DEVELOPMENT OF CYLINDRICAL SHEAR CONNECTOR INSERTED IN PERFORATED STEEL PLATE AND APPLY TO THE HYBRID STRUCTURE

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ABSTRACT

With the recent development of various hybrid structures such as PC box girder bridges with corrugated steel webs, there is a growing need to develop various shear connectors for connecting steel plates with concrete. We have developed a method which enables a perforated steel plate, in which mortar cylinders ("MCY") or mortar-filled steel tubes ("MFST") are inserted, to serve as a shear connector. This shear connector is characterized by excellent workability, eliminating the need for (i) re-bars inserted in perforated plates (which are generally used for strengthened in perfo-bond strips) and (ii) confirmation that steel plate holes are filled with concrete. In this study, we clarified shear strength and slip characteristics of this new type shear connector and reviewed its application to joints in composite steel girders and PC girders by means of the shear connecting method. MFST were applied as connecting key to joint of precast concrete webs and slabs in box girder PC bridge, and Re-bar connector having same concept of MFST were applied to anchoring system of PC cables in hybrid tower of extra-dosed bridges. These application examples are also presented in this paper.

KEYWORDS: joint of mixed girder bridge, shear connector, mortar filled steel tube

1. INTRODUCTION

Generally, headed studs and perfo-bond strips are used as shear connector between steel and concrete in hybrid structures. It is widely known that the shear strength and ductility of perfo-bond strips can be greatly improved by installing re-bars into perforated steel plate. Therefore, diversion of existing re-bars in slab to penetrating re-bar are general in PC box girder bridges with corrugated steel webs and composite steel girder bridges. However, there are some structures where existing re-bars cannot be used as penetrating re-bars. The examples include perfo-bond strips installed in steel girder webs at the joint structure of composite steel girders and PC girders based on the shear connecting method (Shinozaki et al., 2014), and the composite rigid frame bridge where the end steel girder was embedded in RC abutment (Ashizuka et al., 2007). In these structures, some device must be applied to install new re-bars inserted in perforated plates. In addition, their structures make it difficult to confirm that the steel plate holes are filled with concrete. As a result, there is a growing demand for a new shear connector that can overcome these problems.

It is known that the main failure mode of perfo-bond strips is compressive breaking of concrete in holes of steel plates and shear failure along the surface of perforated plate. Therefore, the significant shear strength can be easily obtained by, for example, installing high-strength concrete or MCY or steel tube into the holes of perforated plate. The study performed by Tanaka and Sakai (2010), is known for inserting high-strength mortar into perforated steel plates for shear connection. Also, Yamaguchi, et al., (2011) carried out the push-out test in which the slit steel tube was inserted into perforated steel plates. For overseas, there performed a study of substitution coiled spring pins inserted into perforated steel plates with headed studs (Roger et al., 1997).

In this study, first of all, a direct shearing test using steel fixtures was carried out with the intention of confirming the shear strength of cylindrical members themselves and identifying the

Fig.1-Mortar Filled Steel Tube (MFST) Inserted in Perforated Steel Plate

effects of material strength and specifications of steel tubes on the shear strength. Next, a push- out test was performed to re-evaluate the shear strength and slip characteristics and also a simplified shear strength evaluation formula was proposed. Then, the secant stiffness and the residual slip displacement of this new shear connector were compared with headed studs and perfo-bond strip.

Recently, an easy maintenance and management of bridges structures are highly required, a mixed structure of light composite steel girders and concrete girders is occasionally adopted. The reason behind this adoption is the need to take the span planning and construction method into consideration. At the joint of the mixed structure, the bearing plate method having multi steel shells is generally used. In this method, as concrete-filled multi steel shells and the PC tendons are used for connecting each other, huge amount of steel material is used and careful attention must be paid when casting concrete.

A relatively simple joining method called the shear connecting method has been proposed. There are already some examples of this construction method. In all of these examples, this method was applied to the joint of composite steel girders and PC or RC hollow slabs. If the shear connecting method is applied, the need for concrete-filled multi steel shells and PC tendons is eliminated, and as a result, the amount of steel material can be reduced while improving workability.

