

STUDY ON SHEAR BEHAVIOR OF FOUR-CORNER BENT BAR CONNECTORS IN PREFABRICATED CONCRETE COMPOSITE FLOOR SLAB

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ABSTRACT

The four-corner bent bar connectors are used to connect the prefabricated concrete composite floor slab with the H-shaped steel beam, aiming to ensure working performance of the floor system. To investigate shear behavior of the four-corner bent bar connectors, a push-out test was conducted with the diameter of steel bar as a variable parameter. Additionally, a push-out specimen model was established using ABAQUS, and the accuracy of models was verified through comparison with test results. On this basis, the entire force process and key parameters of the shear behavior of the four-corner bent bar connectors were analyzed. The results indicate that the four-corner bent bar connector exhibits high initial stiffness and bearing capacity. Moreover, the characteristic displacements of four-corner bent bar connectors comply with the requirement that the lower slip limit for flexible connectors is 6mm, as stipulated in Eurocode 4. Therefore, the four-corner bent bar connector belongs to flexible connectors, demonstrating good ductility. Among the factors, the diameter of steel bars has the most significant influence on its shear bearing capacity, followed by the compressive strength of cast-in-place concrete, while the compressive strength of prefabricated concrete has the least influence. It is recommended that the diameter of steel bars for the four-corner bent bar connector should be set at 14mm, and the strength of steel bars should be HRB400. Based on the numerical simulation results, a calculation formula for the shear bearing capacity applicable to four-corner bent bar connectors was proposed using the method of multiple linear regression.

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1. Introduction

The prefabricated steel structure floor systems present advantages such as rapid construction and stable structure [1]. Among them, the prefabricated concrete composite floor system is more prominent due to its excellent integrity, crack resistance, high stiffness, and spacious interior layout. It has been widely employed in prefabricated buildings and has become one of the most commonly used forms in the prefabricated structure system [2]. Nevertheless, the bond effect at the interface between precast composite slabs and steel beams is relatively weak, failing to resist longitudinal shear force subjected to high load levels. In order to ensure the overall working performance of steel beams and precast floor slabs, the shear connectors must be installed between them.

The shear connectors primarily serve to withstand the longitudinal shear force and horizontal uplift forces between the steel beam and concrete [3]. Stud connectors, the most traditional type of shear connectors, are extensively used in practical engineering [4-7]. With the rapid development of composite structures, various types of shear connectors, including channel steel connectors, angle steel connectors, and bent bar connectors have been widely applied in all kinds of building structures. Among these, angle steel connectors offer distinct advantages over channel steel connectors, including a wider range of specifications, ease of installation, material savings, and high shear strength. By using standard welding equipment to secure steel plates to the beam flange plates via angle welds, their mechanical performance is ensured. They are now widely employed in building structures requiring high shear resistance [8].

The bent bar connector was first proposed by German scholar Leonhardt in the 1960s for optimizing the design of shear connectors in steel-concrete composite structures. Dhanush [9] analyzed truss connectors used in composite beams and segmental composite slabs, revealing that their load-bearing capacity remains stable beyond an inclination angle of 60 degrees. For given inclination angles, variations in reinforcement diameter showed no significant impact on performance. Bezerra [10] and Lima [11] investigated truss-type connectors with different angles, analyzing how geometric parameters influence shear capacity and slip behavior. Notably, Bezerra's experimental results indicated that truss-type connectors outperformed bolt-type connectors in both load-bearing capacity and slip performance [12]. Kanchanadevi [13] examined the effects of 30, 45, and 60 degrees inclination angles on the performance of truss-type connectors in floors, finding that a 45 degree angle yields optimal mechanical performance by enhancing shear capacity while effectively controlling slip behavior. Li [14] pioneered curved bar connectors in composite beams, demonstrating their shear resistance under monotonic and cyclic loading conditions. They established corresponding load-bearing formulas that laid the foundation for subsequent studies. Zhou [15] conducted the first lattice-type derivation experiment for reverse-arranged curved bar connectors, analyzing their working mechanisms and deriving load-bearing formulas under reverse arrangement. Zhang [16] investigated serpentine curved bar connectors in

beams, analyzing the effects of bar diameter, anchorage length, and concrete strength through derivation experiments. Zhang [17] first proposed the application of short-steel-bar connectors in bridge decks, using ABAQUS software to specifically examine how connector diameter and weld length influence shear capacity and slip behavior. Shao [18] analyzed short-steel-bar connectors, revealing their excellent shear transfer capability and high shear capacity, with shear capacity increasing as weld length increases. Yu [19] explored steel bar connectors in corrugated composite structures, revealing their shear performance and failure characteristics under complex conditions, and derived shear stiffness calculation formulas. In summary, previous research primarily focused on comparative analysis of connector advantages through shape modifications and the influence of different factors.

Based on the performance characteristics of existing bent bar connectors, this paper proposed a new-type shear connector, which employs the four-corner bent bar shear connectors to connect precast concrete composite floor slabs with H-shaped steel beams for ensuring the integral composite behavior of the floor system (as shown in Fig. 1). Compared with the commonly used stud connectors, the four-corner bent bar shear connectors exhibit superior shear resistance, pull-out resistance, uplift resistance and fatigue resistance. Under identical load conditions, the required number of connectors is significantly reduced, thereby enhancing construction efficiency. Furthermore, due to the structural characteristics of the four-corner bent bar shear connectors, the formed spatial configuration can effectively enhance the synergistic performance between the bent bar and concrete, while overcoming the limitation that conventional bent bar connectors can only bear shear force in a single direction. The push-out tests and finite element analysis were carried out on specimens with four-corner bent bar connectors to investigate the failure modes, entire load-carrying processes, and load-slip curves. The influences of parameters such as steel bar diameter, steel bar yield strength and concrete compressive strength on the shear behavior of connectors were analyzed. On the basis of numerical analysis results, a bearing capacity calculation formula for this type of connector was proposed, providing a reference for its design and engineering application.

