Feasibility and performance of novel tapered iron bolt shear connectors in demountable composite beams

Citation for published version:
Song, S-S, Xu, F, Chen, J, Qin, F, Huang, Y & Yan, X 2022, 'Feasibility and performance of novel tapered iron bolt shear connectors in demountable composite beams', *Journal of Building Engineering*, vol. 53, 104528. https://doi.org/10.1016/j.jobe.2022.104528

Digital Object Identifier (DOI):
10.1016/j.jobe.2022.104528

Published In:
Journal of Building Engineering

Feasibility and Performance of Novel Tapered Iron Bolt Shear Connectors in Demountable Composite Beams

Sha-Sha Song[1], Fei Xu[2*], Ju Chen[1], Fengjiang Qin[2], Yuner Huang[3], Xin Yan[1]

1. Institute of Structural Engineering, Zhejiang University, Hangzhou, Zhejiang, China
2. School of Civil Engineering, Chongqing University and Key Laboratory of New Technology for Construction of Cities in Mountain Area (Chongqing University), Ministry of Education, Chongqing, China
3. School of Engineering, University of Edinburgh, Edinburgh, Scotland, United Kingdom

Abstract: The performance of tapered iron bolt (TIB) shear connectors and their feasibility to be used in demountable composite beams are investigated in this study. The typical TIB shear connector consists of six components, i.e., a connecting bolt, conical iron plug, top covering plate, top fixing bolt, retaining washer and inner sleeve. The connectors are expected to 1) reduce the initial slip between the concrete slab and steel beam, which generally exists for steel-concrete composite beams using bolted shear connectors, and 2) be capable of being replaced for composite beams in-situ without damaging other components. Two series of pushout tests were conducted on the specimens with TIB shear connectors in accordance with Eurocode 4 (EC4). The demountability of TIB shear connectors after severe corrosion induced by accelerated corrosion, i.e., wet-dry cycles, was examined. All pushout test specimens failed in shear fracture at the bolts while the concrete slabs remained undamaged. The test results demonstrated that the replacement of TIB connectors would not affect the failure mode of TIB connectors but have a moderate influence on the shear capacity. Therefore, a reduction factor was proposed to account for the influence of replacement in design. The design equations of TIB shear connectors under bolt shear and local concrete failure were proposed. The proposed design method is able to provide a reasonably accurate prediction for the failure mode and shear strength of the TIB connectors.

Keywords: Bolt shear connectors, Demountable, Load-slip relationship, Shear performance, Steel-concrete composite beams

---

*Corresponding author, E-mail: fei.xu@cqu.edu.cn.

1. INTRODUCTION

Steel-concrete composite beams have been widely applied in civil engineering, such as in buildings [1-3], floor systems [4] and bridges [5-8]. Compared with steel or reinforced concrete beams, the composite beam system has increased flexural strength and stiffness with reduced beam weight and depth due to composite action between the two materials [9,10]. Such composite action in steel-concrete composite beams is provided by shear connectors which transfer the shear force between the steel and concrete. In practical design, the welded shear studs are commonly adopted.

Under severe environmental conditions, concrete decks of bridges often suffer from deterioration caused by concrete cracking and corrosion of steel components, i.e., reinforcement and shear connectors [11,12]. Furthermore, previous research found [11] that the welds of the welded shear connectors were vulnerable to chloride-induced corrosion for steel-concrete composite beams. However, replacement of concrete decks to extend service life is almost impossible due to the welded shear connectors, such as the headed studs.

Meanwhile, with the increasing traffic loads on bridges nowadays, the capacity of existing non-composite bridges would fail to meet the capacity requirements for new traffic loads, which has been reported in North America [13-15] and Europe [16]. One solution to this issue is to exploit structural benefits from composite action by connecting the existing concrete decks to steel girders via shear connectors. However, this cannot be achieved by conventional welded shear connectors. Furthermore, when steel-concrete composite beams come to the end of their life cycles, the commonly used welded shear connectors would be the major obstacle to demolishing and reuse of steel beams. The sustainability and the concept of "3R", i.e., reduce, reuse, and recycle, in civil engineering, are becoming increasingly important in promoting the low carbon footprint worldwide.

To meet these practical demands, the high strength or high strength friction-grip bolts become an option to be adopted in composite beams [14,17-31] to allow for post-installation and demountability. The feasibility of applying high-strength friction-grip (HSFG) bolts in composite beams was firstly investigated by Dallam [32] and Dallam and Harpster [33]. Most studies on the bolted shear connectors were conducted in recent fifteen years. Kwon et al. [13-15] proposed and studied three types of post-installed shear connectors for retrofitting, which developed composite action for the existing non-composite bridges. To achieve sustainable construction and reuse of steel beams at the end of life cycles, previous researchers [17-26,34,35] conducted a series of experimental, numerical and theoretical studies on demountable composite beams with high strength bolts, HSFG bolts or blind bolts. The results demonstrated that demountable shear connectors had comparable shear capacities as welded studs but larger ultimate slips [21]. In addition to bolt diameter and grade, the pretension value would significantly affect the shear performance of the friction-grip bolted connectors [18]. Despite the fact that design methods [13,18,23,36] for the bolted connectors were proposed in previous research, there is no specific design method stipulated in current design codes.

In general, shear connectors in composite beams can be used in two ways, such as 1) concrete deck with embedded bolt heads connected with steel beam through preformed holes; 2) prefabricated concrete deck and steel beam with preformed bolt holes connected using bolt connectors. For the first type, as anticipated, it would be difficult to replace the shear connectors or concrete deck independently. For the second type, previous research [17,18,37,38] indicated that the bolt hole clearance could significantly influence the initial stiffness of HSFG bolts as a slip plateau at the early stage occurred, and this could be overcome by filling the construction clearance leading to a high initial shear stiffness.