The authors studied the applicability of this shear connecting method to the joints of composite steel girders and PC girders with 2 girders each. Fig. 2 is the concept drawing of this joint structure that steel girders of composite steel girders are inserted into PC girders and connected them by shear connectors. Headed stud connectors were arranged to steel girder flanges of this joint structure. Perfo-bond strip are used as shear connector of steel girder webs because it can reduce the number of welded points and prevent an overcrowded arrangement of studs. However, in the case of this joint structure, additional re-bars are needed because existing re-bars are not around the web. And then, as the perfo-bond strips are located deep inside the frame, it is difficult to confirm that concrete has been filled in the steel plate hole. Therefore, possibility of application of MFST that we have developed was examined

In this study, authors have proposed a simple method to design a shear connector for joints, and performed a static loading test using a model specimen with the size of approximately 1/2 of the real joint structure which the shear connector was designed by proposed technique. Two types of test specimens were used: one with studs, and the other with MFST as shear connector in steel webs. Test results showed that the joints had sufficient bearing force. Then, a 3 dimensional non-linear finite

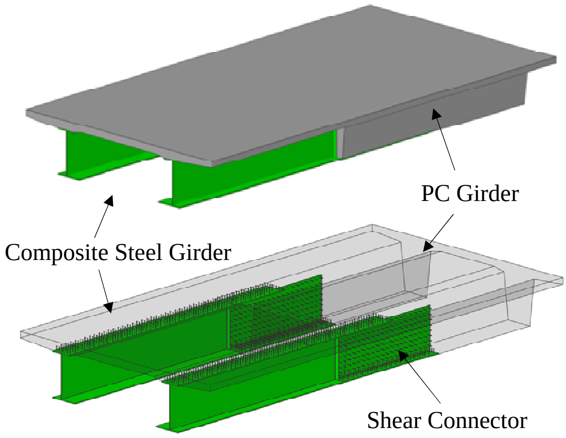


Fig.2-Conceptional Drawing of Joint of Steel Girder and PC Girder using Shear Connector

element analysis was performed in which the characteristics of the shear connector were modeled by a spring element with non-linear property. It was demonstrated by the analysis that the behavior of the joints can be reproduced in detail. The validity of the proposed design technique was also examined by the distribution of shear force generated in the shear connector. It was also demonstrated that the joint structure in which MFST is arranged on webs is workable based on the bearing force and behavior of the joints.

Ideally, loading tests should be performed using the combination of bending moment and shear force assuming the actual situation of structure. However, in the actual design of the shear connector, the number of connectors is determined separately for bending and for shearing. Also, a method to design of shear connector for webs has been established to a certain extent with regard to the similar structure. In the case of this joint structure, in particular, the behavior against bending is expected to be complicated. Therefore, in the loading test, only bending moment acted on the joint, and analyzed shear force transmitted to the shear connector in detail.

2. DEVELOPMENT OF CYLINDRICAL SHEAR CONNECTOR INSERTED IN PERFORATED STEEL PLATE

3.1 Direct Shearing Test

3.1.1 Test Device and Specimen

The direct shearing test is aimed at identifying the effects of the compressive strength of mortar and the specification of steel tubes on the shear strength by obtaining the shear strength of the connector itself. Then, the evaluation approach of the shear strength was defined by the push-out test discussed below.

Fig. 3 shows the direct sharing test device. The test device consists of a steel base, divided into upper and lower sections, and a slit plate located in the middle. The test specimen was fixed on the steel base by the holes opened on the slit plate, and the shearing force was applied by pressing down the slit plate. The outer diameter of the mortar cylinder (MCY) and the mortar filled steel tube (MFST) was 51.6mm and 48.6mm, respectively. The compressive strength range of mortar was 33.6-192N/mm^2, whereas the thickness of the steel tubes was 2.3mm and 3.5mm.

3.1.2 Test Results

All the test specimens displayed shear failure along the surface of the slit plate. Fig. 4 shows the relation of shear strength of MCY and MFST to the compressive strength of mortar (in the case of the MFST, the compressive strength of mortar filled into the tubes). In both cases of MCY and MFST, the shear strength tends to increase in proportion to the compressive strength. MFST have larger shear strength than MCY and their shear strength becomes larger corresponding to the thickness of steel tubes.

Based on the above results, it was demonstrated that the difference in the shear strength of MCY and MFST can be approximately evaluated by the theoretical shear strength (for 2 sides) shown in the formula (1) below, using the tensile strength of steel tubes.

$$V_{spu} = 2 \times \frac{A_{sp} \cdot f_{spt}}{\sqrt{3}} \quad \text{(N)} \qquad (1)$$

Where, V_{spu} is the shear strength (N) of a MFST; A_{sp} is the area of the cross section of a steel tube (mm^2); and F_{spt} is the tensile strength of a steel tube.