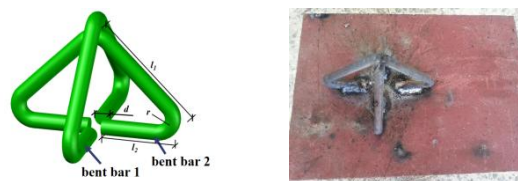


Fig. 1 Four-corner bent bar shear connectors

2. Experimental investigation

2.1. Specimen design

The specimens were designed according to Eurocode 4 [20]. The dimensions of the H-shaped steel beam were specified as 300mm×300mm×10mm×15mm, the composite slab (containing cast-in-place and precast concrete slabs) was 500mm×150mm×400mm, and the cast-in-place

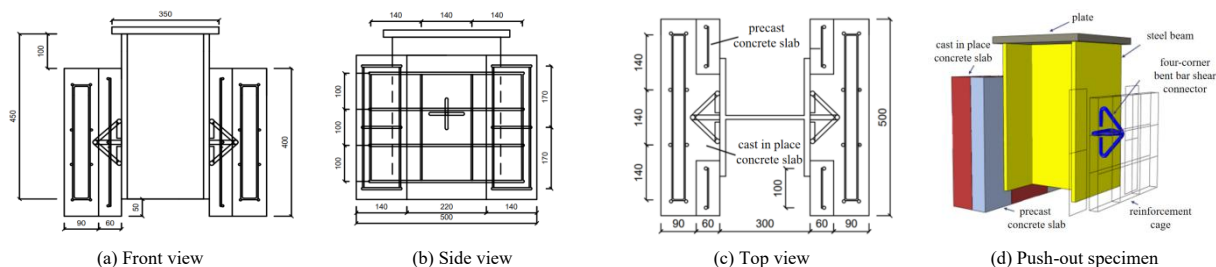


Fig. 2 Geometric dimensions of the specimen

Table 1
Parameters of four-corner bent bar connectors

(a) Bend bar 1						(b) Bend bar 2					
Specimens	d/mm	l_1/mm	l_2/mm	$\alpha/^\circ$	r/mm	Specimens	d/mm	l_1/mm	l_2/mm	$\alpha/^\circ$	r/mm
WJ1-1	12	90	54	45	12	WJ2-1	12	107	54	45	12
WJ1-2	14	105	63	45	14	WJ2-2	14	125	63	45	14
WJ1-3	16	120	72	45	16	WJ2-3	16	143	72	45	16

Note: d is the steel bar diameter; l_1 denotes the inclined length of the steel bar; l_2 denotes the straight segment length of the steel bar; α is the bent bar angle; r is the bent bar radius.

2.2. Material performance

Both the H-shaped steel beam and the end plate were manufactured using Grade Q235B. Four-corner bent bars and steel reinforcement framework adopted Grade HPB300 steel, among which the stressed bars were $\phi 8$ spiral bars and the distribution bars were $\phi 6$ spiral bars. Precast concrete slabs were cast with C30 concrete, while the cast-in-place concrete slab adopted C45 concrete. Standard 150mm cubic specimens were cured consistent with the same field conditions. Test measurements showed that the average cubic compressive strength of C45 and C30 concrete were 45.3MPa and 30.0MPa, and the elastic modulus were 3.35×10^4 MPa and 3.00×10^4 MPa, respectively. The material performance parameters of the four-corner bent bar shear connectors are specified in Table 2.

Table 2
Material performance of steel

Bar diameter	Steel grade	f_y/MPa	f_u/MPa	E_s/GPa
$d=12mm$	HPB300	346.0	530.5	201
$d=14mm$	HPB300	354.2	546.3	209
$d=16mm$	HPB300	403.6	560.4	210

Note: f_y , f_u , E_s represent the yield strength, tensile strength and elastic modulus of the steel respectively.

2.3. Loading device and scheme

The loading device is depicted in Fig. 3. A 100-ton hydraulic jack was adopted for load application. The test was subjected to monotonic concentrated

load, which consisted of two stages: preloading and formal loading. The specific loading scheme was consistent with that detailed in reference [21].

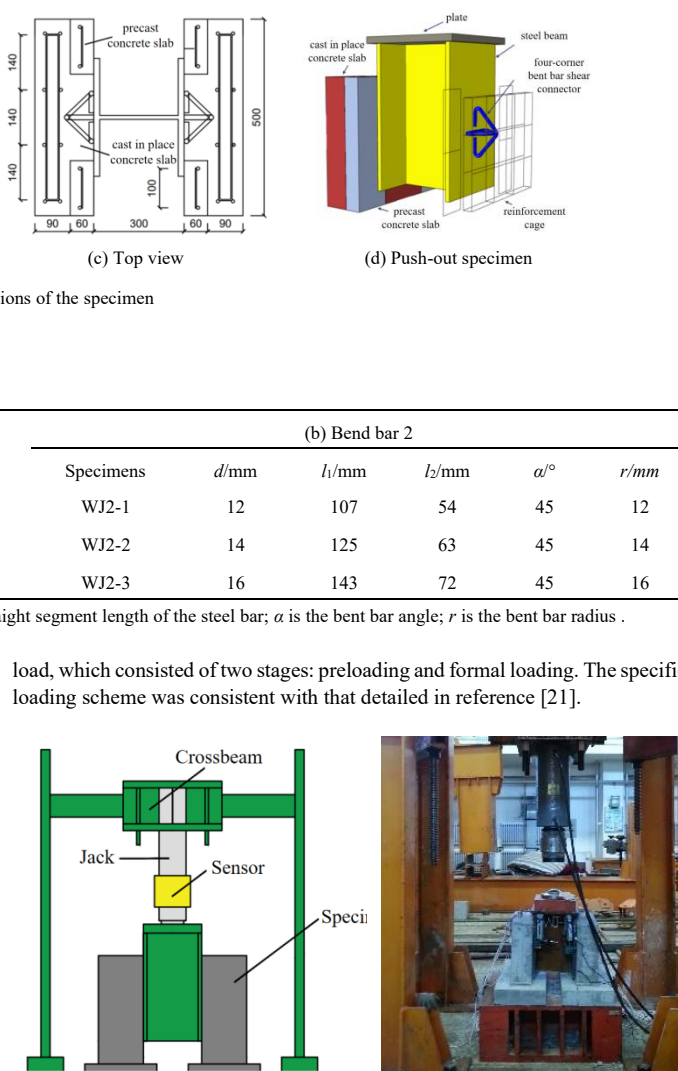


Fig. 3 Test setup

2.4. Measurement scheme

The arrangement of the strain gauges is presented in Fig. 4(a). Strain gauges 1-4 were located on the surface of the steel bars at the welding position of steel beam, while strain gauges 5-8 were located on the surface of inclined steel bars. As depicted in Fig. 4(b), two displacement transducers were set at the parallel and perpendicular positions to the welded joint between the mid-span steel beam and connectors, to measure the relative slip at the connector-steel beam interface, alongside vertical relative displacement between the H-shaped steel beam and concrete slab.



Fig. 4 Arrangement of strain gauges and displacement meters

3. Experimental analysis

3.1. Experimental phenomena and failure modes

The failure modes of specimens in three groups were similar, as shown in Fig. 5. In all cases, there was a significant sliding between the composite floor slab and H-shaped steel beam. Meanwhile, shear failure was observed at the weld joint connecting the steel beam and four-corner bent bar shear connectors, accompanied by spalling of the concrete surface.

