IJCRT.ORG ISSN : 2320-2882

INTERNATIONAL JOURNAL OF CREATIVE RESEARCH THOUGHTS (IJCRT)

An International Open Access, Peer-reviewed, Refereed Journal

RESISTANCE OF BOLTED SHEAR CONNECTORS IN PREFABRICATED STEEL-CONCRETE COMPOSITE DECKS

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Abstract

When a tall building is subjected to earthquake under the action of lateral loads, providing a suitable lateral force resisting system has a significant effect on the performance of the RC frame structure. The present study focuses on the study of system based strengthening systems for RC frame structure. The effectiveness of different locations of shear wall and X bracing on the RC frame structure has been carried out. For this study, a G+9 storied RC frame structure has been considered and structural behavior has been studied for time history at Pin Code: 370020 (Bhuj, Gujarat, India) taken of 2001-01-26 at 03:16:40 UTC of magnitude (M7). The RC frame structure models are analyzing by nonlinear dynamic time history as per IS 1893:2016(part1) using SAP2000 software. A comparative study has been performed on parameters namely, joint displacement, base shear, maximum bending moment, shear force and time period on the considered RC bare Frame and system based strengthening systems, thus, six number of models have analyzed and compared. From the study, it has been observed that the joint displacement of the bare frame with shear wall on corner edge decrease in the structure as compared to the bare frame in seismic zone V. Base shear value increases in the strengthened RC frame in comparison with bare frame. Maximum bending moment, maximum shear force value and time period values in frame model with shear wall and X bracing get reduced as compared to frame model without them. In the present study, analysis of G+9 story RC frame with and without system-based strengthening has been performed by nonlinear time history analysis using SAP2000, conclusions inferred is that the shear walls configured with the shear wall on corner edge with bare frame have been found beneficial as compared with other models.

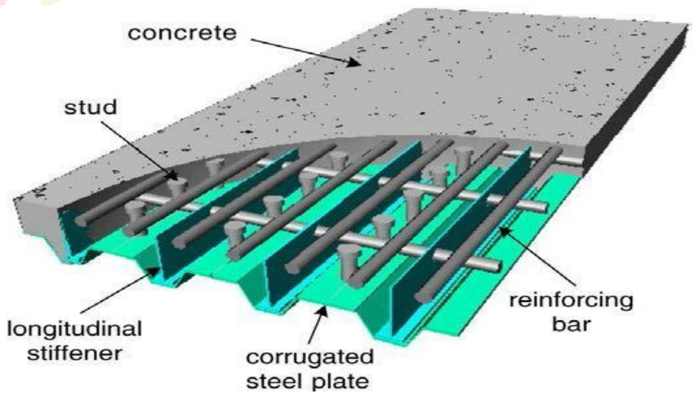
Keywords: Prefabricated steel-concrete composite beams, Shear connectors, High-strength bolts, Shear resistance, Ductility, Push-out tests, Finite element analysis, Parametric study, Plasticity, Damage mechanics.

Chapter 1 INTRODUCTION

1.1. Background

Composite deck slab consists of steel and concrete. In order to ensure suitable connection between two parts of composite plate-steel and concrete, headed studs are used as shear connector. When the concrete has gained sufficient strength, it acts in combination with the tensile strength of the decking to form a 'composite' slab. Slab thickness normally ranges from 100mm to 250mm for shallow decking and from 280mm to 320mm for deep decking. Composite slabs are normally used for spans between 3m and 4.5m supported on beams or walls. If the slab is unpropped during construction, the decking alone resists the self-weight of the wet concrete and construction loads. Subsequent loads are applied to the composite section. If the slab is propped, all of the loads have to be resisted by the composite section. This can lead to a reduction in the imposed load that the slab can support, because the applied horizontal shear at the decking-concrete interface increases. However, for both unpropped and propped conditions, load resistances in excess of loading requirements for most buildings can be achieved. Composite slabs are usually designed as simply supported members in the normal condition. Composite slabs and beams are commonly used (with steel columns) in the commercial, industrial, leisure, health and residential building sectors due to the speed of construction and general structural economy that can be achieved. Although most commonly used on steel framed buildings, composite slabs may also be supported off masonry or concrete components.

Fig- 1.1 Steel–concrete composite deck



1.2. Application of bolted shear connectors

With the use of bolted shear connectors, faster erection methods can be developed, as illustrated in Fig. 1.2. Bolts can be casted in prefabricated concrete slabs and on site assembled to the predrilled top flange of the steel section part of composite member.

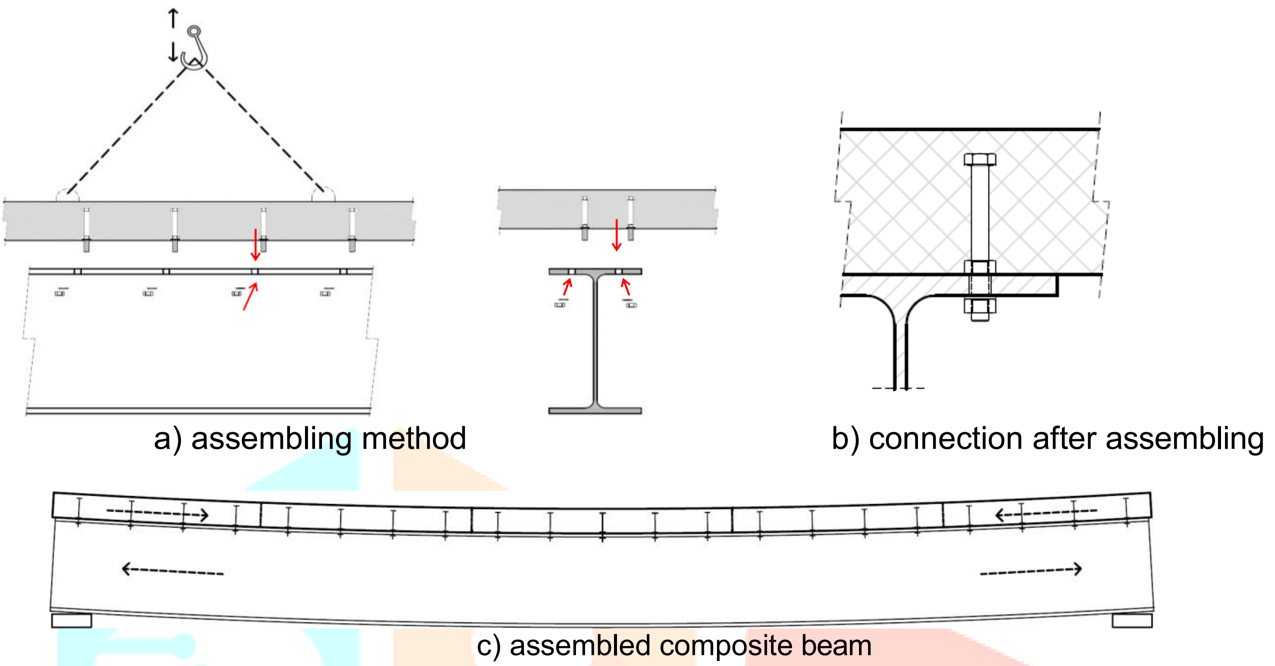


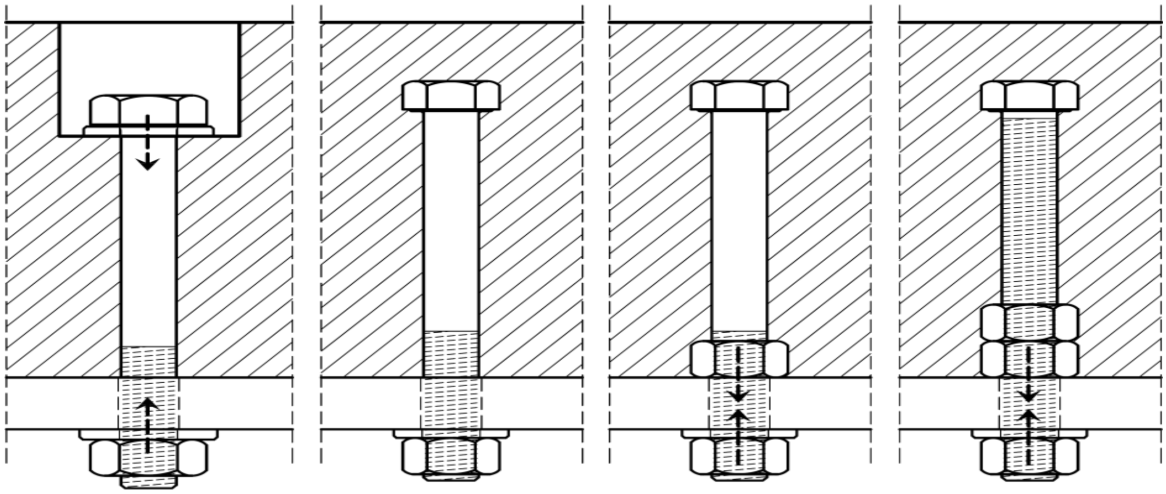
Fig. 1.2 Prefabrication with use of casted bolted shear connectors.