It was demonstrated that the shear strength of MFST can be conservatively evaluated by the sum of the shear strength of mortar and steel tubes.

3.2 Push-out Test

3.2.1 Test Program

Push-out test assumes the actual condition where the shear connector is inserted in concrete. Based on this test, a shear strength evaluation formula was developed and the slip displacement and secant stiffness were identified. Fig.5 shows the push-out test setup.

2 types of outer diameters, 34mm and 51.6mm were assumed for MCY. The length from the steel plate surface to the tip of the shear connector was set to be approximately twice as much as the outer diameters. Therefore, if the outer diameter is 51.6mm, the length is about 100mm (=51.6mm×2), and the total length would be twice the amount plus the thickness of a steel plate, that is, 216mm. Similarly, if the outer diameter is 34mm, the total length would be 144mm. The compressive strength

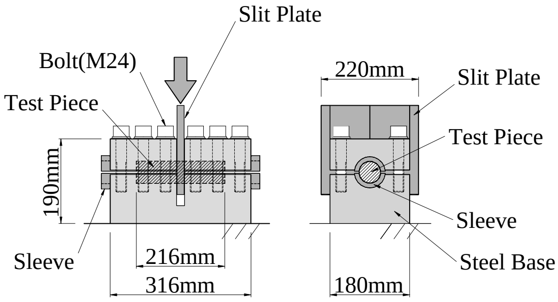


Fig.3-Device of Direct Shearing Test

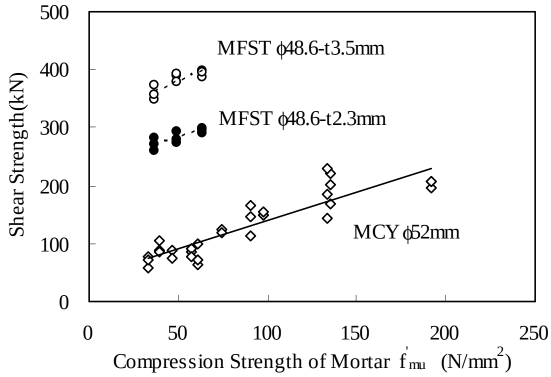


Fig.4-Results of Direct Shearing Test

of mortar was set at 115-223N/mm^2. Then, MFST with three different outer diameters were prepared, namely, 34.0mm, 48.6mm and 60.5mm. All of these are ready made products of STK400, with two different thicknesses, namely, 2.3mm and 3.5mm. The compressive strength of mortar to be filled is set at 73.3-129N/mm^2. The difference between hole diameter of steel plates and outer diameter of MCY and MFST was set at approximately 1mm, and epoxy resin was filled into the gap. The compressive strength of concrete blocks was 35.2-50.3N/mm^2.

A test was also carried out on perfo-bond strips for comparison with cylindrical connector (PBL-1~3). The diameter of the holes of perfo-bond strip was set at 53mm, and three types of penetration re-bars, D10, D19 and D22 were prepared.

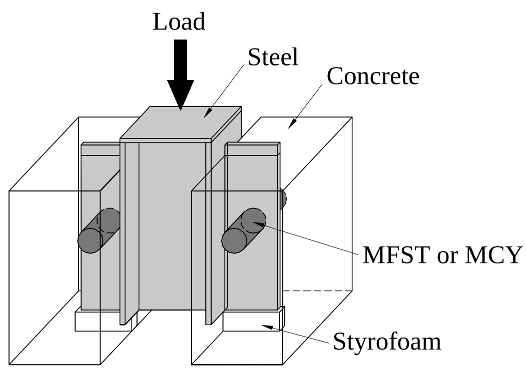


Fig.5-Pushout Test Setup

The test specimen was installed on the steel plate with sufficient rigidity. A width stopper was installed on the concrete blocks to restrict the horizontal movement. Loading and unloading was repeated at every increase by 0.2mm until the slip displacement became 4.0mm.

