-Out Test Results

4.1. Evaluation of Shear Strength. Both the 30 mm stud and the high strength stud exceed the limitation of current design codes, so the applicability of the existing design equations to large diameter and high strength studs need to be evaluated. In AASHTO LRFD [19], the design strength of one stud connector can be determined by the following equation:

$$Q_u = \phi 0.5 A_s \sqrt{E_c f'_c} \le \phi A_s f_u, \tag{1}$$

where Q_u is the nominal shear strength of the welded stud connector (N), A_s is the cross-sectional area of stud shank (mm^2), E_c is the elastic modulus of concrete (MPa), f'_c is the characteristic cylinder compressive strength of concrete (MPa), the nominal tensile strength of welded studs f_u is 415 MPa, and the resistance factor ϕ for welded stud connectors equals 0.85.

EN 1994-1-1 [18] specifies the design strength of one stud connector given by the following equation:

$$Q_u = \min\left\{\frac{0.29\alpha d^2\sqrt{E_c f'_c}}{\gamma_v}, \frac{0.8 A_s f_u}{\gamma_v}\right\}, \tag{2}$$

where d is the diameter of the stud shank (mm), which is specified between 16 mm and 25 mm, α is equal to 1.0 when the overall stud height is larger than four times of its diameter, the tensile strength of the stud material f_u is specified no greater than 500 MPa according to EN 1994-1-1 [18], the recommended value for the partial factor γ_v is 1.25, and other variables have the same meaning with Equation (1).

GB 50017 [23] provides Equation (3) to estimate the shear strength of welded stud connectors:

$$Q_u = 0.43 A_s \sqrt{E_c f_c} \le 0.7 A_s f_u, \tag{3}$$

where f_c is the characteristic compressive strength of concrete prisms (MPa) and other variables have the same meaning with Equation (1). In GB 50017 [23], the resistance factor ϕ or the partial factor γ_v for the shear strength of welded stud connectors is not clearly specified.

For the comparison of the shear strength predicted by Equations (1)–(3) with the test value, the resistance factor in Equation (1) and the partial factor in Equation (2) are not taken into account. The compressive strength of concrete prisms listed in Table 2 is used to predict the concrete cylinder compressive strength and the concrete elastic modulus. The tensile strengths of four groups of welded studs listed in Table 3 are taken into Equations (1)–(3) to predict the shear strength of welded stud connectors. It is found that the shear strength of all the test welded stud connectors is determined by the material tensile strength of welded studs rather than the compressive strength of concrete since the compressive strength of concrete blocks in the push-out specimens is up to 70 MPa in the test day.

Figure 9 shows the shear strength comparison results among Equations (1)–(3) and the test value. Equation (1) provided in AASHTO LRFD [19] could give much accurate shear strength prediction results for large diameter and high strength welded stud connectors as well as the conventional 22 mm welded studs. While Equations (2) and (3) provided in EN 1994-1-1 [18] and GB 50017 [23], respectively, would result in much conservative shear strength prediction results for all the test welded stud connectors. Accordingly, the shear strength of large diameter and high strength welded

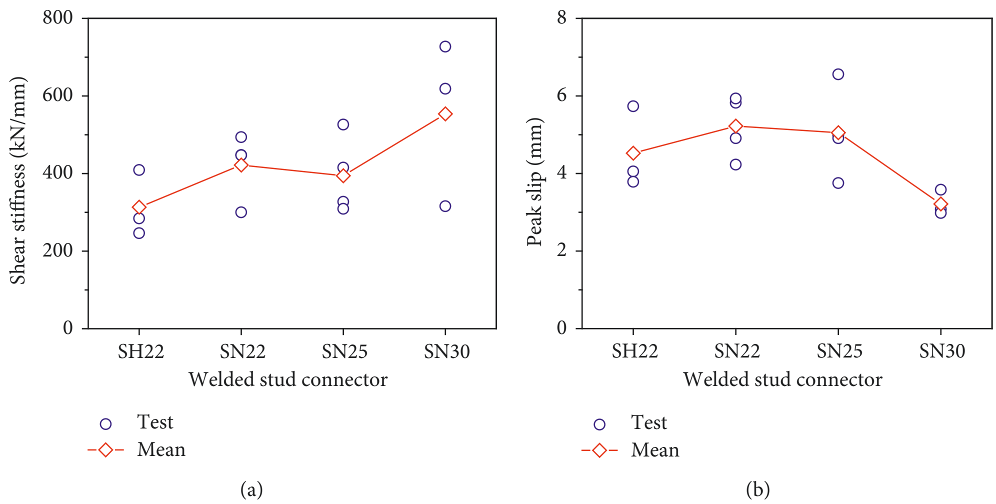


FIGURE 8: Shear stiffness and peak slip distribution for four groups of push-out specimens.