Taking WJ-6 as the representative specimen, the experimental phenomena

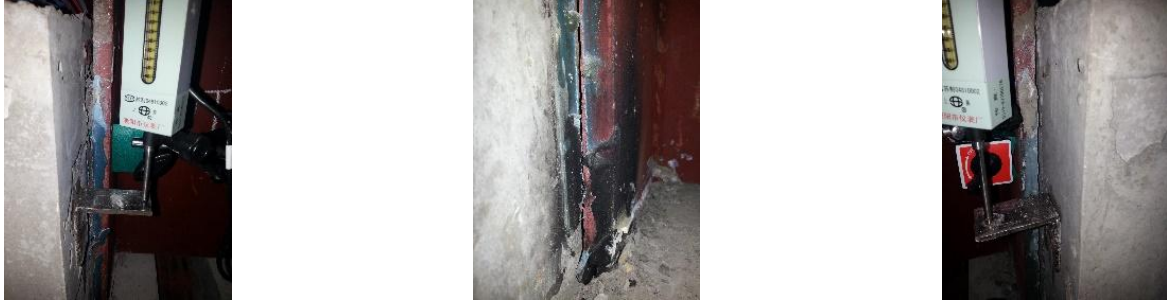


Fig. 5 Failure modes of specimens

3.2. Load versus slip curve

The load-slip curves (P - δ curves) are presented in Fig. 6. It is observable that the loading processes and failure states of all specimens are substantially consistent and can be divided into three stages. Elastic stage: For the 14-mm-diameter four-corner bent bar connector, its initial stiffness outperformed that with 12-mm-diameter connector and 16-mm-diameter connector. In this stage, P - δ curves of the three specimens displayed a linear relationship, indicating that specimens were in elastic working phase. As the loading process proceeds, the slope of the curve begins to decrease, the stiffness of the connector gradually reduces, and the specimen enters the elastoplastic stage. In this stage, the load increased slowly and no significant changes occurred in the specimen when the load reached the peak load. Descending stage: Continued loading, the load began to decrease and the slip value increased rapidly.

The characteristic slip values of all specimens range from 6.04mm to 11.48mm, which meet the slip value requirements (not less than 6mm) for flexible connectors in Eurocode 4 [20]. Therefore, the four-corner bent bar connector belongs to flexible connectors. As the steel bar diameter gradually increased, both the ultimate displacement and peak load increased significantly. The yield load increased by approximately 24.3% and 25.0% respectively, with the most remarkable improvement occurring when the

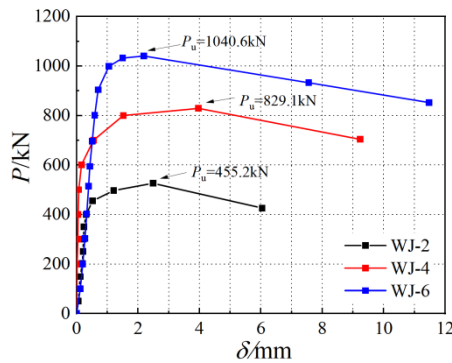
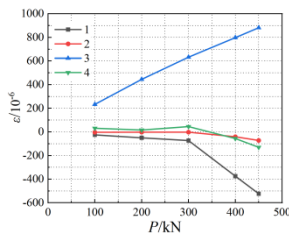
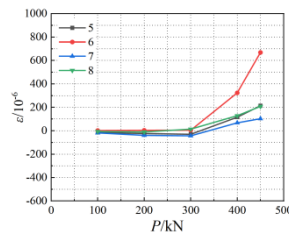


Fig. 6 P - δ curves



(a) Surface of horizontal bar of WJ-2



(b) Surface of diagonal bar of WJ-2

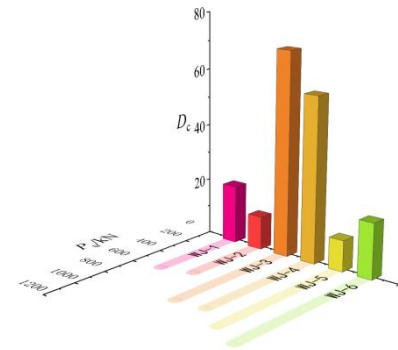
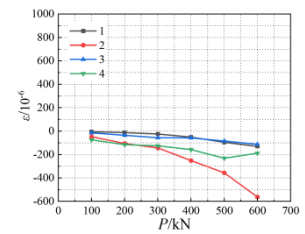


Fig. 7 P - D_c curves



(c) Surface of horizontal bar of WJ-4

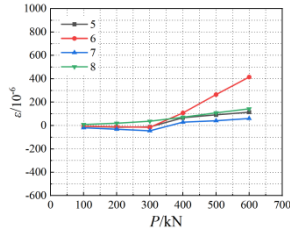
were elaborated. In the initial loading stage, there was no significant change in the specimen. When load reached 77% P_u , a slight sound was heard from interior of the composite slab, and a small amount of concrete blocks fell off at connection between the composite slab and steel beam. When load reached P_u , no obvious phenomena appeared. Continuing loading, a large amount of sliding occurred between the slab and beam, leading to severe damage of the specimen. The loading was terminated once the load reached 80% P_u . It was observed that the specimen ultimately failed via shear failure at the interface of the steel beam and composite slab. Partial concrete spalling appeared on the contact interface, while no obvious cracks were observed in the remaining concrete areas.

diameter increased from 12mm to 14mm, where the ultimate displacement and peak load increase by 34.6% and 36.5% respectively.

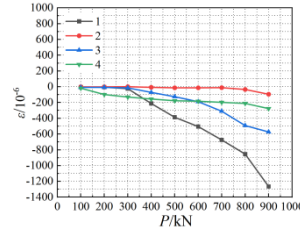
Fig. 7 depicts the bearing capacity and ductility coefficient (D_c)[22] curves. The ductility coefficients of specimens with different diameters were 11.6, 57.7, and 19.5 respectively. After reaching the peak load, all specimens exhibited excellent ductility performance during the failure stage. Notably, when the diameter increased from 12mm to 14mm, ductility improved 4.9 times.

3.3. Load versus strain curve

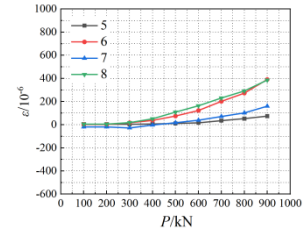
Fig. 8 presents the load-strain curves (positive values indicate tension; negative values indicate compression). An analysis of the curves indicated that the horizontal steel bars of all the specimens were in compression, while the diagonal steel bars were in tension. The strain gauge (2) on the upper surface of horizontal steel bars in WJ-2 was in tension. When compared with other strain gauges, it is speculated that the failure of the strain gauge might have caused the error. Based on the material property tests, the yield strains of 12mm, 14mm and 16mm steel bars were 1721, 1694 and 1922 respectively. By comparing the curve results, it can be seen that none of the bent bars of all the specimens yielded.