In this study, a tapered iron bolt (TIB) shear connector is proposed to facilitate the replacement of shear connectors and concrete slab individually during life cycles and to achieve high initial shear stiffness without scarifying the constructional convenience. The feasibility of this novel TIB shear connector in demountable steel-concrete composite beams is assessed in terms of its demountability and shear performance of TIB connectors before and after replacement. Accelerated corrosion, e.g. wet-dry cycles, is conducted to simulate corrosive environmental action on the composite beams. Demountability after severe corrosion can also be evaluated. Twelve pushout specimens were tested to investigate their failure mode, shear capacity, stiffness and slip of the newly assembled and replaced TIB connectors. Based on the experimental results, the applicability of the existing design methods, which were developed for welded shear connectors, is assessed. Finally, a design method for TIB connectors is proposed with considerations of re-assembly influence. The design approach is expected to enable a safe and economical design of demountable TIB shear connectors in practical engineering.

2. TAPERED IRON BOLT (TIB) SHEAR CONNECTOR

The Tapered Iron Bolt (TIB) shear connector is proposed in this study. It is intended to be used in precast steel-concrete composite bridges to facilitate demountability and replacement of bridge decks at the end of their service life. Figs 1-2 present three-dimensional and plan views of the TIB shear connector, which consists of six components. The novel TIB shear connector is aimed to 1) reduce initial slip between the concrete slab and steel beam using bolted shear connectors; 2) increase shear stiffness of bolted connectors; and 3) be capable of replacing shear connectors without destroying the concrete decks in-situ. These can be achieved by reducing the gaps between each component and thus developing interfacial friction resistance with appropriate geometry design. For comparison, the three-dimensional view of the HSFG bolt shear connector is also presented in Fig. 1(e). As shown in Fig. 1(a), the conical iron plug was used to increase the interfacial surface between the shear connectors and the concrete slab. This could avoid local failure of the concrete around the shear connectors. Meanwhile, the adopted inner sleeve and retaining washer could fill the installation tolerances between each component, which could mitigate the initial slip between the steel beam and concrete slab.

The TIB shear connectors are placed within the inverted conical pockets in the precast concrete deck where the pockets are just fit to the outer dimension of the plugs. The typical arrangement of TIB shear connectors in a steel-concrete composite beam is illustrated in Fig. 3. The TIB shear connector is comprised of the inner sleeve, retaining washer, conical iron plug, top covering plate and two bolts. The nominal size and grade of the bottom connecting high-strength bolt were determined according to the design requirement of AISC360-16 [39], i.e., the shear strength of the steel anchors shall not exceed the limit states of concrete crushing and compression yielding of the steel beam.

Fig. 1(b) and Fig. 2(a) present details of the conical plug and top covering plate. The conical iron plug with 1:10 taper is designed to avoid being pushed out from the bottom of the concrete pocket.

The fully threaded bolt holes are drilled from both the top and bottom ends of the conical plug to match the fixing and connecting bolts, respectively, and the hole depth is three times the corresponding bolt diameter. The top covering plate is 12 mm larger than the plug top surface in diameter. On the other hand, the pressure on the interface between the plug and concrete slab is developed by connecting a top circular cover plate with the conical iron plug using an M16 Grade 6.8 bolt, which enables the conical iron plug to be connected with the concrete slab.

Relatively larger bolt holes are drilled on flanges of steel beams to mitigate the construction difficulty that commonly occurs in precast and prefabricated structures. The hole diameters used in this study are 22 ± 0.5 mm and 26 ± 0.5 mm for M12 and M16 bolts, respectively. A pair of specially designed filters, i.e., the inner sleeve and retaining washer, is adopted to fill the gap between the steel beam and bolt matching. This could minimise the initial slip between the steel beam and concrete slab after overcoming the interfacial friction force. The three-dimensional view and details are presented in Figs. 1(c)-(d) and 2(b)-(c). The retaining washer has an extended sleeve and a conical bolt hole matching the inner conical sleeve. The conical inner sleeve was used to further fill the gap between the retaining washer and bolt shank. To strike a balance between constructional tolerance and minimising the gap between each component, a fine seam of width 1.0 mm and 1.33 mm for the retaining washer and inner sleeve, as shown in Figs. 2(b) and (c), respectively, is designed in both the retaining washer and sleeve, which is capable of achieving a slightly tight contact between each component. A fully threaded high strength (Grade 8.8 or higher) bolt is used to connect the steel beam with the concrete slab via the conical iron plug. The bolt pretension is applied according to Eq. (1) [40]. It should be noted that the lubrication conditions on the bolts and threaded holes before tightening were in accordance with the specifications in ISO5049-1994 [40] that the paint, oil and rusts on the mating surfaces of the bolts and threaded holes shall be cleaned. After the conical plug is assembled to the steel beam, the same grade fine aggregate concrete with the precast slab is filled into the pocket in situ. The top covering plate can also prevent the fine aggregate concrete from the inside of the TIB connectors.

$$F_{\mathrm{pre}} = 0.7 f_{\mathrm{y}} A_{\mathrm{e}} \tag{1}$$

where F_{pre} is the preload in ISO5049-1994 [40], f_{y} is the yield strength of the high strength bolt, $A_{\mathrm{e}} = \pi D_{\mathrm{e}}^2 / 4$ is the effective cross-sectional area of bolt, and D_{e} is the effective bolt diameter.

The geometry and material properties of TIB shear connectors are designed according to construction practice. In this study, M12 and M16 TIB shear connectors of Grade 8.8 are adopted, and the configuration details and material properties are provided in Figs. 1-2 and Table 1, respectively.