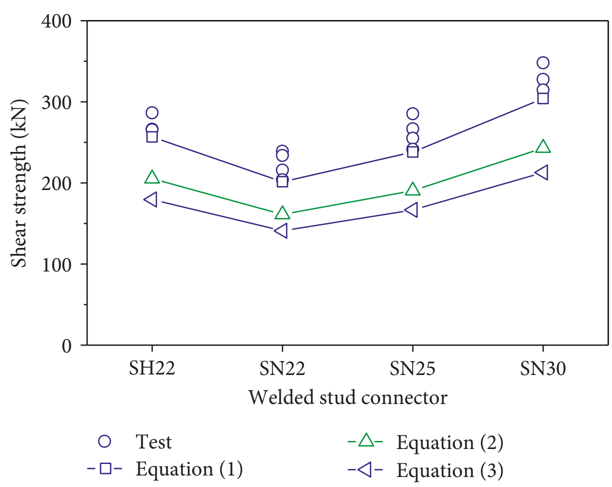


FIGURE 9: Shear strength comparison between the equation prediction and the test value.

stud connectors could also be accurately estimated by AASHTO LRFD [19] as the conventional welded stud connectors. Since the failure modes of the push-out tests are the stud shank fracture without the splitting of the concrete blocks, the shear strength of welded stud connectors could be regarded as the tensile strength of the studs, which is also verified by Pallarés and Hajjar [4, 5] in the research of welded stud connectors with conventional diameters.

4.2. Evaluation of Shear Stiffness. Interaction between steel and concrete elements in composite structures under service loads is mainly attributed to the bond and friction effect at the steel-concrete interface and the shear connectors. The bond and friction effect have great influences on the interaction between steel and concrete elements, and welded stud connectors carry almost no shear load before bond failure. So the behavior of shear connectors in the elastic range has negligible effect on the flexural behavior of composite beams. However, the bond effect may be damaged during the service life of a composite bridge, and friction is too complicated to be determined accurately. Accordingly, the interaction would only rely on the shear connectors to ensure the safety of composite structures, and the shear stiffness of shear connectors has great effect on the shear force distribution at the steel-concrete interface.

Previous researchers have proposed various definitions for the shear stiffness prediction of welded stud connectors. Oehlers and Coughlan [2] proposed an empirical equation to calculate the initial shear stiffness of normal stud connectors based on the experimental results, as shown in Equation (4). It is adopted by Shim et al. [16] to evaluate the shear stiffness of large diameter studs, and the results show that Equation (4) gives rather conservative predictions:

$$k = \frac{Q_u}{(0.16 - 0.0017 f_c)d}. \tag{4}$$

Lin et al. [24] proposed a semi-theoretical equation for predicting the shear stiffness of normal stud connectors based on the theory of a beam on an elastic foundation and ninety-nine test results collected from the available literature, as shown in the following equation:

$$k = 0.32 d E_c^{0.75} E_s^{0.25}. \tag{5}$$

In Equations (4) and (5), k represents the initial shear stiffness of welded stud connectors defined as the ratio of half of the shear strength to the corresponding slip (N/mm); Q_u is the shear strength of welded stud connectors (N); f_c is the cylinder compressive strength of concrete (MPa), which could be taken as 59.8 MPa based on the relationship between the cylinder compressive strength and the cube compressive strength proposed by Mansur and Islam [25]; d is the diameter of the stud shank (mm); E_c stands for the elastic modulus of concrete (MPa), which could be regarded as 37.6 GPa predicted by $E_c = 22[f_c/10]^{0.3}$ proposed by EN 1992-1-1; and E_s is the elastic modulus of the stud material (MPa) which is equal to 2.0×10^5 MPa.

The accuracy of Equations (4) and (5) is evaluated, as shown in Figure 10. The shear strength of welded stud connectors Q_u in Equation (4) is set to be the average shear strength of each group of welded stud connectors. It can be obviously seen from Figure 10 that Equation (5) could give more accurate predictions of the shear stiffness for large diameter and high strength welded stud connectors. While Equation (4) dramatically underestimates the shear stiffness of welded stud connectors, which is consistent with the conclusion made by Shim et al. [16].