Time required for grout hardening can thus be eliminated, which is advantageous when compared to the solution with grouped headed studs. On the other hand, high fabrication precision of prefabricated elements needs to be achieved so as to enable assembling on site and to ensure assumed composite action of the structure.

The construction costs with the use of bolted shear connectors are expected to be higher when compared to traditional headed studs. For certain applications, however, the precast structures with bolted shear connectors may prove to be an economically competitive option due to faster erection and lower life cycle costs.

Long-term behavior and durability issues may require replacement of concrete slabs or their parts during maintenance of composite bridge decks. It is a complicated and time-consuming procedure in case of the, most commonly used, welded shear connectors. With the use of bolted shear connectors easier dismantling and replacement of concrete slabs can be achieved. It is also important from the sustainability point of view since the structure can be easily removed at the end of its lifetime.

Possible uses of bolted shear connectors are shown in Fig. 1.3. The composite action is established with or without nuts embedded in the slab, either with or without preloading of the bolts.



a) friction grip bolt b) without embedded nut c) single embedded nut d) double embedded nuts

Fig. 1.3 Bolted shear connectors types.

Friction grip bolts shown in Fig. 1.3(a) transfer interface shear forces through friction between the concrete slab and flange of the steel profile. Preloading of the friction grip bolt is made through the thickness of the concrete slab. The slab is subjected to high local compressive stresses, which leads to an unfavorable loss of preloading force due to creep of concrete. Embedded bolted shear connectors shown in Fig. 1.3(b), (c) and (d) transfer interface shear forces by bearing on concrete and on the hole in the steel flange and shear across the threaded part of the bolt. Nearly double shear resistance can be achieved by bearing when compared to a friction transferring mechanism using the bolts of the same grade. Preloaded bolts intended for the slip resistant connection need to be of higher grade (10.9) and fabrication class. They are more expensive (around three times) when compared to regular high strength bolts (grade 8.8) not intended for the slip resistant connection. Therefore, embedded bolted shear connectors are suspected to be more feasible for use in steel-concrete composite decks when compared to friction grip bolts. The only shortcoming of their usage is that they are not slip resistant. The influence of incomplete interaction on the composite member behavior, due to slip in the hole of the embedded bolted shear connector, need to be taken into account. Shear stiffness is another important property of the shear connector. Bolts without embedded nuts, shown in Fig. 1.3(b), have low stiffness and therefore their application as shear connectors is doubtful. Bolted shear connectors with embedded nuts, shown in Fig. 1.3(c) and (d), will be examined in this thesis because they have much higher shear stiffnesses. They are more suitable for casting in prefabricated concrete slabs since they can be mounted by the nuts on both sides to a template in the formwork.

Prefabricated composite deck structures with bolted shear connectors may be used in residential and commercial buildings, car parks and modular building systems. They can also be competitive for short span overpass bridges and modular temporary bridge systems. However, bolted shear connectors are rarely used in composite structures. One of the possible reasons could be the lack of detailed research and design rules concerning their specific behavior. In contrast, welded headed studs, as the most widely used shear connectors, are well covered by design rules in many codes and continuous research on their behavior in composite structures extends for decades in the past.

1.3. Objectives of the research

The aim of the research presented in this thesis is to promote the application of bolted shear connectors. As a first step, feasibility of their application in the longitudinal shear connection of composite decks needs to be examined. Further, detailed examinations of their behavior in push-out tests, as the first step towards the design recommendations are necessary. Basic shear connector properties, such as: shear resistance, stiffness and ductility will be examined through comparison with classical welded headed studs. Furthermore, failure modes of bolted shear connectors will be recognized and compared. Based on the recognized bolt and concrete failure modes, a parametric study of the main material and geometrical properties of such type of shear connection will be performed. Shear resistance calculation model and ductility criterion will be proposed on the basis of the parametric study, as the second step towards the design recommendations. Additionally, certain specific behavior of bolted shear connectors: such as initial slip in hole, will also be examined to give ground for their proper application in composite decks

1.4. Methodology of the research

Analysis of literature will be performed to present current state-of-the-art on bolted shear connectors.

Experimental works will be performed such as: push-out tests with bolted shear connectors, shear tests on bolts and standard tests to obtain properties of materials used in the research (steel and concrete). Advanced strain measuring method - Digital Image Correlation (DIC) will be employed for certain tasks.

Advanced 3D finite element (FE) models of push-out tests, shear tests on bolts and standard material tests will be built and calibrated based on experimental results. Quasi-static analyses with explicit dynamic solver and damage material models will be used which leads to the most realistic prediction of the real behavior of the specimens.

Parametric study will be performed using the previously developed and validated advanced FE models.

Analytical methods will be used, based on FE analyses and experimental results to validate the recognized failure modes of the bolted shear connector and to develop shear resistance calculation model and ductility criterion.

1.5. Scope of the thesis

The content of this thesis is organized in nine chapters.

Chapter 2 summarizes previous research on the use of bolted shear connectors. Literature review on most commonly used shear connectors – headed studs is also given, as well as a short overview on other shear connector types.

Chapter 3 presents study of the feasibility of using bolted shear connectors considering requirements for application, technical aspects, cost effectiveness and environmental impacts. A case study is made comparing bolted shear connectors to grouped welded headed studs in a prefabricated composite deck.

Chapter 4 shows procedures and results of experimental investigations comprising two series of push-out tests on M16 and M24 (grade 8.8) bolted shear connectors with single embedded nut. Procedures and results from material properties tests and shear tests on bolts are also shown.

Chapter 5 deals with finite element analyses of push-out tests. FE models are built to match specimens used in experiments. Calibrations are made with the help of data from material properties and push-out tests. Results of FE analyses are validated, for bolted shear connectors, against the experimental results. Additionally, supplemental FE models for welded headed studs are made to match available experimental push-out test data. The intention was to examine and compare bolted shear connectors and headed studs' key properties and failure modes. Initially accumulated slip for bolted shear connectors, during cyclic loading in push-out tests, is also analyzed, based on supplemental FE models.

Chapter 6 shows models and results of FEA parametric studies of geometrical and material properties of a shear connection with bolted shear connectors. Firstly, initial parametric study is conducted in order to analyze the significance of the influence of certain parameters on resistance and ductility of the bolted shear connection. Parameters considered in the initial parametric study are: bolt preloading force, number of embedded nuts, longitudinal spacing between shear connectors and shear connector height. Later, the most significant parameters influencing behavior of bolted shear connectors; bolt diameter, concrete strength and shear connector height are coupled in main parametric study in order to obtain data for the development of shear resistance and ductility criterions.

Chapter 7 comprises analyses and discussion on experimental and FEA results of the push-out tests and the parametric study. Firstly, bolted shear connectors with single embedded nut are compared to welded headed studs based on experimental and FEA results in order to investigate their key properties: resistance, stiffness and ductility. Additionally, cyclic behavior and initial slip during the cyclic loading are analyzed focusing on the bolt-to-hole clearance and threads penetration. Afterwards, experimental and FEA results for bolted shear connectors is analyzed by means of identification of main failure modes of bolt and concrete and development of analytical modes.