3. EXPERIMENTAL INVESTIGATION

3.1. Test specimens

The pushout tests were arranged in accordance with Eurocode 4 (EC4) [41] to investigate the shear performance of TIB shear connectors. The shear resistance, load-slip relationship and stiffness of M12 and M16 Grade 8.8 TIB shear connectors and the effect of replacement on the shear performance of the specimens were assessed. In total, twelve pushout specimens were tested, and three identical repeat specimens were arranged for each series. The pushout test specimens were designed based on the recommendations in EC4, which were developed for welded stud shear connectors. The dimension details and reinforcement layout of the test specimens are shown in Fig. 4. The test specimens consisted of an HM 250×250×9×14 steel beam and two precast concrete slabs of 700-mm height, 600-mm width and 150-mm thickness in size, as shown in Figs. 4(a)-(d). The concrete slabs were connected to the beam flanges through four TIB shear connectors at both sides. The arrangement of the oversized preformed bolt holes on the steel beam for both M12 and M16 TIB shear connectors is presented in Fig. 4(c). All concrete slabs had an identical reinforcement layout and were cast horizontally using the same concrete mix. Hot-rolled ribbed bars HRB335 were adopted, and four pockets to install the conical iron plugs were preserved, as shown in Fig. 4(d). The steel material properties of the TIB connector components, reinforcement and steel beam, are presented in Tables 1 and 2. The material strengths (f_{y} and f_{u}) and elastic modulus (E_{s}) of the bolts and steel beam were determined by the monotonic tensile tests, and the tensile coupons were from the reference steel of the same batch with the test components. Meanwhile, the material properties of the other TIB connector components and reinforcement shall be provided by the manufacturer. The 28-day cubic compressive strength (f_{cu}) and elastic modulus (E_c) were 48.2 MPa and 29.5 GPa, respectively. The test specimens were labelled, such as the connector name, high-strength bolt diameter, bolt status and specimen number. The letter N and R are referred to as 'New' and 'Replaced', respectively. For example, TIB12N-1 and TIB16R-2 referred to the first test specimen with newly assembled M12 Grade 8.8 TIB shear connectors and the second test specimen with replaced M16 Grade 8.8 TIB shear connectors, respectively.

The fabrication procedure of the test specimens is presented in Fig. 5. The concrete slabs were demolded after two days, and the embedded parts for the preserved pockets were removed after 28-day standard curing in the laboratory. Subsequently, the pushout test specimens were assembled using TIB shear connectors. The conical iron plugs were first placed into the pockets (in Fig. 5(c)), and the covering plate was connected to the iron plug using an M16 Grade 6.8 bolt. In Figs. 5(d)-(e), the concrete slabs were subsequently connected with the steel beam using the connecting bolt. Four bolts were tightened in a diagonal sequence to evenly install the bolts on the same side, and a manual torque-wrench was used to check the applied torque values. Finally, fine aggregate concrete was filled in the preserved pockets. A completed test specimen is shown in Fig. 4(e) and Fig. 5(f). The disassembly of the test specimen was carried out in a reversed order as the assembly procedure.

3.2. Test setup

The pushout tests were conducted with a 10,000 kN universal testing machine. The photo and schematic view of the test setup are presented in Fig. 6. The top surface of the steel beam for each test specimen was grinded. A thick steel plate with a spherical hinge fitted with the actuator was placed on the top of the steel beam to sustain the uniformed compressive load on the cross-section. Four displacement transducers and two dial indicators were used to measure the vertical slip and transverse separation between the concrete slab and beam flange, respectively. The slip was measured as the relative displacement between the thick plate and concrete slab. The separation between the concrete slab and beam flange was measured at both sides of two different bolt rows. Details of the instrumentation arrangement are shown in Fig. 7.

3.3. Loading protocol

In this study, a two-step standard loading schedule recommended in EC4 [41] was adopted. The loading protocol is presented in Fig. 8(a). The loading schedule consists of two steps. Firstly, the load was applied to 40% of the expected failure load and then cycled 25 times between 5% and 40% of the expected failure load, where the expected failure load F_{u_AISC} was calculated based on the design calculation method for welded shear studs stipulated in AISC360-16 [39], as expressed in Eq. (2).

$$F_{u\_AISC} = (R_g R_p A_e f_u) N_b \quad (2)$$

where, f_u is the ultimate tensile strength of the connecting bolt, R_g and R_p are determined in accordance with AISC360-16, which focus almost exclusively on cases involving the use of headed stud anchors welded through the steel deck [42], and in this study $R_g = 1$ and $R_p = 0.75$ were adopted. N_b is the number of high-strength bolts in test specimens. The first and second loading steps were under force control at a rate of 1kN/s and displacement control at a rate of 0.2 mm/min, respectively. All test specimens were loaded to failure in the second step, and the test was stopped when the load dropped to 80% of the tested peak load. To assess the demountability of the test specimens subjected to a corrosive environment, two groups of test specimens, i.e., the TIB12R and TIB16R groups, were loaded to 60% of the expected failure load F_{u_AISC}, and then unloaded. After unloading, they were subject to accelerated corrosion, i.e., wet-dry cycles. In the accelerated corrosion period, 3.5 wt.% NaCl solution was sprayed to the test specimens at the interface of the concrete slab and steel beam and the top covering plate every 48 hours. The test specimens were naturally dried in each spraying interval. The accelerated corrosion lasted for 180 days with 90 wet-dry cycles. After that, the corroded TIB shear connectors (in Fig. 9) were removed. This also demonstrated the removability of shear connectors after severe corrosion. The test specimens were re-assembled using new TIB connectors. It should be noted that only the TIB connectors were replaced. The re-assembled test specimens were tested following the standard two-step loading scheme as shown in Fig. 8(a). The whole load programme for this series is illustrated in Fig. 8(b).