3.2.2 Test Results

Fig.6 shows the relation between the shear strength of MCY and the compressive strength of mortar. The horizontal axis shows the cross section area of the cylinders multiplied by the compressive strength of mortar. It was discovered from Fig. 6 that the shear strength of MCY obtained by the push-out test can be expressed by a direct function of the compressive strength of mortar, in the similar way as the results of the direct shearing test. The correlation function of linear approximation was 0.961 and there was strong correlation. Therefore, this formula was proposed as the shear strength evaluation formula of MCY. The following formula (2) is the proposed formula.

$$V_{mu} = 0.470 \times A_m f'_{mu} + 107 \times 10^3 \quad \text{(N)} \qquad (2)$$

$$109 \times 10^3 \leq A_m f'_{mu} \leq 466 \times 10^3 \quad \text{(N)} \qquad (3)$$

Where, A_m is the cross section area (mm^2) of a MCY, and V_{mu} is the shear strength of mortar. Considering variation in the experimental values, we drew a line double the standard deviation shifted lower to the bottom. This line gave close agreement with the approximate expression of the direct shearing test results. Therefore, the following formula (4), which is an approximate expression based on the direct shearing test results, was proposed as a design formula, to take some conservatism into consideration.

$$V_{mu} = 0.470 \times A_m f'_{mu} + 42.3 \times 10^3 \quad \text{(N)} \qquad (4)$$

$$70 \times 10^3 \leq A_m f'_{mu} \leq 466 \times 10^3 \quad \text{(N)} \qquad (5)$$

The following formula (4) was obtained within the range where the outer diameter of a MCY was 34-52mm, and the compressive strength range of mortar was 73.1-223N/mm^2, $t/d \geqq 0.3$(Where t is thickness of a perforated steel plate, and d is the outer diameter of a MCY).

Fig.7 shows the values obtained by the shear strength experiment of MFST. The horizontal axis is the sum of the shear strength of the mortar section, and the shear strength of steel tubes. The shear strength of the mortar section was expressed in one item of the formula (2), to which the shear strength of steel tubes expressed by the formula (1) was added.

It was found out from Fig. 7 that the gradient of the approximate expression of the test values was very close to 1.0, while the correlation

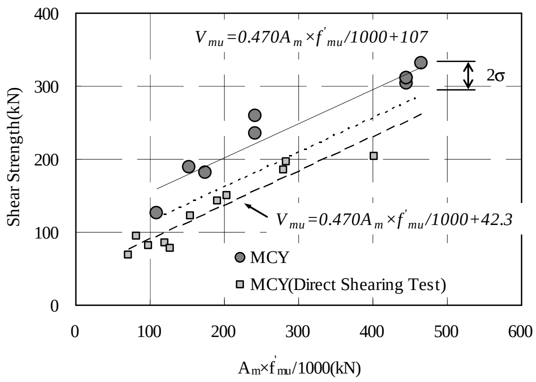


Fig.6-Shear Strength of Mortar Cylinder

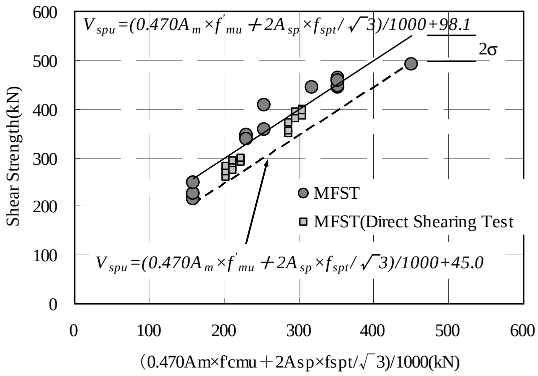


Fig.7-Shear Strength of Mortar Filled Steel Tube

coefficient was as high as 0.957, showing a strong correlation. Therefore, the approximate line shown in formula (6) was set to be the shear strength evaluation formula of MFST.

$$V_{spu} = 0.470 A_m f'_{mu} + 2.0 A_{sp} f_{pst} / \sqrt{3} + 981 \times 10^3 \quad \text{(N)} \qquad (6)$$

$$158 \times 10^3 \leq 0.470 A_m f'_{mu} + 2.0 A_{sp} f_{spt} / \sqrt{3} \leq 450 \times 10^3 \quad \text{(N)} \qquad (7)$$

Concerning the shear strength design formula for MSFT, we proposed the formula (8) which was double the standard deviation shifted lower to the bottom, considering variation in the test values.

$$V_{spu} = 0.470 A_m f'_{mu} + 2.0 A_{sp} f_{spt} / \sqrt{3} + 45.0 \times 10^3 \quad \text{(N)} \qquad (8)$$

Formula (8) is true within the range where the outer diameter of MFST is 34-61mm, thickness is 23-3.5mm, compressive strength of mortar is 73.3~129N/mm^2, and $t/d \geqq 0.3$. This formula shows that the shear strength of MFST can be conservatively expressed with the direct shearing test results also reflected.