4.3. Evaluation of Peak Slip. The peak slip of the load-slip curve s_p, slip corresponding to the ultimate shear capacity, is an important parameter to evaluate the shear deformation capacity of welded stud connectors. Given that the descending branches of the load-slip curves for each push-out test are extremely short, the peak slip could also represent the ultimate slip of the load-slip curves, which is usually used to describe the ductility of welded stud connectors. Adequate ductility is very favorable to the shear force redistribution at the steel-concrete interface of composite structures. Oehlers and Coughlan [2] put forward Equation (6) to predict the peak slip of welded stud connectors:

$$s_p = (0.480 - 0.0042 f_c)d. \tag{6}$$

The International Federation for Structural Concrete [26] suggests the following equation to calculate the peak slip of welded stud connectors:

$$s_p = (0.389 - 0.0023 f_c)d. \tag{7}$$

In Equations (6) and (7), f_c is the cylinder compressive strength of concrete which is equal to 59.8 MPa and d stands for the diameter of the stud shank (mm).

Figure 11 demonstrates the peak slip comparison results among Equations (6) and (7) and the test value. The peak slips of welded stud connectors embedded in the identical strength concrete would increase with the increment of the stud shank diameter according to both Equations (6) and (7), while the test values for four groups of welded stud connectors as listed in Table 4 and shown in Figure 11 do not exhibit the same tendency. The average peak slip of Group SN25 welded stud connectors is almost equal to that of Group SN22 welded stud connectors, while the peak slips of Group SN30 welded stud connectors would be dramatically overestimated based on Equations (6) or (7). The average peak slip of Group SN30 is only 3.2 mm, which is less than the predicted peak slip based on Equations (6) or (7). The test peak slips of welded stud connectors with large diameter are consistent with the test results conducted by Badie et al. [15], which exhibited that the peak slips of welded stud connector with 31.8 mm diameter would decrease about 30% compared with conventional 22 mm welded stud connectors. The peak slips of Group SH22 with the high strength stud material have a lower value compared with those of Group SN22 since the high strength stud material can result in a lower elongation, which has a significant effect on the ductility of welded stud connectors. Accordingly, it can be concluded that Equations (6) and (7) could only be employed in the peak slip prediction of welded stud connectors with diameter less than 25 mm, and Equation (6) could produce a relatively accurate prediction results when the concrete strength are much higher. In spite of above discussion, it still can be seen that the test results from different research studies lack consistency and thus more experimental studies should be performed on the large diameter and high strength welded stud connectors in order to ensure their safe application in composite structures.

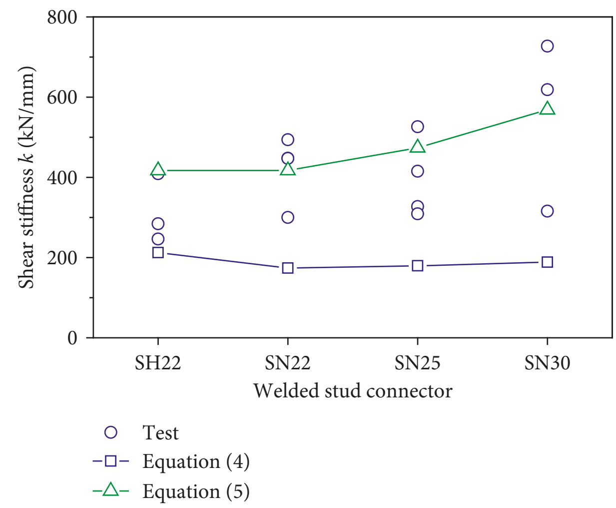


Figure 10: Shear stiffness comparison between the equation prediction and the test value.