4. TEST RESULTS AND DISCUSSIONS

4.1 Failure mode

The failure mode of all newly assembled specimens was shear fracture of high strength connecting bolt at the interface of the concrete slab and steel beam. The load decreased suddenly with a pronounced sound indicating specimen failure. The failed connecting bolts of these test specimens are presented in Fig. 10(a). Two or more connecting bolts were fractured for each specimen, and the bolt fracture was found initiated from the bottom first row. This could cause a load drop and progressive failure of the rest bolts due to the sudden increase of shear force to the bolts. Shear deformation near the fracture region of each failed connecting bolt was found. The concrete crush, which has been reported in the previous experimental research [11,21,24,26] for many types of bolted shear connectors, did not occur around the conical iron plug, as shown in Fig. 10(b). This can be attributed to the enlarged contact area leading to a higher local bearing capacity. Meanwhile, the sound concrete slabs indicated the potential to reuse the slab with replaced shear connectors in demountable composite bridges. For the test specimens with all TIB shear connectors replaced after six-month accelerated corrosion, the failure mode was similar to the newly assembled groups. This demonstrated that the replacement of shear connectors would not alter the failure mode.

4.2 Shear capacity

The ultimate load of each specimen F_{Exp} and shear capacity of individual TIB connectors V_{Exp} are presented in Table 3. The shear capacity of an individual TIB connector was determined based on the hypothesis that each connector of the test specimen was sustained the same shear force. The variation of the shear capacity among three identical test specimens in each group is rather small, with the maximum coefficient of variation being 0.047. The average shear capacity ratios of the M12 to M16 series for the newly assembled and replaced ones, i.e., V_{M12_Ave}/V_{M16_Ave} and $V_{RM12\_Ave}/V_{RM16\_Ave}$, are 0.599 and 0.566, respectively, which are larger than the ratio $f_{y\_M12}A_{e\_M12}/f_{y\_M16}A_{e\_M16}$ (=0.554). This also indicated that the shear stresses of TIB connectors for M16 groups were slightly less developed compared with the M12 groups. The comparison between the shear capacity and ultimate tensile strength ($A_e f_u$) of each connecting bolt is plotted in Fig. 11 and Table 4. The results demonstrated that the shear capacities of all TIB connectors ranged from 62.4% to 81.7% of the tensile strength $A_e f_u$. On average, the bolt shear strengths are 80.7% and 74.1% of the tensile strength $A_e f_u$ for the M12 and M16 newly assembled TIB groups, respectively. And for the replaced connectors, these ratios are 67.4% and 65.6% for the M12 and M16 bolts, respectively. The replaced connectors have relatively lower ratios than those newly assembled ones, with the capacity ratios of V_{RM12_Ave}/V_{M12_Ave}=83.6% and V_{RM16_Ave} / V_{M16_Ave}= 88.5% for the M12 and M16 series, respectively. The reduction of the ultimate resistance for the re-assembled specimens could be due to the microcrack induced damage of concrete during the replacement of TIB connectors. This could have a direct influence on the shear behaviour of TIB shear connectors, such as stiffness and slip, thus might cause a negative effect on the development of shear capacity for the connecting bolt.

4.3 Slip

The slip at the peak load S_{peak}, and maximum slip S_u based on EC4 [41] are summarised in Table 3. The maximum slip S_u was determined at 90% of the characteristic peak load after the load dropped. The characteristic peak load is the minimum peak load of the three identical specimens in the same group [41]. In general, these two slips S_{peak} and S_u varied more significantly for the replaced connectors than the counterpart newly assembled ones. Meanwhile, the replaced connectors could achieve the larger slips S_{peak} and S_u. For the newly assembled connectors, the discrepancy of the slip S_{peak} was larger for the M12 group than M16 one, which might be due to dimension quality control issues for the small size connectors. As shown in Figs. 12(a) and 13(a), the slips of the M12 and M16 newly assembly connector groups increased substantially after the shear load of 15 kN and 25 kN, respectively. It should be noted that the initial slip of the test specimen TIB12N-1 increased significantly at the beginning, and this could be attributed to the early developed separation between the concrete slab and beam flange, as shown in Fig. 12(b). Furthermore, the average values of S_u for each group were less than 6 mm, which is the minimum slip requirement for welded shear studs stipulated in EC4 [41]. Only one specimen TIB16R-1 with the replaced connectors met this minimum slip requirement, as shown in Fig. 13(b).

4.4 Shear stiffness

The shear stiffness was determined as the secant stiffness between zero and specific points from the load-slip curves. In this study, three different stiffness values were calculated at 0.2 mm slip and 50% and 70% of the shear strength, i.e., $K_{ini\_0.2mm}$ [29,43], $K_{ini\_0.5Fu}$ [27,38,44] and $K_{ini\_0.7Fu}$ [29], were adopted for comparison, as presented in Table 3 and Fig. 14. The variations of these stiffnesses are significant, with the maximum coefficient of variation of three identical test specimens being 0.629. The stiffness differences of the identical specimens can also be identified in Fig. 14, where the symbols refer to the average stiffness value of each group and the error bars define the maximum deviation between the individual and mean values. In general, the stiffness of the M12 series was smaller than that of the M16 series, as anticipated. After replacement, the stiffness of TIB shear connectors decreased. Taking $K_{ini\_0.5Fu}$ as an example, the stiffness values of the replaced connectors were approximately 69% and 76% of the newly assembled connectors for the M12 and M16 groups, respectively. This indicated that the stiffness reduction should be considered in the practical design for the re-assembled composite beams.

On the other hand, the initial stiffness $K_{ini\_0.2mm}$ of high-strength friction-grip bolted shear connectors obtained from different studies [18,21,22,45,46] were less than 100 kN/mm, which were lower than the values measured in the present study, i.e., 178.96 kN/mm and 155.00 kN/mm for newly and re-assembly TIB connectors, respectively. The comparison of initial stiffness demonstrated that the reduction of initial slip was achieved by using the proposed TIB shear connectors.