Chapter 8 shows development and validation of shear resistance criterions by means of bolt and concrete failures, based on analyses given in Chapter 7 and results of the parametric study given in Chapter

6. Ductility criterions are also given and validated. Based on the criterions developed here, design rules for shear resistance and ductility are proposed.

Chapter 9 gives conclusions and recommendations for engineering practice and application arising from presented research, as well as the propositions for further research in the field.

Chapter 2 LITERATURE OF REVIEW

2.1. Introduction

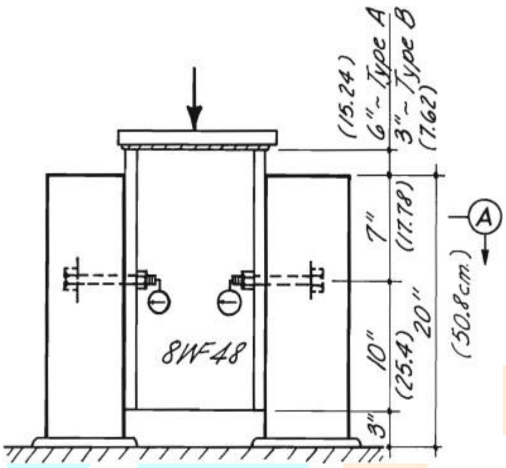
This chapter presents an overview of previous research, which is of significance for examination of resistance of bolted shear connectors. Firstly, research regarding the bolted shear connectors is presented, which is classified according to types given in Fig. Main attention is given to bolted shear connectors with embedded nuts, since they are the main subject of this thesis. Afterwards, short review of research on welded headed studs is given in order to give a basis for comparison in the rest of the thesis. There are no design rules for bolted shear connectors in the design codes. As a starting point for their development in this thesis, design rules for welded headed stud are summarized which have been proposed by several design codes. Other, more or less competitive, shear connector types are given with just short overview at the end of this chapter.

2.2. Bolted shear connectors

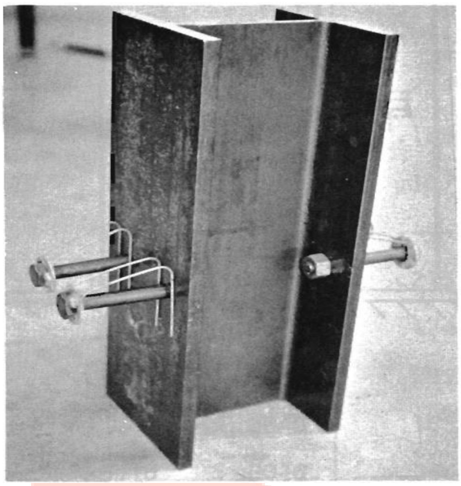
Very limited research on the analyses of behavior of bolted shear connectors is available when compared to the most commonly used welded headed studs. Various types of bolted shear connectors shown in Fig. 1.3 were analyzed in following researches: [Dallam, 1968], [Marshall et al., 1971], [Dedic and Klaiber, 1984], [Hawkins, 1987], [Sedlacek et al., 2003], [Schaap, 2004], [Kwon, 2008], [Lam et al., 2013], [Lee and Bradford, 2013]. Highlights and outcomes of those researches will be presented in following sections, classified in chronological order according to types defined in Fig. 1.3

2.2.1. Friction grip bolts

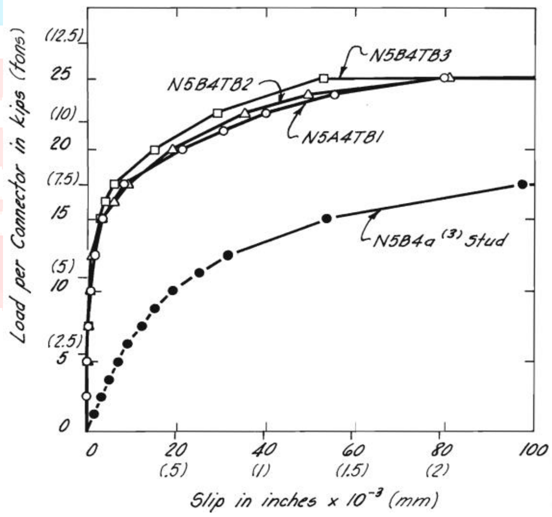
Friction grip bolts shown in Fig. 1.3 (a) transfer interface shear forces through friction between the concrete slab and flange of the steel profile accomplished by preloading of the bolts. They are often used in construction of car parks [ArcelorMittal, 2008]. Since the preloading of the bolt is made through the thickness of the concrete slab, the slab is subjected to high compression stresses. Helical reinforcement is often used around the bolt hole in order to strengthen the concrete subjected to high local stresses. [Dallam, 1968] investigated high strength friction grip bolts in push-out tests, as shown in Fig. 2.1(a). ASTM A325 and A449 bolts were used with measured tensile strengths of 724 MPa and 951 MPa, respectively. Bolt diameters of 12.7, 15.9 and 19.1 mm (1/5, 5/8, 3/4 in.), were varied with height above the flange of 102 mm (4 in.). Bolts were attached to predrilled flanges of a steel profile and held in place by wire springs as shown in Fig. 2.1(b). Four bolts were used for each specimen. Concrete slabs were cast on edge and after 28 days, bolts were preloaded by turn-of-nut method ("snug tight" + 1/2 turn) to achieve minimum specified bolt preloading.



a) test set-up



b) bolt installation

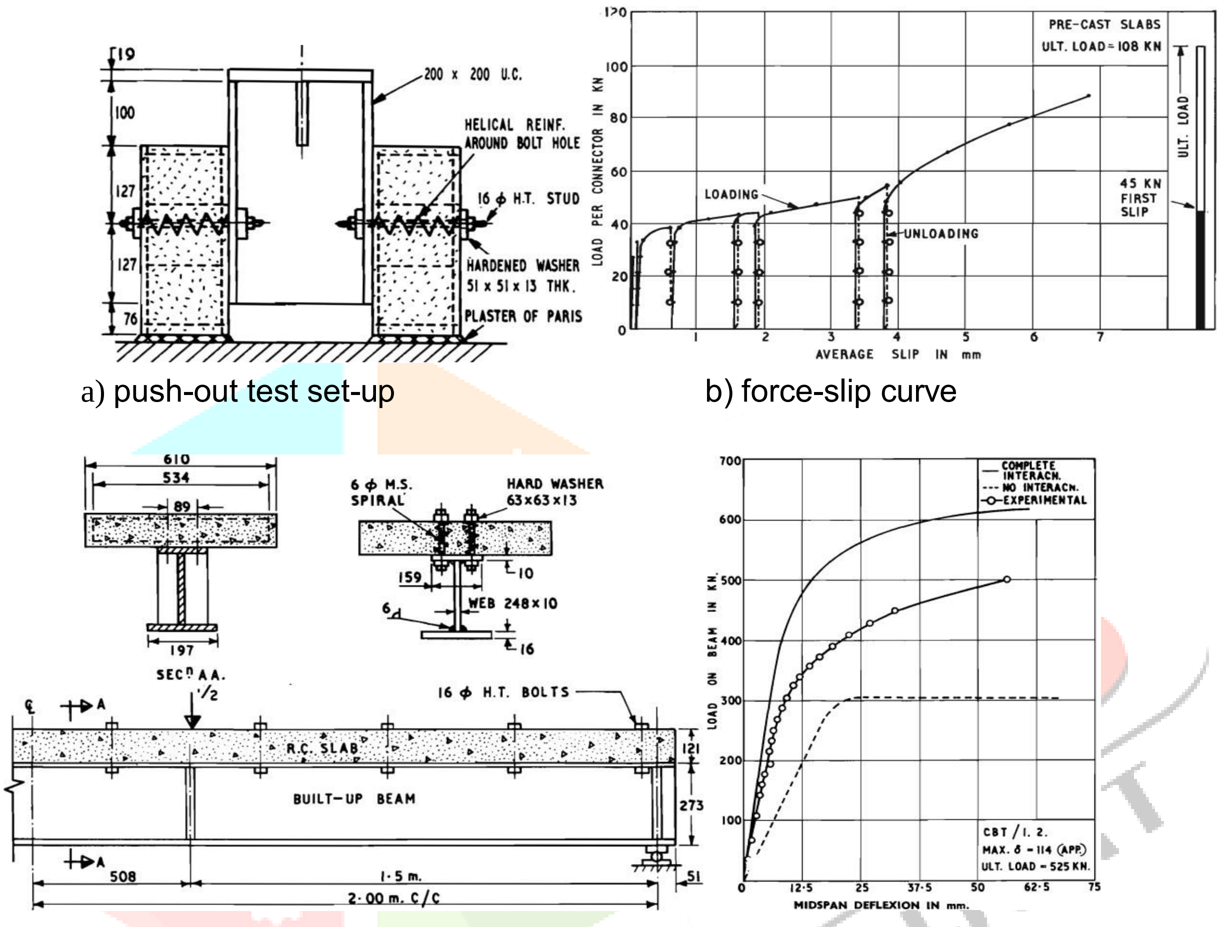


c) force-slip curves for 15.9 mm bolts d) bolts after failure

Fig. 2.1 Friction-grip bolts push-out tests [Dallam, 1968].