Fig. 8 shows the comparison between the shear strength obtained by push-put tests and the shear strength of perfo-bond strips (PBL). Solid and dotted lines show the calculated values based

on the Standard Specifications for Composite Structures, on the assumption that the compressive strength of concrete is 40N/mm^2. Concerning PBL, the relation between the outer diameter of perforated steel plates and the outer diameter of steel bars is defined on the assumption that concrete is reliably filled into the holes. According to the Standard Design of Railway Structures, etc. and its Explanation[12)], the outer diameter of a perforated steel tube shall be at or larger than the sum of the outer diameter of a steel bar and the aggregate size. On the other hand, the Design Manual Volume 2 issued by the Japan Expressway Companies13) prescribed that the outer diameter of a perforated steel tube shall be at or larger than 3/4 of the outer diameter of
a steel bar and the aggregate size. In this study, the aggregate size is assumed to be 25mm and the range of the outer diameter of a perforated steel tube was set based on the prescription of the latter, corresponding to the outer diameter of a steel bar.

The shear strength of MCY and MFST are exceeds that of PBL without re-bars inserted in perforated plates. The shear strength of MCY with the outer diameter of 53mm is at or below the shear strength of PBL with re-bars inserted in perforated plates. On the other hand, it is shown that the shear strength of MCY with the outer diameter of 34mm and MFST with the outer diameter of 34mm and 48.6mm exceeds the range of the shear strength of PBL with re-bars inserted in perforated plates.

Thus, it is shown by this figure that although the shear strength of MCY and MFST does not considerably exceed the shear strength of PBL, but there are effective in the cases, including when re-bars inserted in perforated plates cannot be used, or when the outer diameter of steel bars is restricted.

3.2.3 Slip Characteristics

Slip characteristics of MFST were compared with those of perfo-bond strips and studs. Secant stiffness of studs is used as an inclination of the initial secant line of $V_{max}/3$ (V_{max} is the shear strength) in the envelope curve of relation between shear strength-slip displacement (Standard Specifications for Hybrid Structures JSCE 2009). This is because if the secant stiffness is about 1/3 of the shear strength, it is considered to be within the range where shear connector behave elastically.

Fig. 9 shows the relation between the residual slip displacement of MFST and V/V_{max} before unloading. In all the test results, the residual slip displacement is extremely small if V/V_{max} is about 0.3 (=$V_{max}/3$), and thus, it is within the range where MFST behave elastically.

The secant stiffness of MCY was 685-2776kN/mm, and showed relatively wide variation. No correlation was observed between the secant stiffness and the outer diameter of a cylinder and the thickness of steel plate. The values of the secant

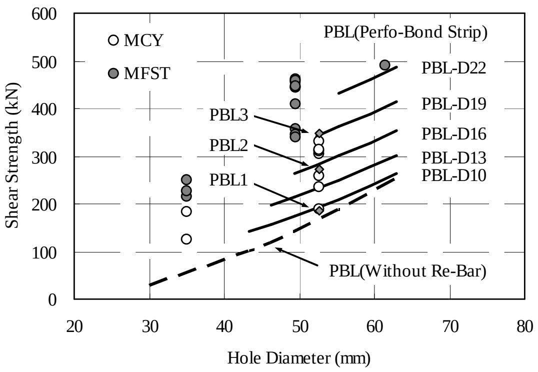


Fig.8-Comparison of Shear Strength

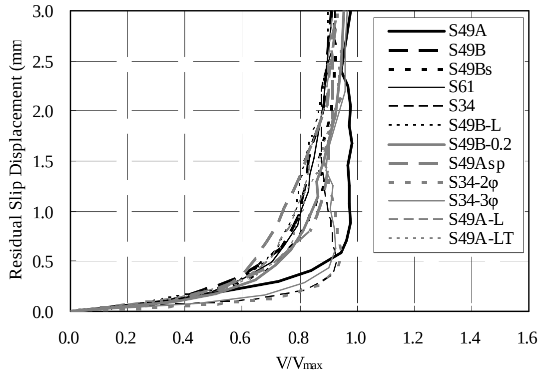


Fig.9-Residual Slip Displacement of MFST

stiffness of MFST were 487-835kN/mm, showing a relative stability. No correlation was observed between the secant stiffness and the outer diameter and the thickness of a steel tube. The average value of the secant stiffness of MFST was 673kN/mm, which is smaller than that of MCY.