(d) Surface of diagonal bar of WJ-4



(e) Surface of horizontal bar of WJ-6



(f) Surface of diagonal bar of WJ-6

Fig. 8 Load-strain curves

4. Finite element analysis

4.1. Model establishment and validation

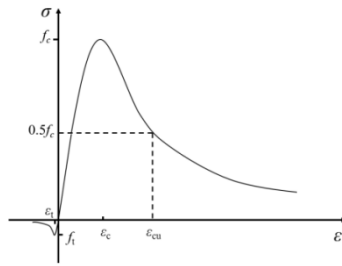
4.1.1. Material constitutive relation

A detailed numerical simulation analysis of mechanical performance of push-out specimens were conducted using ABAQUS. The constitutive model of the concrete was defined by the stress-strain curve stipulated in Appendix C of GB50010-2010 [23], as depicted in Fig. 9(a). For steel, a bi-linear elastic-plastic constitutive model [24] was utilized, as shown in Fig. 9(b).

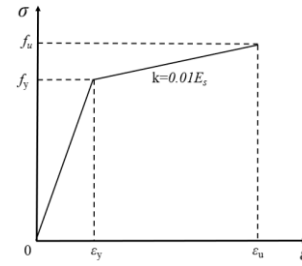
4.1.2. Element type selection and grid division

The models encompass precast concrete slab, cast-in-place concrete slab, H-shaped steel beam, four-corner bent bar shear connectors, and steel reinforcement frameworks, with the geometric dimensions of each member

corresponding to those of the test specimens. The concrete slabs and H-shaped steel beam were simulated utilizing three-dimensional solid eight-node hexahedral (C3D8R) elements, and the contact surfaces of slabs and connectors were modeled using three-dimensional solid four-node tetrahedral (C3D4) elements. The four-corner bent bar shear connectors were simulated with C3D8R, and the steel reinforcement framework was simulated via three-dimensional two-node (T3D2) truss elements [25]. In terms of grid division, mesh sensitivity analysis was performed to determine a global mesh size of 20 mm for the entire component, aiming to improve computational efficiency and facilitate model convergence. Meanwhile, to accurately simulate the mechanical behavior at the connection zone, mesh refinement was implemented for the contact interface between the concrete and the steel beam as well as the connectors. A mesh size of 2.5 mm was assigned to the concrete and the steel beam within this contact region, whereas a mesh size of 5 mm was used for four-corner bent bar connectors. The specific grid division is shown in Fig. 10.

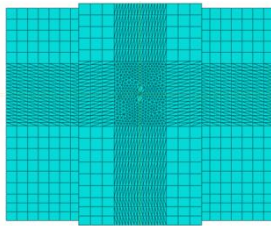


(a) Concrete

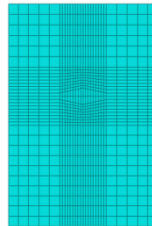


(b) Steel

Fig. 9 Material constitutive model



(a) Cast-in-place concrete slab



(b) H-beam



(c) Four-corner bent bar connector

Fig. 10 Grid division diagram

4.1.3. Contact relations and boundary conditions

Tie constraints were applied between precast concrete slabs and cast-in-place concrete slab, as well as between steel beams and connectors. Hard contact was used between precast concrete slabs and steel beam. Surface-to-surface contact was defined for interfaces of cast-in-place concrete slabs with steel beam and connectors, among which, hard contact was set in the normal direction and penalty friction coefficients in the tangential direction were 0.2 and 0.4 respectively. To guarantee the convergence of the model, contact control was executed at the interface between cast-in-place concrete slab and four-corner bent bar connectors, and embedded region constraints were imposed on the steel reinforcement frameworks. The bottoms of precast concrete slabs and cast-in-place concrete slabs were subjected to fully fixed constraints, while the upper surface of the beam was coupled and simulated using displacement loading. The specific modeling approach is illustrated in Fig. 11.

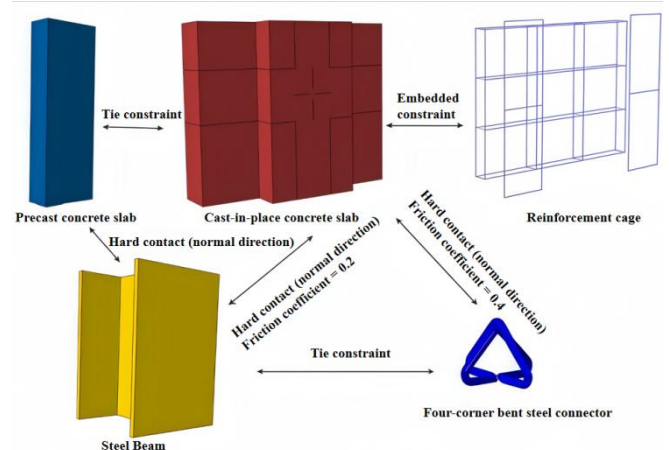


Fig. 11 Contact relations and boundary conditions diagram

4.1.4. Modelling verification

The comparison between experimental results and load-slip curves obtained from finite element simulation (Fig. 12 and Table 3) indicates that the overall trends of the simulated load-slip curves agree well with the experimental

data. Excellent consistency is observed in initial stiffness, and the relative error of peak loads is within 10%. The above comparisons verify the reliability of the proposed modeling method, laying a solid foundation for subsequent parametric investigations.

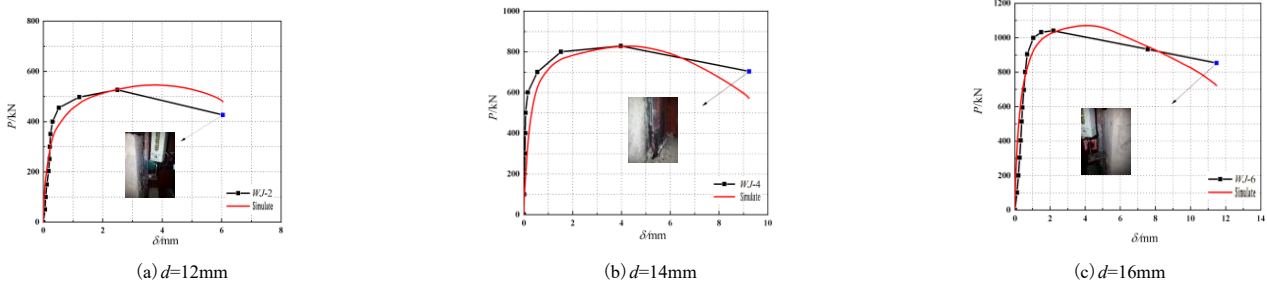


Fig. 12 Comparison of P - δ curves between test and finite element

Table 3

Table of test and FEM results

Specimen Number	Test		FEM	
	P_u /kN	Mean	$P_{u,FEM}$ /kN	$P_{u,FEM}/P_u$
WJ-1	576.3	551.5	545.7	0.99
WJ-2	526.7			
WJ-3	846.7			
WJ-4	829.1			
WJ-5	950.6	995.6	1070.6	1.07
WJ-6	1040.6			

Note: P_u denotes peak load of the test; $P_{u,FEM}$ represents the peak load obtained from the finite element model.