Force-slip curves for specimens with $d = 15.9$ mm (5/8 in.) ASTM A449 bolts ($f_u = 951$ MPa) are shown in Fig. 2.1(c), together with results for welded headed studs with same diameter. The tensile strength of stud material was 482 MPa. It was reported that bolts have zero slip at the serviceability stage load level and up to two times the ultimate shear resistance compared to welded headed studs of same dimensions.

[Marshall et al, 1971] conducted static push-out tests with friction-grip bolts of diameter $d = 16$ mm, as shown in Fig. 2.2(a). Variations were made with concrete slabs being either precast or in-situ, as well as the different concrete cube strengths (36 to 50 MPa). In total eleven push-out tests were conducted and only in one case failure of concrete occurred (with cube strength of 36.2 MPa). Bolt preloading forces of approximately 90 kN were achieved. Achieved coefficient of friction was about 0.45 for cases with precast slabs.



a) push-out test set-up b) force-slip curve

c) beam test set-up d) load-deflection curve

Fig. 2.2 Friction-grip bolts push-out and beam tests [Marshall et al, 1971].

Force-slip curve for a specimen with precast concrete slab is shown in Fig. 2.2(b). First slip occurred after the friction force was overcome, while ultimate resistance was more than two times higher.

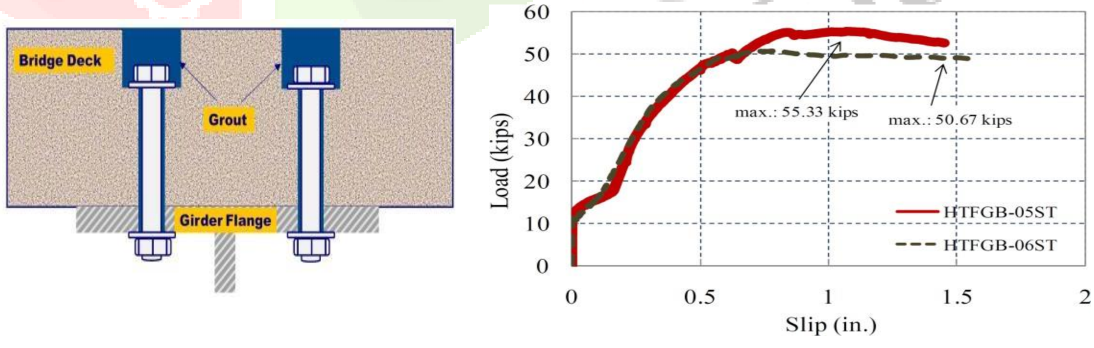
Additionally, five beam tests with 4.00 m and 2.03 m spans were conducted, again the variation of the concrete slab being either precast or cast in-situ. Test set-up and the results are shown in Fig. 2.2(c) and (d). The aim was to examine effect of slip on the degree of interaction and compare it to the incomplete interaction theory by [Newmark et al., 1951]. Conclusions were made that slip coefficient of friction 0.45 can be used for the precast slabs, and if the adequate shear connection is provided (not slipping at the working load range) complete interaction between the steel bean and the concrete slab can be obtained within the working load range.

[BS 5400-5, 1979] gives rules for application of friction-grip bolts in composite beams in its section 10. The design rule is given as: “The longitudinal shear resistance per unit length developed by friction between the concrete flange and steel beam should not be less than the longitudinal shear force per unit length at the serviceability limit state”. The design frictional resistance, developed by each bolt at the interface, is given in Eq. 2.1, where $\mu = 0.45$ is the recommended value for the friction coefficient and $F_{p,C}$ is the bolt preloading force.

$$P_{fric} = \mu \cdot F_{p,C} / 1.2 \qquad 2.1$$

It is noted that account should be taken of the loss of the bolt preloading force due to shrinkage of the concrete and creep of the steel and concrete, but no practical directions are given. It is assumed that ultimate limit state is satisfied with Eq. 2.1 limited by the loads for serviceability load level. Notably lower values of shear resistances can be obtained according to Eq. 2.1 when compared to those obtained by [Dallam, 1968]. However, shear resistances according to Eq. 2.1 are comparable to results obtained by [Marshall et al, 1971] and it seems that their research served as the background for the design rules in [BS 5400-5, 1979].

[Kwon, 2008] examined friction grip bolts, shown in Fig. 2.3(a), as post installed shear connectors for use in strengthening existing non-composite bridges. Single bolt shear tests were conducted under static and fatigue loading. ASTM A325 bolts (830 MPa nominal tensile strength) were used with diameter of 22 mm and 127 mm height above the flange and preloading force of 175 kN. Holes with diameter of 25 mm were drilled in concrete, while gaps between the bolt and the hole were not filled. In total two specimens were tested for static loads and one for fatigue loading with 5 million cycles. Force-slip curves for static single bolt shear tests are shown in Fig. 2.3(b). One of the specimens failed by fracture of the bolt (HTFGB-06ST) while other failed by crushing of the concrete (HTFGB-05ST). Initial slip, after the friction due to preloading of the bolts was overcome is noticed at relatively low load level. Fatigue test with shear stress range of 241 MPa showed good performance, as the shear connector did not fail after 5 million cycles.



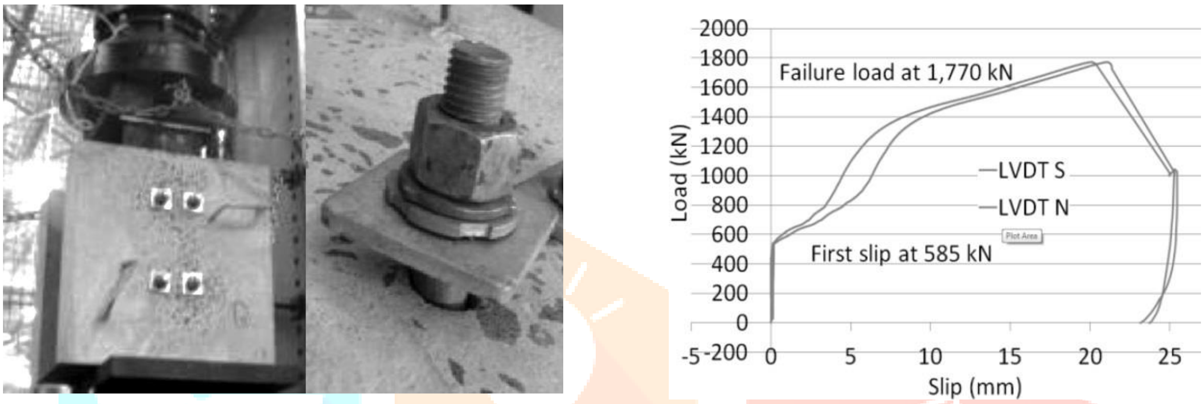
a) connection layout b) force-slip curves

Fig. 2.3 High-tension friction-grip bolt (HTFGB) [Kwon, 2008].