4.5 Transverse separation

The transverse separation between the steel beam and concrete slab could result in a negative influence on the shear performance of TIB shear connectors. However, this could inevitably exist in the traditional pushout test and cause the performance discrepancy between the test results and practical engineering due to the induced tension on the connectors. The load-separation relationships of TIB12N and TIB16N groups are shown in Figs 12(b) and 13(b), respectively. A minor discrepancy of the separations at two sides measured using D5 and D6, as shown in Fig. 7, was found with the maximum separation of less than 0.5 mm (except for the test specimen TIB12N-1). The early developed separation was found in the test specimen TIB12N-1, which might be caused by the poor installation levelling. This would also cause a significant initial vertical slip between the concrete slab and steel beam, as shown in Fig. 12(b). The maximum separations were 0.40 mm (except for TIB12N-1) and 0.42 mm for the M12 and M16 groups, respectively. In general, the transverse separation that occurred during the test was relatively small, indicating the satisfactory pushout resistance of the TIB connectors. Furthermore, the effects of separation on the shear performance of TIB connectors can be reasonably ignored. For the re-assembled specimens, the focus was placed on the shear performance, i.e., the initial stiffness, slip and shear capacity. The test results of newly-assembled specimens showed that the transverse separations were rather small. This demonstrated that the taper of 1:10 was adequate for the conical plug to resist the transverse separation. Therefore, the transverse separations of the re-assembled specimens using the same kind of conical plugs were not measured in the second-phase tests.

5. DESIGN RECOMMENDATIONS

5.1 Existing design calculation methods

There is no existing design calculation method for the shear capacity of the proposed TIB shear connectors. Design methods for the welded shear connectors are provided in current design provisions, such as EC4 [41], AISC360-16 [39] and GB50017-2017 [47]. For the specimens in this study, only shear fracture of the connecting bolts was observed, and no concrete slab failure occurred, such as spalling or splitting. Therefore, the applicability of existing design equations for shear failure of bolt connectors is assessed in this study. Design equations for connector shear fracture in EC4, AISC360-16, and GB50017-2017 are summarised below.

$$V_{\mathrm{EC4}}=\frac{0.8A_{\mathrm{e}}f_{\mathrm{u}}}{\gamma} \tag{3}$$

$$V_{\mathrm{AISC}}=R_{\mathrm{g}}R_{\mathrm{p}}A_{\mathrm{e}}f_{\mathrm{u}} \tag{4}$$

$$V_{\mathrm{GB}}=0.7A_{\mathrm{e}}f_{\mathrm{u}} \tag{5}$$

where V_{EC4}, V_{AISC} and V_{GB} are the bolt shear fracture strength in the design codes of EC4, AISC360-16 and GB50017-2017, respectively. γ is the partial factor in EC4.

In this study, the effective cross-section area was used in each equation for comparison. The partial factors in each design provision were not considered in the calculations, while all high strength connecting bolts fractured at the threaded portion. The shear strength predictions calculated from the design equations were compared with the test results, as presented in Fig. 15. The deviation of tested shear strength with each corresponding design prediction can be found in Table 4. For the newly assembled shear connectors, the Chinese steel structural design code GB50017-2017 provided the conservative predictions, whilst EC4 and AISC360-16 slightly overestimate the shear capacity for the M16 series and provide reasonable predictions on the safe side for the M12 series with mean values of $V_{\mathrm{AISC}}/V_{\mathrm{Exp}}$ and $V_{\mathrm{EC4}}/V_{\mathrm{Exp}}$ equal to 0.93 and 0.99, respectively. In general, the design equation in AISC360-16 provides the best predictions for the newly assembled TIB shear connectors. However, all design equations overpredicted the shear capacity of the replaced shear connectors. The maximum over-predictions were 22% for the TIB16R group.

5.2 Proposed design calculation method

5.2.1 Bolt shear capacity

As discussed in Section 4.2, the shear capacities of the replaced connectors are approximately 84% and 89% for the M12 and M16 series of those newly assembled ones, respectively. The reduction factor for the shear capacity should be adopted for the replaced connectors to account for the reduced shear strengths induced by the disassembly and re-assembly procedures. Therefore, the average shear strength ratio of the replaced to newly assembled ones was adopted as a strength reduction factor, i.e., $\gamma_{\mathrm{v}}=0.87$ for the replaced TIB shear connectors. For the newly assembled connectors, the design equation in AISC360-16 [39] for the connector shear capacity was adopted. Therefore, the proposed design equation of bolt failure for the TIB shear connectors is provided in Eq. (6).

$$V_{\mathrm{TIB}}=\gamma_{\mathrm{v}}R_{\mathrm{g}}R_{\mathrm{p}}A_{\mathrm{e}}f_{\mathrm{u}} \tag{6}$$

where V_{TIB} is the shear strength of the re-assembly TIB connector, γ_{v} is the reduction factor accounting for the replacement influence, $\gamma_{\mathrm{v}}=1.0$ and $\gamma_{\mathrm{v}}=0.87$ for the newly assembled and replaced TIB connectors, respectively. Due to the limited test data, the applicability of the proposed calculation equation for the shear bearing capacity of TIB connectors are limited to the range of test specimens. Therefore, the proposed design equation shall be applicable for TIB connectors with M12 and M16 connecting bolts of Grade 8.8.

5.2.2 Concrete local failure

To further optimise the geometry design of the TIB shear connectors in accordance with structural design requirements, the local failure capacity of concrete should be evaluated. The design strength for welded shear connectors under concrete local failure mode in AISC360-16 [39] is provided in Eq. (7). The design strength of welded shear connectors is, therefore, determined by the smaller value calculated from Eqs. (6) and (7).