4.2. Analysis of full mechanical response process

Taking WJ-4 as the typical specimen, corresponding numerical analysis model was established. The P - δ curve acquired via numerical simulation (Fig. 13) revealed three distinct stages demarcated by three characteristic points: the elastic working stage (OA), the elastoplastic working stage (AB), and the descending stage (BC). Point A represented the yield load at position S of the connector (where the lower bending point of the longitudinal horizontal steel bar of the connector was defined as point S). Point B represented the peak load (P_u) at position S of the connector. Point C was the termination point of loading. The stress and strain distributions of components at the three characteristic points are presented in Figs. 14 to 16.

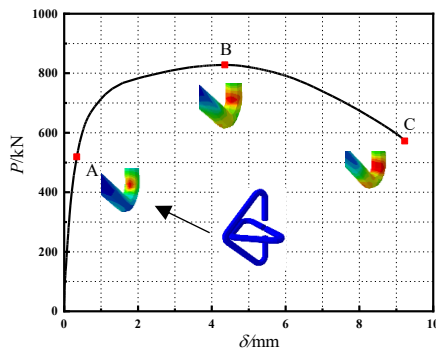


Fig. 13 P - δ curve

From the initial loading stage up to point A, the component demonstrated high initial stiffness, and the four-corner bent bar connectors remained in the elastic stage. Upon reaching point A, considerable stress was generated along the steel beam from the loaded end up to the welding surfaces with transverse bars of the four-corner bent bar connectors (Fig. 14(b)). It can be seen from Figs. 14(c) to 14(e) that extrusion contact emerged at the interfaces between the bent parts of transverse horizontal bars and diagonal bars of connectors and the cast-in-place concrete slab, where local regions reached the ultimate compressive strain. Simultaneously, extrusion contact also occurred at the upper bends of longitudinal horizontal bars and diagonal bars of connectors, as well as at the welding positions between the undersides of longitudinal horizontal bars and steel beams. However, almost no strain was generated between the cast-in-place

concrete slab and precast concrete slabs. The transverse horizontal bars near the bending point, as well as the longitudinal horizontal bars, and the longitudinal inclined bars all experienced significant stress. Among them, the inner side of the transverse horizontal bars near the bending point reached the yield load and began to exhibit local yielding phenomenon (Fig. 14(f)). At this point, the load is $0.72 P_u$, with a displacement of 0.49mm .

When loading from point A to point B, the component entered the elastoplastic working stage. Upon reaching point B, the component attained its ultimate load. Considerable compressive strains emerged in the cast-in-place concrete slab at the inner side of bending positions of the transverse horizontal bars, longitudinal horizontal bars, and longitudinal inclined bars of the connector, and some local strains even exceeded the ultimate compressive strain, indicating that the cracks had already occurred in the concrete slab at this point (Figs. 15(c)-15(e)). Large stresses were generated at the welded joints of steel beams and connectors (Fig. 15(b)), and local regions of the welded joints of the transverse horizontal bars reached the yield load (235MPa). Meanwhile, the bent regions of transverse horizontal bars, longitudinal horizontal bars and inclined bars all reached the yield load. Notably, the lower inner side of the bending section of the longitudinal horizontal bars locally reached the ultimate tensile stress (546.3MPa), indicating that local failure and damage had occurred in the steel bars here (Fig. 15(f)).

When loading from point B to point C, the component entered the descending stage. Upon reaching point C, large strains were observed at the welding joints between the steel beams and the horizontal bars, as well as at the bending positions where the longitudinal horizontal bars met the inclined bars (Fig. 16(b)). At the same time, the concrete in the immediate vicinity of the horizontal bars of the connectors and the bending points of these bars, as well as the longitudinal horizontal bars and their bending points, all reached the ultimate compressive strain (Figs. 16(c)-16(e)). Compared with the test results, no obvious cracks appeared in the concrete slabs, and the phenomenon remained consistent. The upper portions of transverse horizontal bars, longitudinal horizontal bars and inclined bars, as well as the lower segments of inclined (from the bending point of the horizontal bars to 1/3 of the length of the inclined bars) all generated large stresses. The stress at the upper bent part of the longitudinal horizontal bars was greater than that at the lower bent part, and it failed earlier than the transverse horizontal reinforcement (Fig. 16(f)). At this time, high weld stress existed at the welding interface between the connectors and the steel beam. The connector lost its shear capacity, which further induced shear failure of the component.

4.3. Extended parameter analysis

The bending angle of the four-corner bent bar connector designed in this paper was kept constant at 45° . By changing other variables to analyze the influence of different factors on performance of the four-corner bent bar connector, where the lengths of the horizontal bars ($1-l_1$, $2-l_1$), the longitudinal bars ($1-l_2$, $2-l_2$), and the height of the connector (H_1 , H_2), due to their special structural form, changed with the diameter of the bars. According to GB50010-

2010[23], the length requirements were as follows. Fig. 17 presents the schematic diagram of the four-corner bent bar connector. This study established 21 numerical models, and the selected parameters included: steel bar diameter d (ranging from 8 mm to 18 mm), precast concrete compressive strength f_{cu} (30MPa-70MPa), f_{cu} (30MPa-70MPa), and f_{yk} (300MPa-500MPa). The specific parameters are detailed in Table 4.

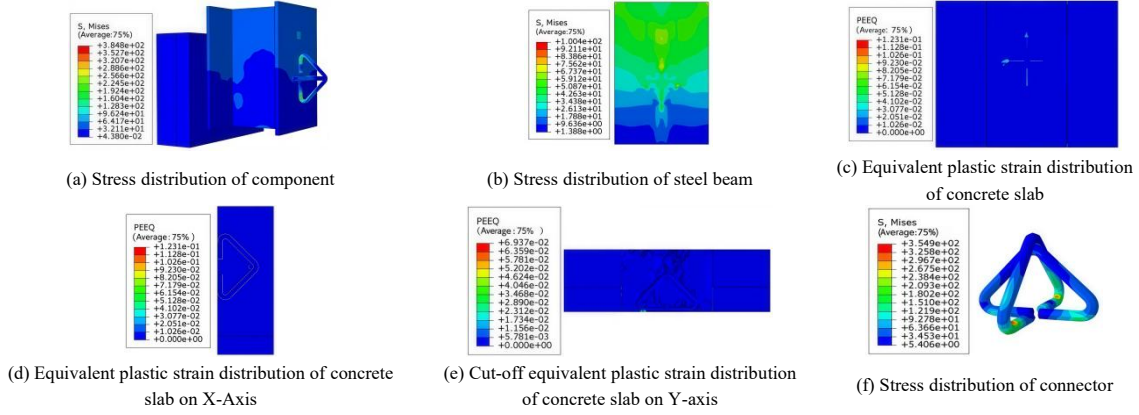


Fig. 14 Stress and strain distributions of WJ-4 (Point A)