Final conclusion is made that HTFGB showed similar or better shear resistance when compared to conventional headed studs, while fatigue strength is reported to be much better. Additionally, beam tests were made, for different shear connector types, as shown later in Fig. 2.11(a), with results shown in Fig. 2.11(b). Almost 50% increase in load bearing capacity was achieved even with 30% of shear connection ratio when compared to a non-composite beam.

[Lee and Bradford, 2013] conducted two push-out tests according to [EC4, 2004] specifications using bolts M20, grade 8.8. Bolts were preloaded by the force of 145 kN within depth of a concrete slab, through the large steel plates shown in Fig. 2.4(a).

Hole in the concrete slab was 24 mm diameter, 4 mm larger than the bolt diameter. Geopolymer concrete slabs were used, with compressive cylinder strength of 48 MPa. Force-slip curve for one specimen is shown in Fig. 2.4(b). Both specimens failed due to fracture of the bolts. Conclusion is made that after the friction is overcome, large slip occurs, which is caused by oversized holes in the concrete slab. Large ultimate slip indicates ductile behavior of the shear connector.



a) test set-up b) force-slip curves

Fig. 2.4 Friction grip bolts [Lee and Bradford, 2013].

2.2.2. Bolted shear connectors without embedded nuts

[Hawkins, 1987] conducted experimental research on anchor bolts without the embedded nut (Fig. 1.3 (b)) loaded in shear and tension. Variables for the single bolt shear tests were the anchor bolt diameter (19 and 25 mm), embedment depth (76, 127 and 178 mm) and concrete strength (20.7 and 34.5 MPa). It was shown that such anchors have 80% shear resistance when compared to welded headed studs and only 15% of their shear stiffness (see Fig. 2.5).

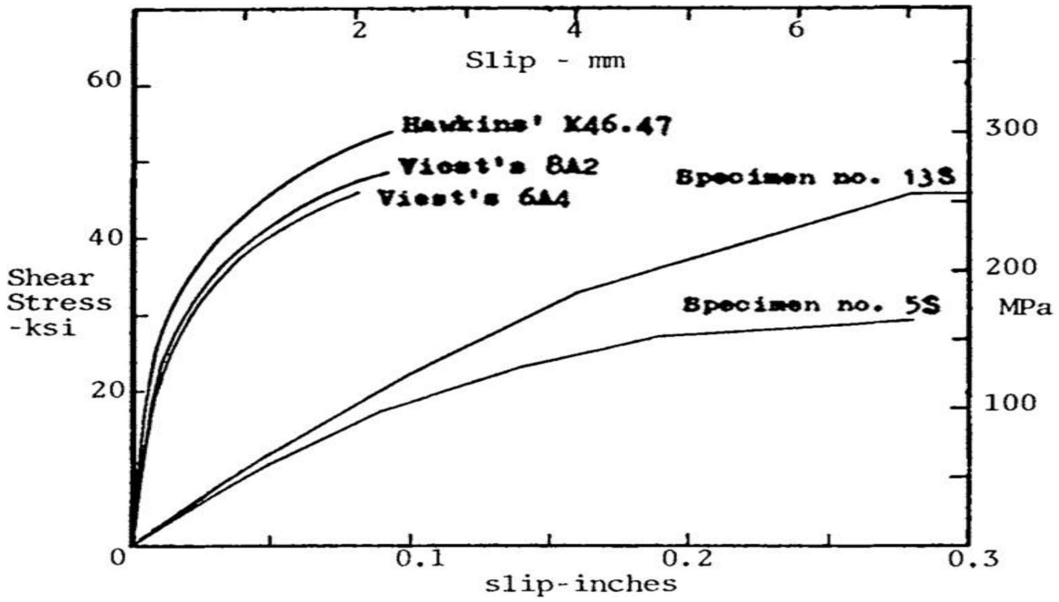


Fig. 2.5 Force-slip curves for studs and bolts without embedded nut [Hawkins, 1987].

[Lam et al., 2013] investigated demountable shear connectors, shown in Fig. 2.6(a) to assess its potential and suitability in terms of replacing the welded headed studs. Eight push-out tests with four connectors were conducted using studs with diameter of 19 mm and various concrete strengths. Two failure mechanisms were observed: fracture of shear connectors near the threaded end and concrete crushing. It was pointed out that slabs were easily removed after the tests, thus proving the ability of the structure to be dismantled. Reference tests with welded headed studs were also made and comparison of the results is presented in Fig. 2.6(b). It was concluded that those shear connectors have similar shear resistance as welded headed stud with better performance in terms of ductility, but with much lower stiffness.

a) shear connectors layout b) force-slip curves

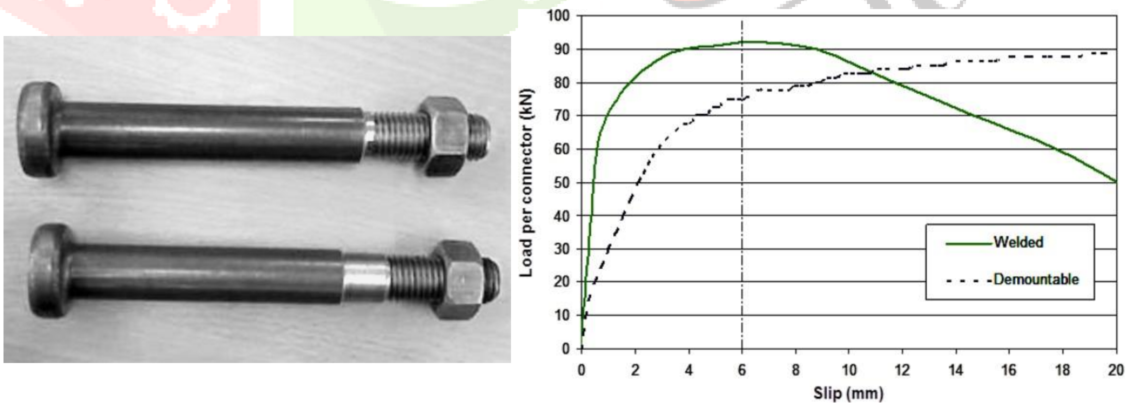
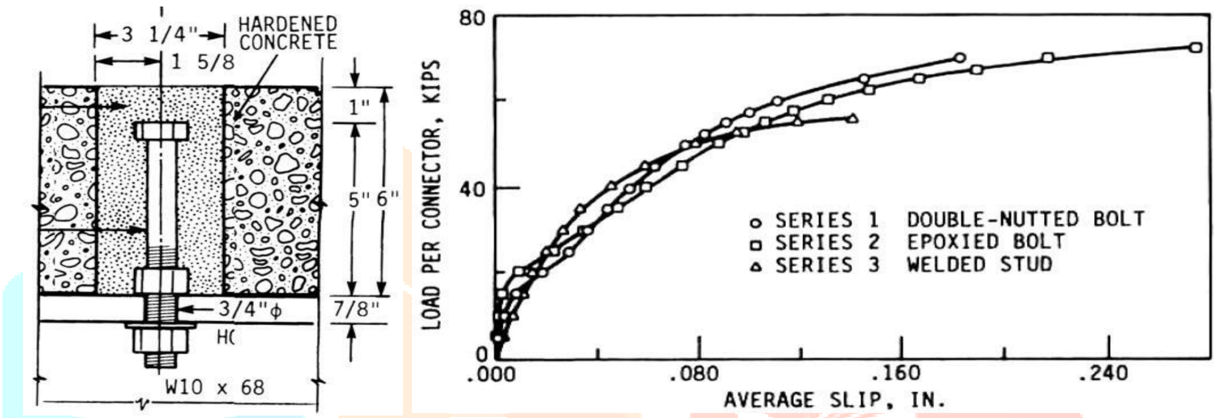


Fig. 2.6 Demountable shear connectors [Lam et al., 2013].

2.2.3. Bolted shear connectors with embedded nuts

Bolted shear connectors with one or two embedded nuts have similar behavior. They were investigated mostly in terms of rehabilitation work so as to strengthen the existing non-composite steel-concrete bridges. Since the resistance of bolted shear connectors with single embedded nut is the subject of this thesis, previous research for this type of bolted shear connectors will be presented with more detail. Results presented here will be summarized later in Table 8.10 to Table 8.12 (section 8.4) and used for validation of proposed shear resistance and ductility criterions.