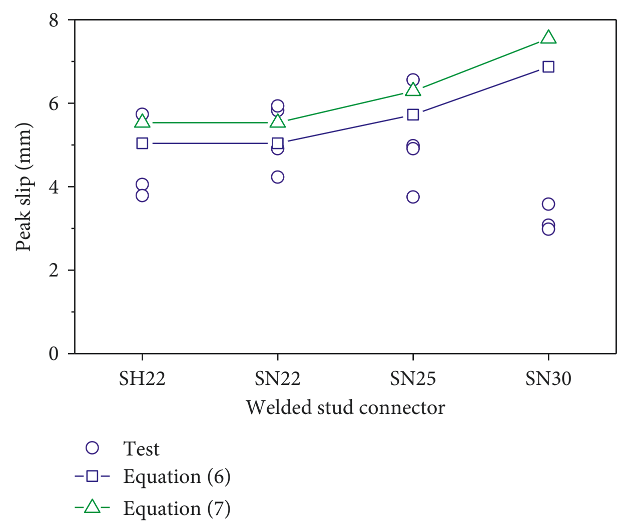


Figure 11: Peak slip comparison between the equation prediction and the test value.

4.4. Evaluation of Load-Slip Curve. Load-slip relationship can reflect the nonlinear behavior of welded stud connectors at the ultimate limit state. An expression capable of describing the load-slip curve accurately is indispensable for the nonlinear analysis of steel-concrete composite structures

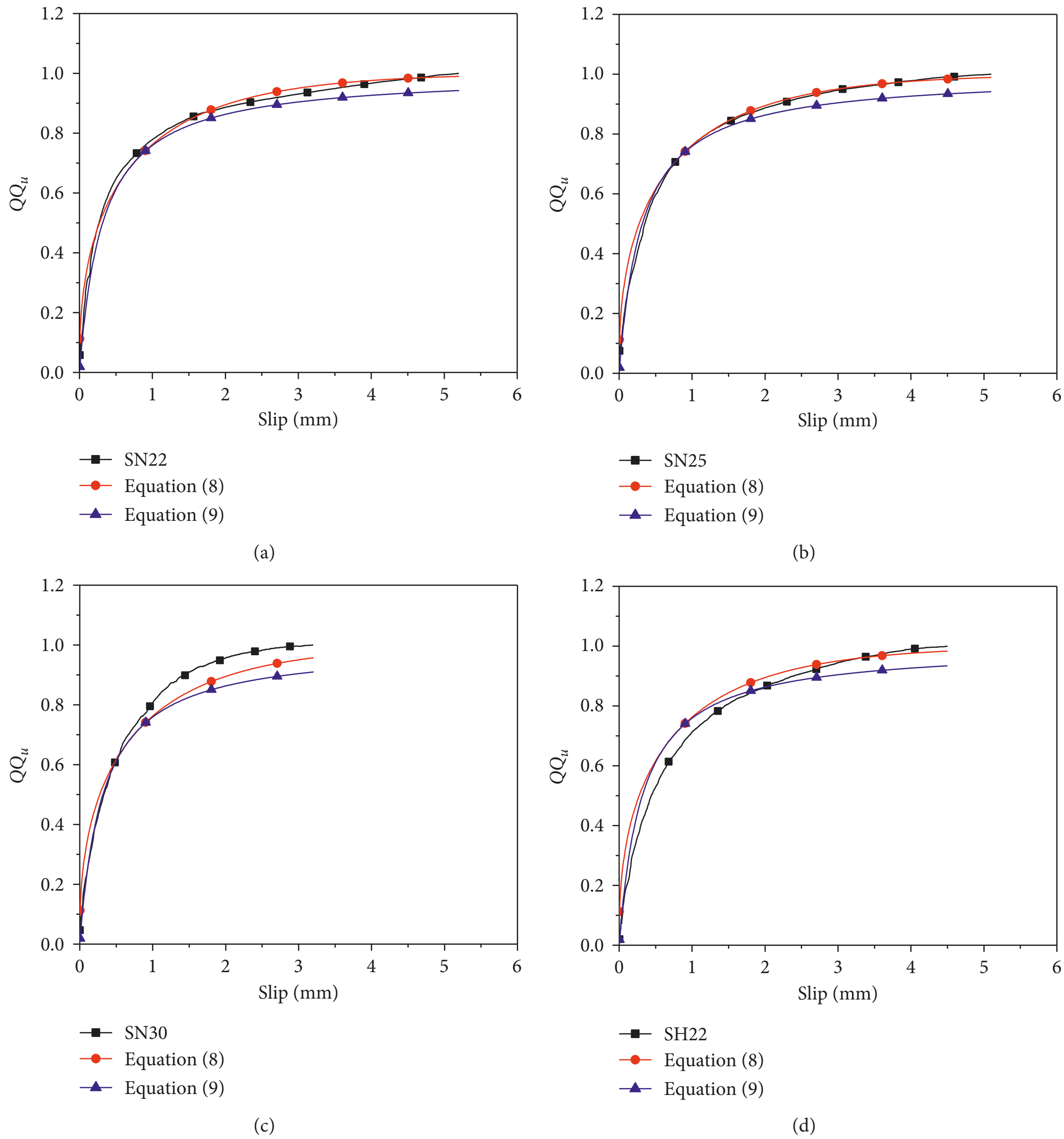


FIGURE 12: Load-slip curve comparison between the equation relationship and the test curve.