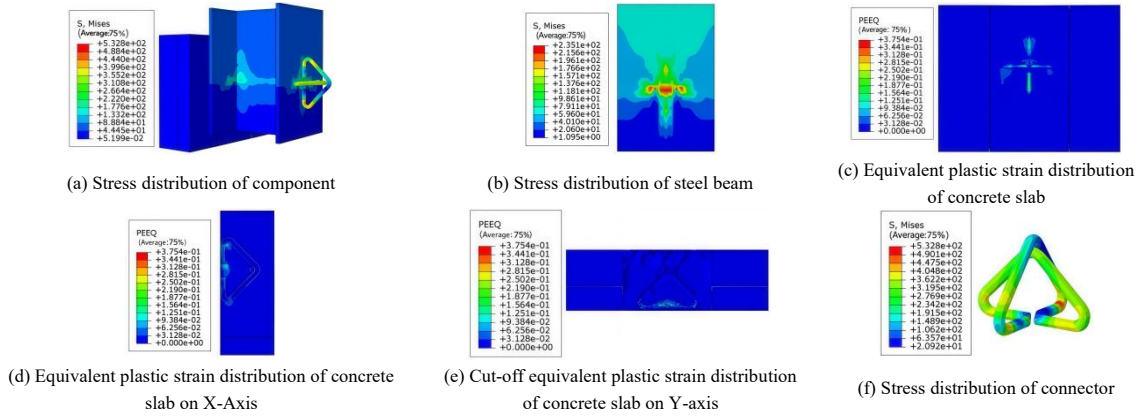


Fig. 15 Stress and strain distributions of WJ-4 (Point B)

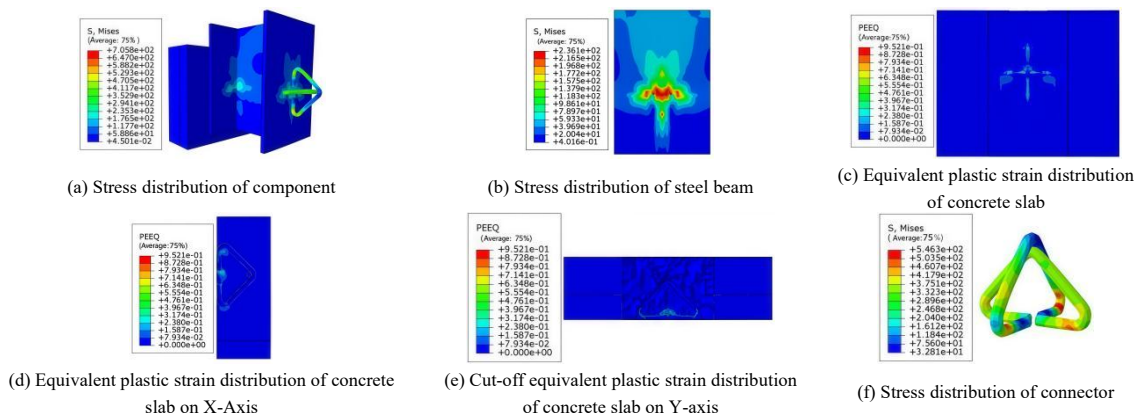


Fig. 16 Stress and strain distributions of WJ-4 (Point C)

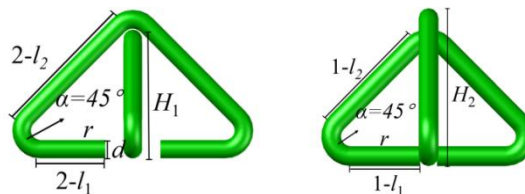


Fig. 17 The four-corner bent bar connector

Table 4

Details of components

Component number	d/mm	r/mm	$1-l_1/\text{mm}$	$1-l_2/\text{mm}$	$2-l_1/\text{mm}$	$2-l_2/\text{mm}$	f_{cu}'/MPa	f_{cu}/MPa	Grade	$P_{ub,FEM}/\text{kN}$
WJS-1-1	8	8	28	44	28	55.5	30	40	HPB300	222.1
WJS-1-2	10	10	35	55	35	69.5	30	40	HPB300	235.5
WJS-1-3	12	12	42	66	42	83	30	40	HPB300	248.7
WJS-1-4	14	14	49	77	49	97	30	40	HPB300	276.9
WJS-1-5	16	16	56	88	56	111	30	40	HPB300	322.5
WJS-1-6	18	18	63	99	63	126	30	40	HPB300	355.7
WJS-2-1	14	14	49	77	49	97	30	40	HPB300	276.9
WJS-2-2	14	14	49	77	49	97	40	40	HPB300	278.5
WJS-2-3	14	14	49	77	49	97	50	40	HPB300	279.5
WJS-2-4	14	14	49	77	49	97	60	40	HPB300	280.2
WJS-2-5	14	14	49	77	49	97	70	40	HPB300	280.7
WJS-3-1	14	14	49	77	49	97	30	30	HPB300	256.8
WJS-3-2	14	14	49	77	49	97	30	40	HPB300	276.9
WJS-3-3	14	14	49	77	49	97	30	50	HPB300	292.9
WJS-3-4	14	14	49	77	49	97	30	60	HPB300	304.8
WJS-3-5	14	14	49	77	49	97	30	70	HPB300	314.2
WJS-4-1	14	14	49	77	49	97	30	40	HPB300	276.9
WJS-4-2	14	14	49	77	49	97	30	40	HRB335	282.2
WJS-4-3	14	14	49	77	49	97	30	40	HRB400	293.35
WJS-4-4	14	14	49	77	49	97	30	40	HRB500	307.75
WJS-4-5	14	14	49	77	49	97	30	40	HRB600	318.9

Note: $1-l_1$ represents the diagonal length of the transverse bar; $1-l_2$ indicates the straight length of the transverse bar; $2-l_1$ denotes the diagonal length of the longitudinal bar; $2-l_2$ indicates the straight length of the longitudinal bar; $P_{ub,FEM}$ is the bearing capacity of the four-corner bent bar connector obtained from the finite element model.