Eq. (7) was proposed by Ollgaard et al. [48], which was derived from the regression of large quantities of test data of welded shear connectors, that the capacity was found to be proportional to the cross-section area of shear connectors. By adopting this equation for the TIB shear connectors, the concrete local failure strength V_{con} can be approximately determined as an equivalent strength. It can be calculated as the integration of Eq. (8) along the full connector height (H_{c}) dividing the connector height (H_{c}). The calculation schematic diagram is presented in Fig. 16.

$$V_{\mathrm{con\_AISC}}=0.5A_{\mathrm{e}}\sqrt{f_{\mathrm{c}}E_{\mathrm{c}}} \tag{7}$$

$$V_{\mathrm{con}} = \frac{0.5\cos\theta\sqrt{f_{\mathrm{c}}E_{\mathrm{c}}}}{H_{\mathrm{c}}}\int_{0}^{H_{\mathrm{c}}} A_{\mathrm{plug}}\,\mathrm{d}h \tag{8}$$

where f_{c} and E_{c} are the cylinder compressive strength and elastic modulus of concrete, respectively, H_{c} is the height of the conical iron plug, h is the distance from integration point to the circular bottom surface of the plug, A_{plug} is the cross-sectional area of the plug without considering the bolt holes, as expressed in Eq. (9).

$$A_{\mathrm{plug}} = \frac{\pi}{4} D_i^2 \tag{9}$$

Substituting $D_i = D_1 + h\tan\theta$ into Eq. (9), the equation can be simplified, and the concrete local failure strength can be determined in Eq. (10).

$$V_{\mathrm{con}} = \pi\cos\theta\sqrt{f_{\mathrm{c}}E_{\mathrm{c}}}\left(\frac{1}{8}D_1^{\,2} + \frac{1}{8}D_1 H_{\mathrm{c}}\tan\theta + \frac{1}{24}H_{\mathrm{c}}^{\,2}\tan^2\theta\right) \tag{10}$$

where D_1 is the diameter of the top face of the plug, θ is the conical degree presented in Fig. 16.

The shear capacity V_{pro} of the TIB shear connectors can be determined as the smaller strength calculated from the proposed Eqs. (6) and (10). The calculated shear capacity V_{pro} was compared with the test results in Table 5, with the mean value and CoV of 1.00 and 0.047, respectively. All the test specimens were assumed to be failed by bolt shear fracture. The comparison demonstrated that the proposed design method could accurately predict the shear capacity of both newly assembled and replaced TIB connectors.

Based on the design equation Eq. (10), the geometry of the conical iron plug can be optimised to achieve a balance between minimising the conical size and developing full shear strength of the high strength connecting bolt. Furthermore, the slip capacity could be improved by adopting a double-skin conical iron plug. The reduced lateral stiffness of the plug allows the development of bolt deformation before failure. Therefore, this may provide an adequate slip capacity at the ultimate status.

For construction feasibility in practical engineering, we would suggest setting a certain constructional tolerance for both the conical pockets and bolts, and the suggested tolerance values would be 5% of the larger diameter of the conical plug D_2 (in Fig. 2(a)) for conical pockets and 0.5 mm for bolt holes (in Fig. 4(c)) where the bolt hole diameter is equal to the outer diameter of retaining sleeve. It should be noted that the high initial stiffness would be compromised by the constructional tolerances, although we have made some solutions to mitigate the negative effects. On the other hand, the larger constructional tolerance could facilitate the more convenient assembly. Therefore, the optimal constructional tolerances should be further studied to have a balance between high initial stiffness and convenient construction for the TIB shear connectors.

6. CONCLUSIONS

The shear performance and feasibility of the novel tapered iron bolt (TIB) shear connectors in demountable steel-concrete composite beams were assessed in this study. The demountability of TIB shear connectors after severe corrosion was evaluated. Pushout tests on the specimens with newly assembled and replaced TIB shear connectors were conducted to assess the shear performance of the TIB connectors. The following conclusions have been drawn from this study.

(1) The pushout test results demonstrated that the failure mode of the newly assembled TIB shear connectors was bolt shear fracture at the interface of the beam flange and concrete slab, while the concrete slab remained undamaged at the ultimate load. On average, the bolt shear strengths were approximately 81% and 74% of the ultimate tensile strength for M12 and M16 newly assembled ones, respectively. Large shear stiffness and limited interfacial slip were found for TIB shear connectors in the tests. All newly assembled TIB connectors failed at a slip less than 6 mm.

(2) The TIB shear connectors in the test specimens were corroded under the accelerated corrosion method, i.e., wet-dry cycles. These corroded connectors were easily demounted and replaced with the concrete slabs and steel beam undamaged after the replacement. This demonstrated the replaceability of TIB shear connectors in the corrosive environment.

(3) The failure mode of the replaced TIB shear connectors was identical to the newly assembled

ones, while the reductions were found in the shear capacity and stiffness. It is suggested that the shear capacity and stiffness reductions of the replaced TIB shear connectors should be considered in practical design.

(4) The applicability of design equations for the welded shear connectors at shear failure in the current design codes, i.e., EC4, AISC360-16, GB50017-2017, were assessed by comparing the predictions with test results. The Chinese steel structural design code GB50017-2017 provided the most conservative predictions for the newly assembled TIB connectors. However, these three design provisions overpredicted the shear capacity of the replaced TIB shear connectors.

(5) A reduction factor was adopted into the design equation for the shear failure of connectors in AISC360-16 to predict the shear capacity of the TIB connectors, accounting for the influence of replacement. The design equation for the TIB connectors that failed at concrete local failure was proposed related to the geometry of the conical iron plug and concrete properties, which can be used to optimise the TIB shear connectors. The proposed design method can well predict the failure mode and shear capacity of both the newly assembled and replaced TIB connectors.

Acknowledgements

The research work described in this paper was supported by the National Natural Science Foundation of China (52108116 and 52078249).