[Dedic and Klaiber, 1984] performed four push-out tests with four ASTM A325high strength bolts 19 mm in diameter. Nominal tensile strength of such bolt material is 830 MPa (120 ksi). Shear connector layout is shown in Fig. 2.7(a). Concrete compressive strengths, determined by tests, were 35.4 MPa and 31.4 MPa (5140 psi and 4550 psi) for concrete slab and the grout around the shear connector, respectively. Comparable tests for welded headed studs were also conducted. They showed that shear resistance and load-slip behavior of bolted shear connectors with single embedded nut shown in Fig. 1.3 (b), are similar to those of welded headed studs of same dimensions. Average ultimate shear force of 152.1 kN was achieved for bolted shear connectors. Bolt failure was reported, but unfortunately the end of force slip curve is not shown.



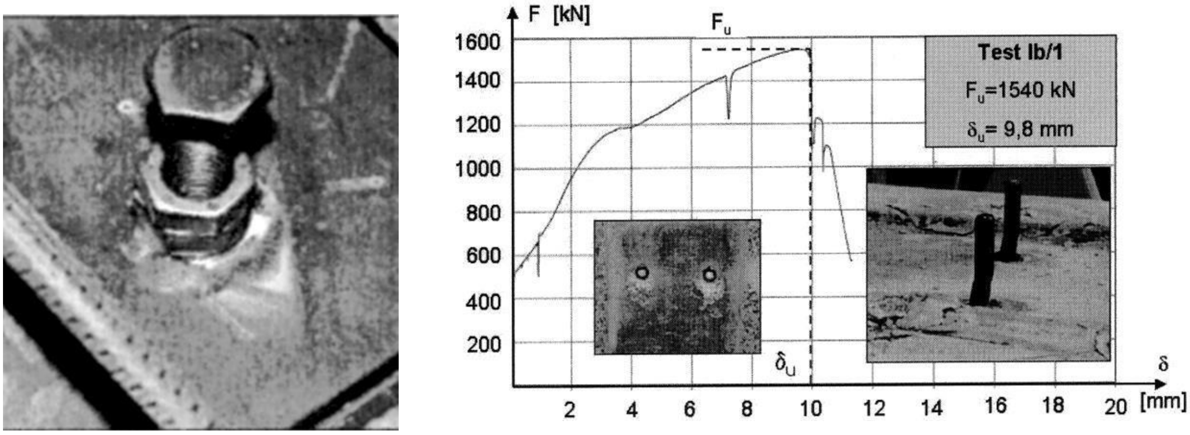
a) shear connector layout b) force-slip curves

Fig. 2.7 Shear connector with embedded nut [Dedic and Klaiber, 1984].

[Sedlacek et al., 2003] conducted research founded by the European Commission under project named: “Composite bridge design for small and medium spans”.

As part of this research, several solutions for full and partial prefabrication of the concrete slab were investigated, using headed studs and bolts, which was carried out at University of Wuppertal by Prof. Dr.-Ing. Gerhard Hansville. Among those, bolted shear connectors were investigated in order to examine the possibility to replace the concrete deck during design life time of temporary bridges.

High strength bolts M20, grade 10.9, were experimentally tested for static and fatigue loads using standard [EC4, 2004] push-out test. Totally three specimens were tested: two for static loads and one for fatigue. Double embedded nuts (see Fig. 1.3 (d)) were used as shown in Fig. 2.8(a). Tensile strength of bolt material 1160 MPa and concrete compressive strength of 46,9 MPa were reported. Bolt shear failures were present in all tests, as shown in Fig. 2.8(b), together with force-slip curve for one specimen. Average ultimate shear force per shear connector was 189 kN with average slip to failure of 10.3 mm. Fatigue test was conducted with force range $\Delta P = 510$ kN and $P_{max} = 1050$ kN, in 3 million cycles. No fatigue failure occurred and no significant increase of slip was observed. Afterwards, this specimen was statically loaded until failure, and same resistance was obtained as for the specimens with only static loads applied. Results were evaluated by some simple hand calculation model, based on bearing capacity of headed studs in concrete and shear failure of the bolts. Mismatch of predicted shear resistance to the test results was too high. Further tests were recommended for development of the design rules.



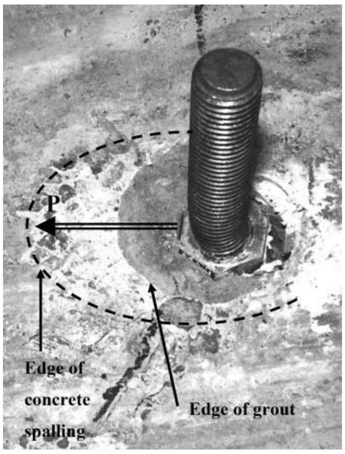
a) shear connectors b) force-slip curve

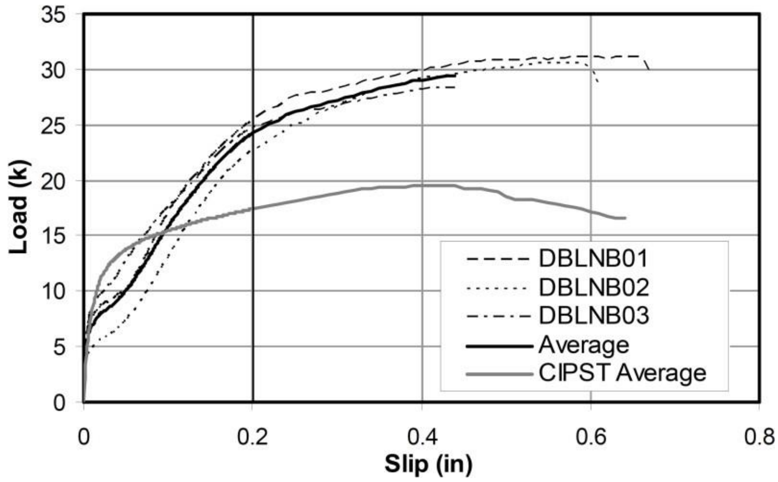
Fig. 2.8 Bolted shear connectors M20 [Sedlacek et al., 2003].

[Schaap, 2004] conducted three single bolt shear tests on bolted shear connectors with double embedded nuts (see Fig. 1.3 (d)), among large number of various post- installed shear connectors analyzed for use in strengthening existing non-composite bridges. Bolts were 19 mm diameter (3/4 in.), while height above the flange was 150mm. ASTM A490 bolt material was used with nominal tensile strength of 1034 MPa (150 ksi). Shear connectors layout is shown in Fig. 2.9(a).

Bolts were post-installed by drilling the 50 mm diameter hole in the concrete slab and filled with a grout afterwards (see Fig. 2.9(b)). Concrete strengths of 23.7 MPa and 21.9 MPa were achieved for the slab and the grout, respectively. Results are presented in Fig. 2.9(c). Average shear resistance of 133.6 kN was achieved, while shear failure of the bolts did not occur. Average maximum slip that was reported is 14.6 mm. Initial slip in hole due to the overcoming of friction is noticed at relatively low load level. Unfortunately, bolt preloading force was not reported. Based on comparison of the results to other post-installed shear connector types in the research, conclusion was made that those shear connectors perform well and their further examination was recommended.

shear connector layout



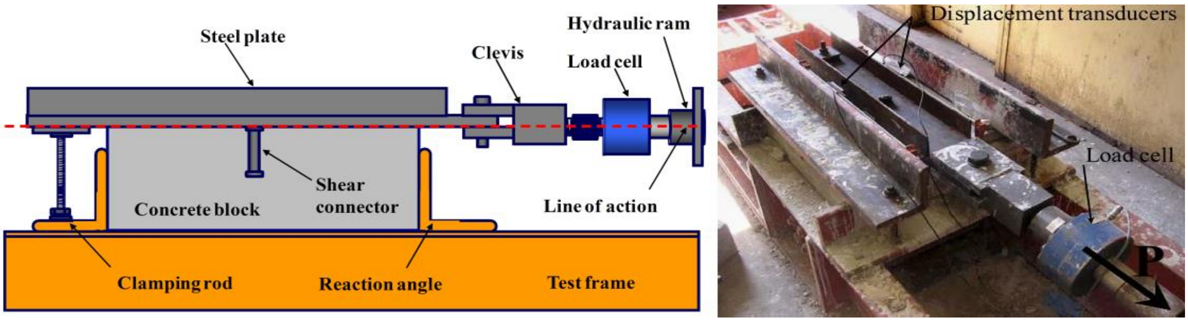


a) bolt after failure c) force-slip curves

Fig. 2.9 Double-nut bolt (DBLNB) shear connector [Schaap, 2004].