The reference values of the secant stiffness of studs and PBL are stated in the Design and Construction Standards of Composite Bridges 2007. The values are 90-230kN/mm for studs of ϕ13- ϕ22. Concerning PBL with a hole diameter of 35mm, the secant stiffness is 432-1190kN/mm when the thickness of steel plate is 8mm, and 1146-2009kN/mm when the thickness is at or more than 12mm. In comparison to these values, the secant stiffness of MCY is larger than that of studs and almost equivalent to PBL. MFST is intermediate between those of studs and PBL. The secant stiffness obtained from the PBL1-4 test results was 616-1690kN/mm.

Residual slip displacement at $V_{max}/3$ was 0.03-0.07mm for MCY and 0.05-0.12mm for MFST. According to the Standard Specifications for Composite Structures 2007, if studs and PBL are designed in such a manner to make residual slip displacement approximately 0.1mm, slip displacement will not rapidly increase. As the test results of MCY and MFST were both at and below approximately 0.1mm level, it was found out that residual slip displacement can be limited to the same level as studs and PBL.

All things considered, the newly developed shear connector has sufficient shear strength and secant stiffness in comparison to studs and PBL. Also, as residual slip displacement of this shear connector at $V_{max}/3$ is sufficiently small, this device can be of practical use as a shear connector.

3. A STUDY ON JOINT OF COMPOSITE STEEL GIRDER AND PC GIRDER USING SHEAR CONNECTING METHOD

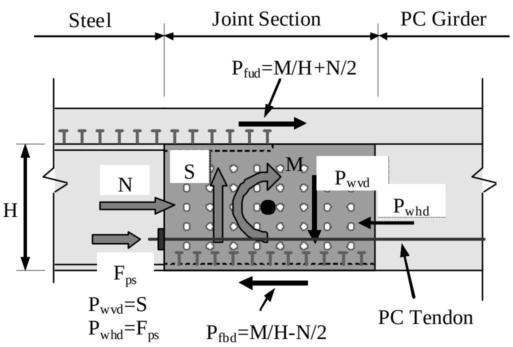
3.1 Overview of Joint Structure

In this joint structure, the steel member of a composite steel girder is inserted into a PC girder, and they are connected by the shear connectors. The shear connectors are installed on the upper face of the upper and lower flanges and also on the web plate. PC tendons are extended as far to the joint section, and anchored at the multiple cross sections by providing additional anchoring blocks on the side of the joint section. The shear stress generated in the shear connectors with pre-stress can be dispersed by installing anchors on multiple cross sections. The upper flange of a girder was installed only in half of the joint section to reduce the local stress based on the results obtained from the preliminary analysis discussed below. The length of the joint depends on the layout of the shear connectors and the additional anchoring blocks of PC tendons, etc. Concerning the trial design of the actual structure and the specimen modeled on the basis of the trial design, the length of the joint was set at 2×D (D is the height of the girder).

3.2 Design of Shear Connector at Joint Section

A simple design method of the shear connectors arranged in joint section was proposed in reference to the previous instance of shear connecting methods, and of rigid connection between steel girder and RC pier. Studs installed at upper and lower flanges of a steel girder were assumed to resist the shear force generated by bending moment and axial force. The shear connectors installed at web plate were assumed to resist the shear force generated by shear force and pre-stress. Axial force was assumed to be shared equally to studs on upper flange and lower flanges of the girder. Fig. 10 shows the relation between the cross-section force and pre-stress acting on the joint and the shear force acting on the shear connector.

Number of shear connectors required for the cross-section force at the ultimate state can be calculated by dividing the shear force generated in each shear connector shown in Fig. 10 by the shear strength per shear connector. We decided to obtain the shear force acting on the shear connectors of web plate by adding each number of shear



M : Bending Moment, S: Shear Force, N: Axial Force
F_{ps} : Prestress
P : Various Forces Transmitted to Shear Connectors
P_{fud} : at Upper Flange, P_{fbd} : at Lower Flange
P_{whd} : at Web (Horizontal), P_{wvd} : at Web (Vertical)

Fig.10-Cross Section Force Transmitted to Shear Connector in Joint Section

connectors required in vertical and horizontal directions.