(1) Steel bar diameter (d)

As depicted in Fig. 18(a), when the steel bar diameter was incrementally increased from 8mm to 18mm, the shear capacity of components increased by 6%, 12%, 24.7%, 45.2%, and 60.2% respectively. Among them, the most substantial improvement occurred when d increased from 12mm to 14mm, increasing by 16.5%. This indicates that d is primary factor affecting shear capacity of the four-corner bent bar shear connectors. This is mainly attributed to two aspects: the excellent tensile performance of the steel bars themselves, and the unique spatial configuration of the four-corner bent bars, which provides effective confinement to the surrounding concrete and the concrete inside connectors, thereby significantly improving the shear capacity. Additionally, the ductility of components gradually increases as d increases. These results indicate that increasing d can effectively improve the shear capacity of components within a certain range. It is recommended that the steel bar diameter should not be less than 12mm.

(2) Precast concrete compressive strength (f_{cu}')

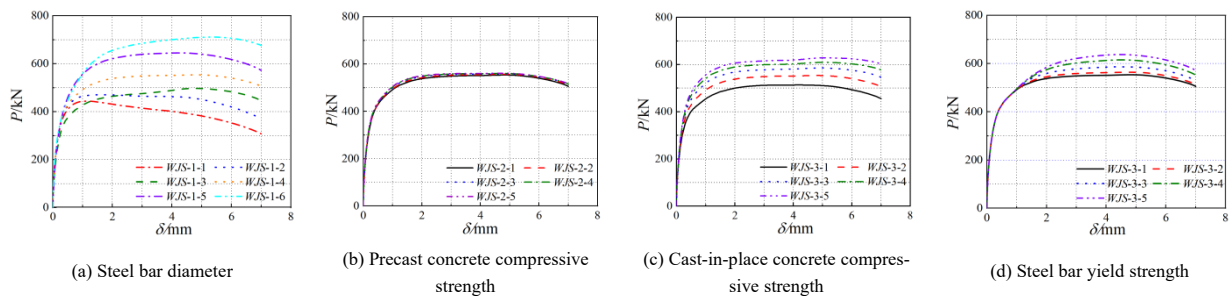
As shown in Fig. 18(b), when the precast concrete compressive strength was increased from 30MPa to 70MPa, the bearing capacity improved by 0.6%, 0.9%, 1.2%, and 1.4% respectively. The increase rate was less than 2%, which was considered negligible. Only when f_{cu}' was enhanced from 30MPa to 40MPa, does the bearing capacity increased relatively significantly. Therefore, considering both bearing capacity and economic benefits, it is recommended to select precast concrete with a compressive strength of 40MPa.

(3) Cast-in-place concrete compressive strength (f_{cu})

As presented in Fig. 18(c), when the cast-in-place concrete compressive strength was raised from 30MPa to 70MPa, the shear capacity of components increased by 7.8%, 14.1%, 18.7%, and 22.4% respectively. This demonstrates that f_{cu} is a crucial factor in enhancing bearing capacity of connectors. When the connector bears shear load, the load is primarily resisted by the bearing action of longitudinal bars, the concrete beneath longitudinal bars, and the concrete enclosed within the four-corner bent bars. As f_{cu} increases, the bearing capacity improves accordingly, which effectively enhances the shear behavior of components. However, the improvement effect of bearing capacity of four-corner bent bar connectors gradually decreases as f_{cu} increases (Fig. 3.19(b)). Therefore, it is recommended to select the cast-in-place concrete compressive strength according to construction requirements.

(4) Steel bar yield strength (f_{yk})

As illustrated in Fig. 18(d), the shear capacity of four-corner bent bar connectors increases non-linearly with the increase in the steel bar yield strength. When the strength grade of the steel bar was gradually upgraded from HRB300 to HRB335, HRB400, HRB500, and HRB600, the bearing capacity increased by 1.9%, 6%, 11.2%, and 15.2% respectively. This shows that connector with HRB400 steel bars exhibits a more significant improvement in shear capacity and better economic efficiency.

**Fig. 18** The influence of different parameters on shear capacity

4.4. Design suggestions of four-corner bent bar connector

The parameter analysis indicates that the shear performance of four-corner bent bar connectors is significantly affected by the steel bar diameter, the cast-in-place concrete compressive strength, and the steel bar yield strength, whereas the precast concrete compressive strength has a negligible impact. Therefore, the recommended design parameters are as follows: the steel bar diameter should be greater than or equal to 12 mm, and the grade of steel bar should be less than or equal to HRB400. Taking economic efficiency and structural requirements into account, when the steel bar diameter is 14 mm and the strength grade is HRB400, the four-corner bent bar connector exhibits better shear behaviour in connecting the prefabricated concrete composite floor slab to H-shaped steel beam. The strengths of other materials should be selected in accordance with design requirements.

5. Shear bearing capacity calculation

5.1. Calculation formula for shear capacity of bent bar connector

The European Code (EC 4) [20] takes into account factors such as the steel bar strength, geometric properties of connectors, and bonding characteristics between materials. The calculation considers not only the shear capacity but also the bond strength between the steel bar and concrete. The proposed formula focuses on analyzing the internal forces of truss web members and the local crushing concrete, as presented in the following equation:

$$P_x = A_u f_y \sqrt{\cos^2 \alpha + \frac{\sin^2 \alpha}{\gamma}} \quad (1)$$

Where: A_{st} denotes the cross-sectional area of the steel bar and f_y refers to its

yield strength; α is the angle between the steel bar and steel beam; γ is the resistance coefficient, typically ranging from 1.00 to 1.25.

When $\alpha \leq 45^\circ$, the expression is as follows:

$$P_r = \frac{A_{st} f_y}{1.2\gamma} \quad (2)$$

The American Concrete Institute (ACI) [26] has considered the frictional interaction between the steel beam and concrete, and concluded that shear force is transmitted through shear reinforcement, the following formula is provided:

$$P_x = A_x f_y (\cos \alpha + \mu \sin \alpha) \quad (3)$$

where μ is the coefficient of the resistance component, and $\mu = 0.7$.

Li [27] had analyzed the effect of flexible connectors in composite beams, indicating that these connectors not only transmit shear force but also control the deformation at the beam end. It is of vital importance to the overall stability and bearing capacity. Further, local pressure of the concrete at welded end of the bent bar was considered, and a formula was provided:

$$V_u = A_f \sqrt{(\cos \alpha + \mu \sin \alpha)^3 + \frac{1}{2} (\sin \alpha - \mu \cos \alpha)^3 + P} \quad (4)$$

Where: $p = \beta A_{st} f_y$, $\beta = 5.3 \sim 12$.