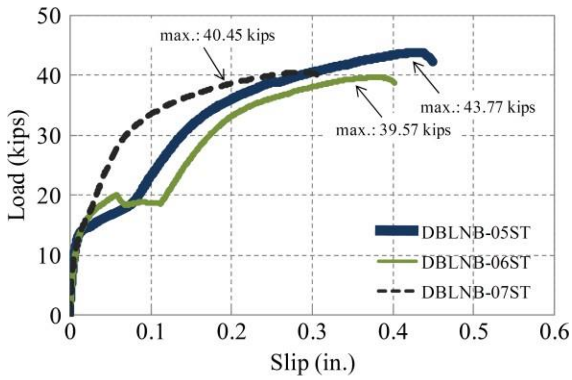
[Kwon, 2008] continued the research conducted by [Schaap, 2004]. He examined bolted shear connector with double embedded nuts, shown in Fig. 1.3 (d) with diameter d = 22 mm and height above the flange h_{sc} = 127 mm, as post installed shear connector for use in strengthening existing non-composite bridges. Single bolt shear tests were conducted for static and fatigue loading, with test set-up shown in Fig. 2.3(a).

DBLNB shear connector was composed of threaded rod and nuts with layout shown in Fig. 2.3(b). ASTM A193 B7 threaded rod material was used with tested tensile strength of 1013 MPa (147 ksi). Holes with diameter of 57 mm were drilled in the concrete slab and filled with high-strength grout after installation of the connectors. Compressive strength of the concrete slab material was 20.3 MPa, while 25.3 MPa was reported for the high strength grout around the connectors. Bolts were preloaded with a force of 173 kN through the thickness of the steel flange. Totally three specimens were tested for static loads and one for fatigue loading with 5 million cycles. Force-slip curves for static single bolt shear tests are shown in Fig. 2.10(c). All of the specimens failed by fracture of the bolt. Average shear resistance of 183.5 kN, per shear connector was obtained. Initial slip, after the friction due to preloading of the bolts was overcome is noticed at relatively low load level. Average slip to failure of 8.7 mm was achieved. One fatigue test with shear stress range of 310 MPa showed good performance, as the shear connector did not fail after 5 million cycles. Final conclusion is made that DNLNB showed similar or better shear resistance when compared to conventional headed studs, while fatigue strength is reported to be much better since the connection is welding free.



a) single bolt shear test set-up

b) connection layout



c) force-slip curves

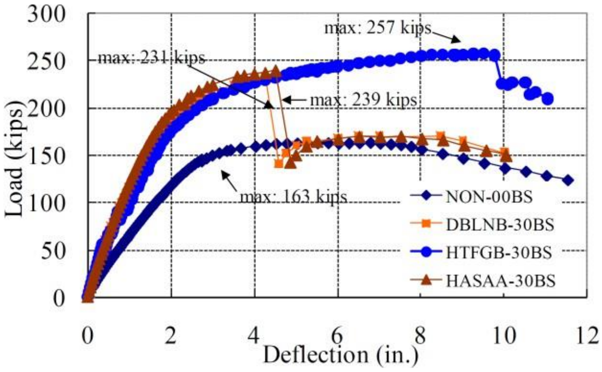
Fig. 2.10 Double-nut bolt (DBLNB) shear connectors [Kwon, 2008].

Additionally, beam tests were made, as shown in Fig. 2.11(a). Beam test set-up consisted of simply supported beam of 11.6 m (38 ft.) span, with W30x99 steel beam and concrete slab 2.13 m wide and 180 mm thick. Partial shear connection with 30% of shear connection ratio was achieved using 16 connectors in a shear span (32 in total).

Reference beam test for a non-composite beam, as well as for other shear connector types were conducted (totally four tests). Results are shown in Fig. 2.11(b). Almost 50% increase in load bearing capacity was achieved even with 30% of shear connection ratio when compared to the non-composite beam. Sudden drop of load was noticed at deflection of approximately 100 mm which is attributed to shear failure of the bolted shear connectors. After this point, the beam behaved as the non-composite beam. Even though the initial slip due to bolt-to-hole clearance was noticed in the single bolt shear tests (see

Fig. 2.10(c)), no significant loss of initial stiffness was noticed in the beam tests (see Fig. 2.11(b)).

a) beam tests



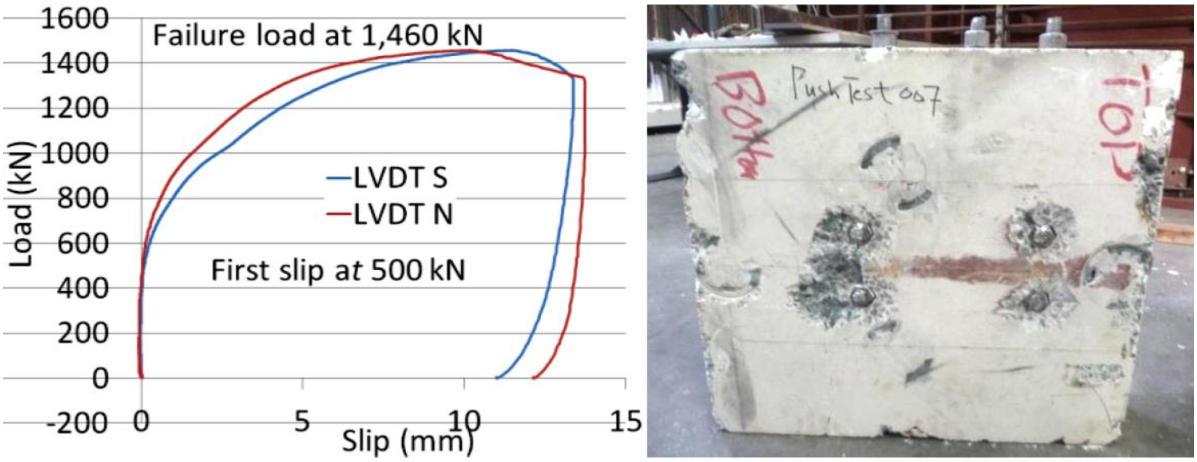
b) load-deflection curves

Fig. 2.11 Beam tests [Kwon, 2008].

Additional FEA of composite beam using nonlinear spring elements for bolted shear connectors was made to investigate this phenomenon. Similar conclusion is made that oversized holes does not significantly influence the behavior (stiffness, strength and ductility) of the composite beam with bolted shear connectors. However, recommendations for limiting the bolt-to-hole clearances were not given.

[Lee and Bradford, 2013] conducted two push-out tests according to [EC4, 2004] specifications using M20, grade 8.8, bolted shear connectors with single embedded nut (Fig. 1.2(c)) and 135 mm height above the flange. Bolts were preloaded by a force of 130 kN within thickness of the steel flange. Geopolymer concrete slabs were cast in place and compressive cylinder strength of 48 MPa was reported.

Force-slip curve for one specimen is shown in Fig. 2.12(a). Both specimens failed due to fracture of the bolts. Characteristic failure is shown in Fig. 2.12(b), where shearing of the bolts, and crushing of concrete in front of shear connectors can be noticed. Average ultimate shear resistance of 177.5 kN, per shear connector, was obtained, with average slip to failure of 11 mm. The tensile strength of the bolt’s material obtained from the test was 946 MPa. Reported ultimate shear resistance was higher than the shear resistance of the bolts at the threaded part when calculated with the tested tensile strength. Authors provided the information that failure of the bolts occurred at the shank, not the threaded part of the bolt, with use of specially designed clamps.



a) force-slip curve b) concrete slab and bolts after failure

Fig. 2.12 M20 bolted shear connectors [Lee and Bradford, 2013].