With regard to the length of the joint, the length more than required for installation of the shear connectors and PC tendons shall be secured. It is considered that the longer the length of the joint, the more stable the structure becomes. On the other hand, a preliminary analysis was performed to examine the effects if the length of the joint is reduced. A three-dimensional non-linear finite element analysis approach, discussed in section 3.4, was taken in the preliminary analysis. The subject of the analysis was the joint model specimen used in this study with the joint length of 1×D. Upper flange was installed over the entire length of the joint section. The shear connector was designed by the method shown in Fig. 10. Only bending moment is applied on the joint by four-point loading.

The analysis results are shown in Fig. 11. Bending compression failure was occurred in underneath of loading plate at PC girder side. This figure shows the strain contour in the vertical direction to the girder when half of the maximum load was applied. Tensile strain becomes larger as

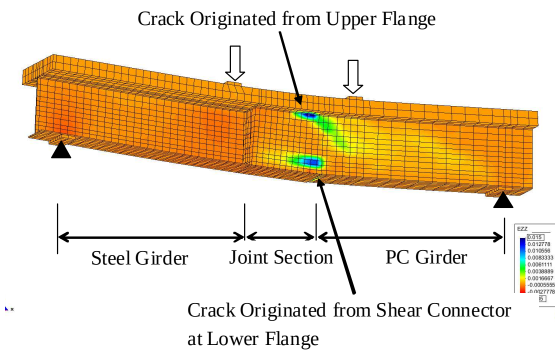


Fig.11-Result of Preliminary Analysis

the color becomes darker. The figure shows that large strain was generated in the upper and lower parts of the joint on the side of the PC girder. Upper strain was generated by the action of the bearing force in the direction that the upper flange pressed down concrete, and this large strain was extending obliquely downward on the side of the PC girder. On the other hand, in the lower part of the girders, the lower flange was displaced in the direction of coming off from concrete. Therefore, large strain seems to be generated by relatively large vertical shear force acting on the shear connector at this point. These strains seem to be generated by the rotation of the web plate at the joint.

Fig. 12 shows the design concept of the shear connector at the web. The parts shaded by dark gray show the force generated to be in balance with the bending moment originated from the rotation of web plate. In the upper right of the figure, the force of the flange pushing down concrete is acting, and in the lower right of the figure, the force of the flange coming off from concrete is acting. This figure agrees well with the results of the preliminary analysis.

Thus, it is shown that if the length of the joint is reduced, the unexpected destruction may occur on the joint due to web plate rotation. Therefore, the following two points were taken into consideration in the design of the shear connector. One is to install the upper flange only to the middle of the joint. In this way, cracks generated in the above-mentioned analysis can be prevented. The flange shall be cut off where the force between the flange and concrete shown in Fig. 12 becomes the minimum.

Another point to be considered is to add the number of shear connectors to the web required to resist the rotation of the web plate. Originally, the rotating force of the web plate is bending moment shown in Fig. 10. So, it is redundant to install shear connectors for this moment in the web. However, the direction of the shear force acting on the shear connector considered here is different. Also, if some force similar to the force generated between the flange and concrete shown in Fig. 12 is generated, various types of phenomenon undesirable in terms of durability may occur, for example, tensile force in axial direction acting on studs, or generation of gaps between steel plate and concrete.

We have decided to design the shear connector of the girder webs by considering the balance between bending moment and the shear force, P_n, generated in the shear connector. Shear force acts on each vertical column of the girder web, like *P1* and *P2*, against the bending moment around the axis of rotation set in the center of the joint. The following formula can be obtained by assuming that the distance between the axis of

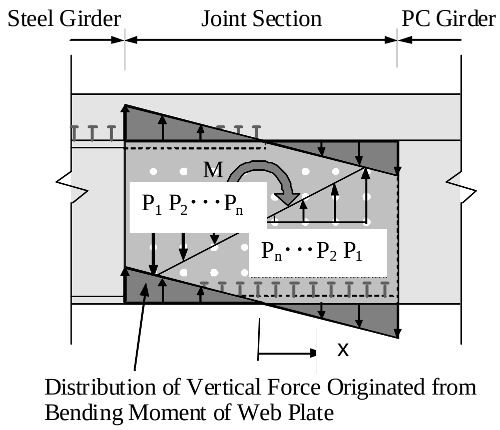


Fig.12- Design Concept of Shear connector at Steel Girder Web

rotation to the point where the shear force is acting as x1, x2, and so on. The number of shear connectors in each vertical column shall be set so that the formula (10) can be true.

$$M = \sum_{n=1}^{n} P_n \cdot x_n \tag{9}$$

$$P_n \le V_u \times N \tag{10}$$

Where, V_u is the shear strength per shear connector. Assuming that the horizontal and vertical distance between shear connectors is set at a certain value, the number of columns of shear connectors shall be decided by the shear force acting on the furthest column to the center of rotation. The shear connectors for the test specimen discussed in Section 3.3 shall be designed based on this method.