References

[1] Wang YH, Nie JG. Effective flange width of steel–concrete composite beam with partial openings in concrete slab. Materials & Structures, 2014, 48(10):3331-3342.
[2] Wang YH, Yu J, Liu JP, et al. Experimental study on assembled monolithic steel-prestressed concrete composite beam in negative moment. Journal of Constructional Steel Research, 2020, 167:105667.
[3] Nie JG, Tao MX, Nie X, et al. New technique and application of uplift-restricted and slip-permitted connection. China Civil Engineering Journal, 2015, 48(4):7-14. (in Chinese)
[4] Vigneri V, Odenbreit C, Romero A. Numerical study on design rules for minimum degree of shear connection in propped steel-concrete composite beams. Engineering Structures, 2021, 241(4):112466.
[5] Ding JN, Zhu JS, Kang JF, et al. Experimental study on grouped stud shear connectors in precast steel- UHPC composite bridge. Engineering Structures, 2021, 242(8):112479.
[6] Wang YH, Yu J, Liu JP, et al. Experimental and numerical analysis of steel-block shear connectors in assembled monolithic steel-concrete composite beams. Journal of Bridge Engineering, 2019, 24(5):04019024.
[7] Nie X, Fan JS, Lei FL, et al. Experimental research on improved composite box-girder with corrugated steel webs. Journal of Building Structures, 2014, 35(11):53-61.(in Chinese)
[8] Di J, Fan JH, Zhou XH, et al. Hysteretic behavior of composite bridge columns with plastic hinge enhanced by engineered cementitious composite jacket for seismic resistance. Engineering Structures, 2022, 251:113532.
[9] Nie JG. Steel-concrete composite beam structures: Experiment, theory and application. Beijing, China: Science Press,2005. (in Chinese)
[10] Song SS, Chen J, Xu F. Mechanical behaviour and design of concrete-filled K and KK CHS connections. Journal of Constructional Steel Research, 2022, 188:107000.
[11] Xue W, Chen J, Xu F, et al. Corrosion development of carbon steel grids and Shear connectors in cracked composite beams exposed to wet–dry cycles in chloride environment. Materials, 2018, 11(4):479.
[12] Chen J, Zhang H, Yu QQ. Static and fatigue behavior of steel-concrete composite beams with corroded studs. Journal of Constructional Steel Research, 2019, 156:18-27.
[13] Kwon G, Engelhardt MD, Klingner RE. Behavior of post-installed shear connectors under static and fatigue loading. Journal of Constructional Steel Research, 2010, 66(4):532-541.
[14] Kwon G, Engelhardt MD, Klingner RE. Experimental behavior of bridge beams retrofitted with postinstalled shear connectors. Journal of Bridge Engineering, 2010, 16(4):536-545.
[15] Kwon G, Engelhardt MD, Klingner RE. Parametric studies and preliminary design

recommendations on the use of postinstalled shear connectors for strengthening non-composite steel bridges. Journal of Bridge Engineering, 2011, 17(2):310-317.
[16] Hallmark R, Collin P, Hicks SJ. Post-installed shear connectors: Fatigue pushout tests of coiled spring pins. Journal of Constructional Steel Research, 2018, 153:298-309.
[17] Liu XP, Bradford MA, Ataei A. Flexural performance of innovative sustainable composite steel-concrete beams. Engineering Structures, 2017, 130:282-296.
[18] Liu XP, Bradford MA, Lee MSS. Behavior of high-strength friction-grip bolted shear connectors in sustainable composite beams. Journal of Structural Engineering, 2014, 141(6):04014149.
[19] Ataei A, Bradford MA, Valipour H. Sustainable design of deconstructable steel-concrete composite structures. Procedia Engineering, 2016, 145:1153-1160.
[20] Lam D, Dai XH, Ashour A, et al. Recent research on composite beams with demountable shear connectors. Steel Construction Design and Research, 2017, 10(2):125-134.
[21] Dai XH, Lam D, Saveri E. Effect of concrete strength and stud collar size to shear capacity of demountable shear connectors. Journal of Structural Engineering, 2015, 141(11):04015025.
[22] Rehman N, Lam D, Dai XH, et al. Experimental study on demountable shear connectors in composite slabs with profiled decking. Journal of Constructional Steel Research, 2016, 122:178-189.
[23] Pavlovic M, Markovic Z, Veljkovic M, et al. Bolted shear connectors vs. headed studs behaviour in pushout tests. Journal of Constructional Steel Research, 2013, 88:134-149.
[24] Pathirana SW, Uy B, Mirza O, et al. Flexural Behaviour of composite steel–concrete beams utilising blind bolt shear connectors. Engineering Structures, 2016, 114:181-194.
[25] Ban HY , Uy B, Pathirana SW, et al. Time-dependent behaviour of composite beams with blind bolts under sustained loads. Journal of Constructional Steel Research, 2015, 112:196-207.
[26] Pathirana SW, Uy B, Mirza O, et al. Strengthening of existing composite steel-concrete beams utilising bolted shear connectors and welded studs. Journal of Constructional Steel Research, 2015, 114:417-430.
[27] Yang F, Liu YQ, Jiang ZB, et al. Shear performance of a novel demountable steel-concrete bolted connector under static pushout tests. Engineering Structures, 2018, 160:133-146.
[28] Suwaed ASH, Karavasilis TL. Novel demountable shear connector for accelerated disassembly, repair, or replacement of precast steel-concrete composite bridges. Journal of Bridge Engineering, 2017, 22(9):04017052.
[29] Kozma A, Odenbreita C, Brauna MV, et al. Pushout tests on demountable shear connectors of steel-concrete composite structures. Structures, 2019, 21(SI):45-54.
[30] Ataei A, Zeynalian M, Yazdi Y. Cyclic behaviour of bolted shear connectors in steel-concrete composite beams. Engineering Structures, 2019, 198:109455.