2.3. Welded headed studs

A very good state-of-the-art on the existing experimental results for welded headed studs from the research of the past few decades (391 push-out tests), and comparisons to design codes are given by [Pallar's and Hajjar, 2010]. As the world- wide database of experimental results for welded headed studs is large, present research is often being conducted using FEA. [Lam and El-Nobody, 2005] conducted parametric FEA by varying headed stud height and concrete strength and compared the results for headed studs shear resistance to predictions in design codes. [Nguyen and Kim, 2009] analyzed shear resistance and ductility of large headed studs with diameter up to 30 mm in their parametric FEA. Prefabrication of composite structures became interesting subject in the past decade. Grouped behavior of welded headed studs, for their application with prefabricated slabs with openings (pockets), have been studied recently by [Okada et al., 2006], [Shim et al., 2008], [Xu et al., 2012] and [Spermic, 2013].

Welded headed studs are the most used shear connectors in steel-concrete composite decks. One of the reasons is that design rules for those shear connectors are well covered in design codes. Short overview of those design rules will be given here since the similar ones will be developed for bolted shear connectors in this thesis.

[EC4, 2004], known as the Eurocode 4, defines shear resistance of welded headed studs as minimum of two values given in Eq. 2.2 and Eq. 2.3. It is not explicitly specified, but it is obvious that those two presents the criterions for failure of the stud and concrete, respectively.

$$P_{\mathrm{Rd}} = 0.8 \cdot f_{\mathrm{u}} \frac{\pi \cdot d^2}{4} \frac{1}{\gamma_{\mathrm{V}}} \quad 2.2$$

$$P_{\mathrm{Rd}} = 0.29 \cdot \alpha \cdot d^2 \sqrt{f_{\mathrm{ck}} E_{\mathrm{cm}}} \frac{1}{\gamma_{\mathrm{V}}} \quad 2.3$$

with:

$$\alpha = 0.2\left(h_{\mathrm{sc}} / d + 1\right) \leq 1.0, \text{ for } h_{\mathrm{sc}} / d \geq 3 \quad 2.4$$

In previous expressions:

b is the stud shank diameter in mm;

h_{sc} is the shear connector height above flange in mm;

f_u is the stud ultimate tensile strength in N/mm^2;

f_{ck} is the concrete characteristic cylinder compressive strength in N/mm^2;

E_{cm} is the secant modulus of elasticity of concrete in N/mm^2;

γ_V is the partial safety factor for shear connector resistance ($\gamma_V = 1.25$).

[JSCE, 2005], the Japanese Standard Specifications for Steel and Composite Structures, also defines shear resistance of the welded headed studs as minimum value for two separate failure modes (stud and concrete). Those are given in Eq. 2.5 and Eq. 2.6. Height does diameter ratio is limited to $h_{ss} / d_{ss} \geq 4$

$$V_{sud} = (31 A_{ss} \sqrt{(h_{ss} / d_{ss}) f'_{cd}} + 10000) / \gamma_b \quad 2.5$$

$$V_{sud} = A_{ss} f_{sud} / \gamma_b \quad 2.6$$

In previous expressions:

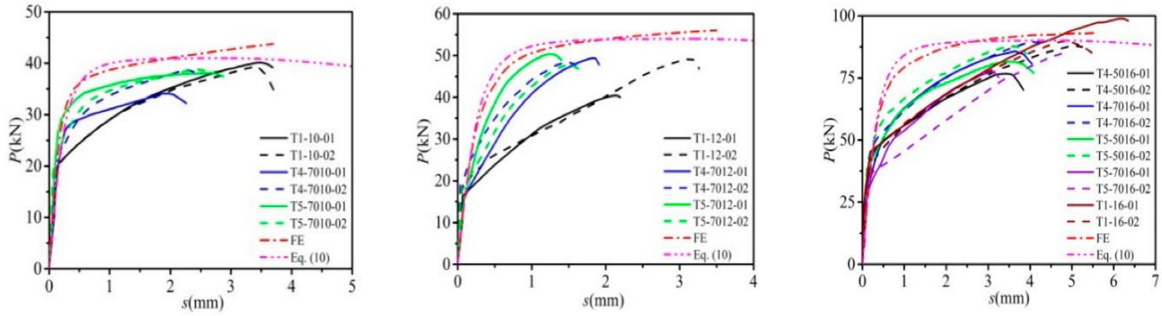
A_{ss} is the stud shank cross sectional area in mm^2;

d_{ss} is the shear connector diameter in mm;

RESULTS AND DISCUSSION: -

1. Load-Displacement Relationship

The measured load-slip relation curves of the tested specimens is presented in Shown Figure. The longitudinal slip is presented as the averaged value for four LVDTs divided by the initial accumulated slip during the preloading process. The load-displacement curves generally included three parts, namely the linear elastic stage, plastic hardening stage, and the failure stage. At the initial period of loading, high shear resisting stiffness was displayed due to the mechanical friction between the slab and steel flange which resulted from the high pretension from the high-strength bolts



(a) (b) (c)

2. Experimental Results Analysis

Summary of the shear stiffness, bearing capacity, and the maximum slip amount for each connector IS refer to the ultimate load and the mean value.

3. Influence of Bolt Diameter

The effect of bolt diameter on the shear stiffness, shear capacity, and the maximum slip given the same concrete slab. With the increase in bolt diameter from 10 to 16 mm, shear stiffness displayed an upward trend in the majority of specimens.

4. Influence of the Reserved Hole Constraint Condition

The bolt sizes of specimens were set as the same, the majority of T5 specimens primarily presented the maximum shear stiffness, except for individual specimens. The main failure mode as shown in specimens with bolt fractures, was that shear capacity was influenced little by the reserved hole constraint condition.

5. Influence of Hole Diameter Ratio

A comparison was made between specimens with the same diameter for detection of the shear stiffness, shear capacity, and the maximum slip under the same reserved hole constraint condition, however, at different hole diameter ratios (D/d). the shear stiffness increased. To be more detailed, compared to the T4 specimen with D/d = 3.1, the average shear stiffness presented an increase by 6.5% in the T4 specimen with D/d = 4.4. In addition, with the increase of hole diameter ratio from 3.1 to 4.4, an increase of the average shear stiffness by 11.4% was observed in T5 specimens.

Conclusions

In this paper, a new high strength bolt shear connector was proposed with the corrugated pipe hole serving as the grout hole in the prefabricated slab. Experimental studies and FE simulations were conducted to investigate the shear resisting mechanism and strengths of the proposed connector. The objective of this work is to better understand the influences of main characteristics such as bolt diameter, yield strength, length-to-diameter ratio, and pretension as well as the concrete strength on the ultimate shear capacity, therefore, extensive parametric studies were also performed. The tests and numerical analysis made in this work led to the following conclusions.

1. The primary failure mode of the push-out tests was bolt failure. Additionally, the process of failure and crack developing was affected by the bolt diameter and the reserved hole constraint condition. The inhibition of crack developments in a concrete slab can be achieved by using precast concrete slab with corrugated pipes.

2. Experimental results demonstrated that bolt connectors with corrugated pipe had higher shear stiffness than normal reserved hole types and cast-in-place slab specimens. The ultimate shear capacity of the bolt connector was mainly influenced by the bolt diameter.

3. The developed numerical model produced satisfactory predictions of the behavior and ultimate shear capacity for the push-out tests on the bolted shear connector. The high strength bolt pretension presented little correlation to the shear capacity on the basis of FE simulations.

4. Based on the experimental data and numerical results, a new formula was proposed for calculation of the shear capacity of bolted shear connectors. The accuracy was discussed through the comparison to design code in GB 50017-2017, AISC, EC4, and the experimental and numerical data in this study and extracted from relative references. The proposed calculation method exhibited a better prediction to the experimental results than other formulas. By modifying the parameters, the load-slip curve of welded stud shear connectors proposed by Ding, can describe the load-slip relationship of bolted shear connectors, especially for T5 specimens, reasonably well.

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