The Chinese Code (GB50017-2003) [28] only considers the strength effect of the steel bar itself and gives the following formula for the bearing capacity calculation:

$$N_{\pi}^k = A_{st} f_{st} \quad (5)$$

Table 5

Comparison of calculated values from various national standards and simulation results

Component number	$P_{ub,FEM}/kN$	μ_1	μ_2	μ_3	μ_4
WJS-1-1	222.1	0.07	0.08	0.46	0.07
WJS-1-2	235.5	0.10	0.12	0.68	0.10
WJS-1-3	248.7	0.14	0.16	0.92	0.14
WJS-1-4	276.9	0.17	0.20	1.13	0.17
WJS-1-5	322.5	0.19	0.22	1.27	0.19
WJS-1-6	355.7	0.22	0.26	1.45	0.21
WJS-2-1	276.9	0.17	0.20	1.13	0.17
WJS-2-2	278.5	0.17	0.20	1.12	0.17
WJS-2-3	279.5	0.17	0.20	1.12	0.17
WJS-2-4	280.2	0.17	0.20	1.12	0.16
WJS-2-5	280.7	0.17	0.20	1.11	0.16
WJS-3-1	256.8	0.19	0.22	1.22	0.18
WJS-3-2	276.9	0.17	0.20	1.13	0.17
WJS-3-3	292.9	0.16	0.19	1.07	0.16
WJS-3-4	304.8	0.16	0.18	1.03	0.15
WJS-3-5	314.2	0.15	0.18	0.99	0.15
WJS-4-1	276.9	0.17	0.20	1.13	0.17
WJS-4-2	282.2	0.19	0.22	1.24	0.18
WJS-4-3	293.4	0.22	0.25	1.42	0.21
WJS-4-4	307.8	0.26	0.30	1.69	0.25
WJS-4-5	318.9	0.30	0.35	1.96	0.29
AVE		0.18	0.21	1.16	0.17
STE		0.08	0.52	0.08	0.18

Note: μ_1 , μ_2 , μ_3 , and μ_4 represent the ratios of the calculated values P_c from the Equation 1, 3, 4, 5 to $P_{ub,FEM}$, respectively. AVE indicates the data average, while STE denotes the standard deviation.

As shown in Table 5, a comparison between the calculated values from various national standards and the simulation results. It can be seen that when the steel bar diameter is less than 10 mm, the results of four formulas for the bearing capacity are all relatively small. The main reason for this is that the four

formulas only consider the influence of a single steel bar on the bearing capacity. Specifically, Formula 2, Formula 3, and Formula 5 are overly conservative in their calculations of the shear bearing capacity for four-corner bent bar shear connectors, mainly due to the special structural form of concrete and the four-

corner bent bar was not considered for the influence on the bearing capacity. The calculation results of Formula 4 are relatively close, but the mean error is still large and the dispersion is high. The primary reasons for this phenomenon are twofold: the contribution of concrete to bearing capacity was neglected, and the effect of bent bar on shear capacity was considered twice.

5.2. Calculation formula for shear capacity of four-corner bent bar connector

Based on an investigation of the parametric influence laws of four-corner bent bar shear connectors and a summary of shear capacity formulas for steel bars presented in various national design codes, multivariate nonlinear regression analysis was carried out on the numerical results of 21 finite element models, and the following shear capacity calculation formula was obtained:

$$P_u' = (2.4A_{s2}f_r - 2.8A_{s1}f_r)(\cos \alpha + \phi \sin \alpha) + 0.27A_c f_c \quad (6)$$

Where: A_{s1} , A_{s2} represent the cross-sectional area of transverse steel bar and longitudinal steel bar, respectively; α is the bending angle (a value of 45° is recommended); A_c is the cross-sectional area of cast-in-place concrete slab; ϕ is the coefficient of the resistance component, and $\phi = 0.7$; 2.4, 2.8, 0.27 represent the weighted contributions of the transverse steel bar, longitudinal steel bar, and cast-in-place concrete slab, respectively.

Table 6

Comparison of calculated values and test results

Component number	P_{ub}/kN	Mean value	μ_1	μ_2	μ_3	μ_4	δ_1
WJ-1	288.2	275.8	0.15	0.17	0.96	0.14	1.10
WJ-2	263.4						
WJ-3	423.4	419	0.21	0.24	1.34	0.20	0.80
WJ-4	414.6						
WJ-5	475.3	497.8	0.31	0.35	1.99	0.29	0.78
WJ-6	520.3						
AVE			0.22	0.25	1.43	0.21	0.90
STE			0.08	0.09	0.52	0.08	0.18

Note: P_{ub} represents the bearing capacity of the four-corner bent bar connector obtained from the test. δ_1 represents the ratio of the calculated value P_u' in Equation 6 to P_{ub} .

Table 6 shows a comparison between the theoretical values calculated by the proposed equations and the measured experimental data. The results indicate that the shear capacity calculated using Equation 6 is in good agreement with experimental data, with an average error of 10% and a standard deviation of 0.18. Therefore, the proposed calculation formula can be effectively utilized to predict the shear capacity of the new-type connector. The applicable range is as follows: $d = 8 - 18$ mm, $f_{cu}' = 30 - 70$ MPa, $f_{cu} = 30 - 70$ MPa, $f_{yk} = 300 - 500$ MPa and bending angle of 45° .

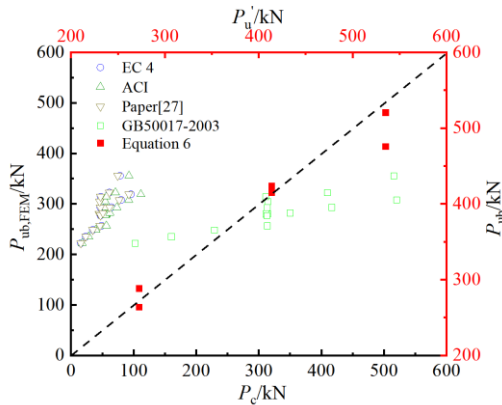


Fig. 19 Comparison of the simulated value of shear capacity of connector with the calculated value of national codes

6. Conclusions

1) The four-corner bent bar connector possesses relatively high initial stiffness and favourable ductility within the serviceability limit state. The characteristic slip values of all specimens vary from 6.04 to 11.48 mm, which meet the requirement of Eurocode 4 for flexible connectors (not less than 6 mm). Consequently, the four-corner bent bar shear connectors are categorized as flexible connectors.

2) The failure mode of every specimen was shear failure at the connection position between the four-corner bent bar shear connectors and the H-shape steel beam. Local concrete block detachment occurred at the welded interface of the composite slab and the steel beam.

3) Modifying the cross-sectional dimensions of the four-corner bent bar and material strength exerts varying degrees of influence on the shear bearing capacity of components. Among these factors, the steel bar diameter imposes the most prominent influence on the shear capacity, followed by the cast-in-place concrete compressive strength. In contrast, the precast concrete compressive strength has almost no effect. Recommendations for the optimal cross-sectional dimensions of the four-corner bent bar connectors are proposed with due consideration of shear performance, cost-effectiveness and structural detailing requirements.

4) Based on extensive numerical analysis results, the shear capacity calculation formula of four-corner bent bar shear connector is proposed through the method of multivariate linear regression. The theoretical predictions exhibit satisfactory agreement with measured test results.

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