[31] Wang LZ, Webster MD, Hajjar JF. Pushout tests on deconstructable steel-concrete shear connections in sustainable composite beams. Journal of Constructional Steel Research, 2019, 153:618-637.
[32] Dallam LN. High strength bolt shear connectors push out tests. Journal Proceedings. 1968, 65(9):757-769.
[33] Dallam LN, Harpster JL. Composite beam tests with high-strength bolt shear connectors. The National Academies of Sciences Engineering Medicine, 1968.
[34] Hosseini SM, Mamun MS, Mirza O, et al. Behaviour of blind bolt shear connectors subjected to static and fatigue loading. Engineering Structures, 2020, 214:110584.
[35] Tan EL, Varsani H, Liao FY. Experimental study on demountable steel-concrete connectors subjected to combined shear and tension. Engineering Structures, 2019, 183:110-123.
[36] Chen YT, Zhao Y, West JS, et al. Behaviour of steel–precast composite girders with through-bolt shear connectors under static loading. Journal of Constructional Steel Research, 2014, 103:168–178.
[37] Ataei A, Bradford MA, Liu XP. Experimental study of composite beams having a precast geopolymer concrete slab and deconstructable bolted shear connectors. Engineering Structures, 2016, 114:1-13.
[38] Zhang YJ, Liu AR, Chen BC, et al. Experimental and numerical study of shear connection in composite beams of steel and steel-fibre reinforced concrete. Engineering Structures, 2020, 215:110707.
[39] ANSI/AISC 360-16. Specification for structural steel buildings. Chicago, USA: America Institute of Steel Construction (AISC); 2016.
[40] ISO5049-1994. Mobile equipment for continuous handling of bulk materials. Part 1: Rules for the design of steel structures. Geneva, Switzerland: The International Organization for Standardization (ISO); 1994.
[41] CEN (European Committee for Standardization). Eurocode 4: Design of composite steel and concrete structures—Part 1-1: General rules and rules for buildings. London, Britain: EN 1994-1-1; 2004.
[42] Roddenberry MR, Easterling WS, Murray TM. Behavior and strength of welded stud shear connectors. Blacksburg, USA: Virginia Polytechnic Institute and State University; 2002.
[43] Tong LW, Chen LH, Wen M, et al. Static behavior of stud shear connectors in high-strength-steel–UHPC composite beams. Engineering Structures, 2020, 218:110827.
[44] Oehlers DJ, Coughlan CG. The shear stiffness of stud shear connections in composite beams. Journal of Constructional Steel Research, 1986, 6(4):273–284.

[45] Yang T, Liu SY, Qin BX, et al. Experimental study on multi-bolt shear connectors of prefabricatedsteel-concrete composite beams. Journal of Constructional Steel Research, 2021, 173:106260.

[46] Zhang YJ, Chen BC, Liu AR, et al. Experimental study on shear behavior of high strength bolt connection inprefabricated steel-concrete composite beam. Composites Part B-Engineering, 2019, 159:481-489.

[47] Chinese Code GB50017-2017. Code for design of steel structure. Beijing, China: China Architectural Industry Press; 2017. (in Chinese)

[48] Ollgaard JG, Slutter RG, Fisher JW. Shear strength of stud connectors in lightweight and normal-weight concrete. AISC Engineering Journal, 1971, 8(2):55-64.

Notation

The following symbols are used in this paper:

Latin upper case letters

A_e	$=\pi D_e^2/4$, the effective cross-sectional area of bolt;
A_{e_M12}	= the effective cross-sectional area of the M12 series bolt;
A_{e_M16}	= the effective cross-sectional area of the M16 series bolt;
A_{plug}	= the cross-sectional area of the plug without considering the bolt holes;
D	= the nominal diameter of the bolt;
D_1	= the diameter of the top face of the plug;
D_e	= the effective bolt diameter;
D_i	$=D_1+h\tan\theta$, the diameter of the *i*th face of the plug;
E_c	= the elastic modulus of concrete;
E_s	= the elastic modulus of the steel material;
F_{pre}	= the preload in ISO5049-1994;
F_{Exp}	= The ultimate load of each specimen;
F_{u_AISC}	= the design load for welded shear studs stipulated in AISC360-16;
H_c	= the height of the conical iron plug;
$K_{ini\_0.2mm}$	= the stiffness value was calculated at 0.2 mm slip;
$K_{ini\_0.5F_u}$	= the stiffness value was calculated at 50% of the shear strength;
$K_{ini\_0.7F_u}$	= the stiffness value was calculated at 70% of the shear strength;
N_b	= the number of high-strength bolts in test specimens;
R_g	=1, a coefficient specified in the design code AISC 360-16;
R_p	=0.75, a coefficient specified in the design code AISC 360-16;
S_{peak}	= the slip value at the peak load of TIB shear connector;
S_u	= the maximum slip value of TIB shear connector, which was determined at 90% of the characteristic peak load after the load dropped
T_u	= the ultimate tensile strength of each connecting bolt;
V_{AISC}	= the bolt shear fracture strength in the design code of AISC360-16;
V_{con}	= the concrete local failure strength;

$V_{\mathrm{con\_AISC}}$ = the concrete local failure strength proposed by Ollgaard et al;

V_{EC4} = the bolt shear fracture strength in the design code of EC4;

V_{Exp} = the experimental shear capacity of individual TIB connectors ;

V_{GB} = the bolt shear fracture strength in the design code of GB50017-2017;

$V_{\mathrm{M12\_Ave}}$ = the average shear capacity of the M12 series for the newly assembled one in test;

$V_{\mathrm{M16\_Ave}}$ = the average shear capacity of the M16 series for the newly assembled one in test;

V_{pro} = the proposed shear capacity of the TIB shear connectors;

$V_{\mathrm{RM12\_Ave}}$ = the average shear capacity of the M12 series for the replaced one in test;

$V_{\mathrm{RM16\_Ave}}$ = the average shear capacity of the M16 series for the replaced one in test;

V_{TIB} = the shear strength of the re-assembly TIB connector;

Latin lower case letters

f_{c} = the cylinder compressive strength of concrete;

f_{y} = the yield strength of the high strength bolt;

$f_{\mathrm{y\_M12}}$ = the yield strength of the M12 series high strength bolt;

$f_{\mathrm{y\_M16}}$ = the yield strength of the M16 series high strength bolt;

f_{u} = the ultimate strength of the connecting bolt;

h = the distance from integration point to the bottom circular surface of the plug;

Greek case letters

γ = the partial factor in EC4;

γ_{v} = the strength reduction factor for the replaced to newly assembled TIB shear connector;

θ = the conical degree of the plug;