connected by welded stud connectors. The expressions of load-slip relationships for large diameter and high strength welded stud connectors are not examined before. Thus, it is necessary to investigate the applicability of the existing load-slip relationships to large diameter and high strength welded stud connectors.

Ollgaard et al. [1] proposed an empirical formula for the load-slip curve of conventional welded stud connectors under continuous loading:

$$\frac{Q}{Q_u} = \left(1 - e^{-0.71S}\right)^{0.4}. \tag{8}$$

Much earlier, Buttry [27] put forward a fractional formula for the load-slip curve of conventional welded stud connectors:

$$\frac{Q}{Q_u} = \frac{3.15S}{(1 + 3.15S)}. \tag{9}$$

In Equations (8) and (9), S is the slip value of welded stud connectors (mm) and Q/Q_u stands for the ratio of the loading force to the shear strength of welded stud connectors.

For comparing the test load-slip curves of four groups of welded stud connectors with the fitting load-slip curves by Equations (8) and (9), the average load-slip curves of each group of welded stud connectors are computed based on the average shear strength and peak slip of each group of welded stud connectors as listed in Table 4. The final load-slip curve comparison results among the test curves and prediction results by Equations (8) and (9) are depicted in Figure 12.

It can be obviously seen from Figure 12 that both Equations (8) and (9) could not only describe the load-slip curve of Group SN22 conventional welded stud connectors accurately but also give acceptable prediction results for Group SN25 and Group SN30 large diameter welded stud connectors and Group SH22 high strength welded stud connectors. Additionally, Equation (8) will predict the nonlinear part of the load-slip curves much well, while Equation (9) will produce a much accurate result for the initial or linear part of the load-slip curves for all groups of welded stud connectors. Consequently, both Equations (8) and (9) can be used to predict the load-slip curves of large diameter and high strength welded stud connectors for the nonlinear structural analysis of composite structures.

5. Conclusion

In this research, fourteen push-out tests were carried out to investigate the shear performance of large diameter and high strength welded stud connectors. The shear strength, shear stiffness, peak slip, and load-slip relationship of these welded stud connectors were examined and compared with the existing design codes or prediction equations. The following conclusions can be drawn:

(1) The ductile failure mode could be achieved for large diameter and high strength welded stud connectors when the same welding technique and concrete strength used for conventional welded stud connectors are employed.

(2) The average shear strength of Group SN25, SN30, and SH22 welded stud connectors are about 17%, 48%, and 22% larger than that of Group SN22, respectively, confirming that using large diameter or high strength welded studs increases the shear strength of a welded stud connector. The shear strength of large diameter and high strength welded stud connectors could be accurately estimated by AASHTO LRFD [19] as the conventional welded stud connectors.

(3) The shear stiffness equation proposed by Lin et al. could give much accurate and reasonable shear stiffness prediction for welded stud connectors including large diameter and high strength welded stud connectors.

(4) The existing peak slip prediction equations could only be employed in the welded stud connectors with diameter less than 25 mm, and the peak slip prediction equation proposed by Oehlers and Coughlan could produce relatively accurate results when the concrete strength is much higher.

(5) The existing load-slip relationships proposed by Ollgaard and Buttry could not only describe the load-slip curve of Group SN22 welded stud connectors accurately but also give acceptable prediction results for Group SN25 and Group SN30 large diameter welded stud connectors and Group SH22 high strength welded stud connectors.

(6) Since the push-out tests on the large diameter and high strength welded stud connectors are rather limited in this paper, a detailed parametric analysis on the shear performance of these welded stud connectors using the finite element analysis method is underway.

Data Availability

The load-slip curve data of the push-out tests in this paper are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

Acknowledgments

This work was financially supported by the National Natural Science Foundation (Grant no. 51578406) of People's Republic of China.

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