3.3 Specimen

The specimen is half the size of the actual structure and one of the two main girders was modeled. Concerning the size of a composite steel girder, the height of a girder is 1.1m, a slab thickness is 0.2m. The steel girder and the slab are joined by studs with the shaft diameter of 16mm and the length of 80mm. The height of the PC girder is 1.1m, the same as the composite steel girder, and 10 PC tendons of 1S19.3 are installed. The length of the joint is 2.5m. Six of ten PC tendons are anchored on the edge face of concrete on the composite girder side, and the remaining 4 tendons are fixed by the additional anchoring blocks installed on 2 cross sections.

Studs with the shaft diameter of 16mm and the length of 80mm are used for upper and lower flanges in the joint. In the specimen named S-1, same size studs are arranged in webs, while in the specimen named K-1, MFST are used for webs. Steel tubes with the outer diameter of 48.6mm, thickness of 3.5mm and length of 200mm are used for MFST. These tubes are filled with approximately 80N/mm^2 of high-strength mortar. MFST are inserted into the perforated steel plates with the outer diameter of 49mm and fixed by epoxy resin (Fig. 1).

Width of the slab, etc. was set in such a manner that the upper and lower edge stress becomes identical when the design load is applied in the actual structure based on the trial design and the test specimen. Also, these were reproduced by four-point loading. Joint section was set between two loading points, and the load was controlled so that the load on both loading points becomes identical. The total length of the girder is 9.5m, while the shear span is 2.5m, and the distance between the loading points is 3.5m. Fig. 13 shows a scene of the test.

The load stated in the following description shall be the sum of the load at two loading points. Unloading was performed at the design load (approximately 800kN) and also at 1/3, 2/3 and 3/3 of the calculated bending strength of the PC girder.

3.4 Details of Analyses

Three-dimensional nonlinear finite element analysis (hereafter "FEM") was performed (Maekawa et al., 2003). The analysis model is shown in Fig. 14. Reinforced concrete is modeled by the solid element having 8 Gauss points, and the non-linear constitutive model of RC element based on the multi-directional smeared crack approach was applied. In the constitutive law, an elasto-plastic fracture model was applied to the compressive side, and a tension stiffening model was applied to the tensile side of the concrete (Okamura and Maekawa 1991). In the meantime, shear transfer in fixed crack surface was taken into consideration. Softening parameters of the compressive and tensile model were decided considering element size and fracture energy.

The relation of average stress and average strain was given to the tensile constitutive law of steel bars, considering effects of adhesion with concrete. PC tendons were separately modeled by the truss element, to which tri-linear stress-strain relation was given. Steel girders, loading plates and PC tendon anchor plates were also modeled by the solid element having 8 Gauss points. The Von Mises yield condition was given to the steel girder elements, and the loading plates and anchor plates were assumed to have linear elasticity. Joint element was deployed between the girder element and RC element, while considering contact, exfoliation and slip between the two elements.

Shear connectors were modelled by spring elements connecting nodal point of steel girder elements and RC element separated by the joint elements. Spring element was assumed to have linear elasticity in off plane direction, and non-linear elasticity in tangent plane direction.

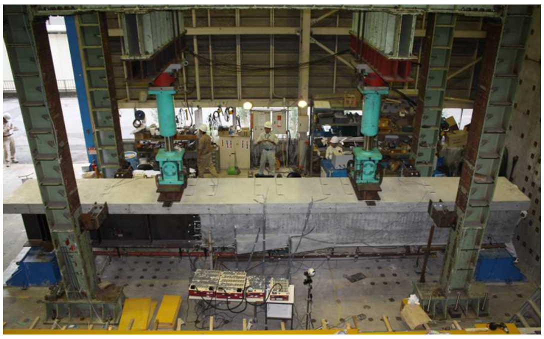


Fig.13-Loading Test of 1/2 Scaled Model

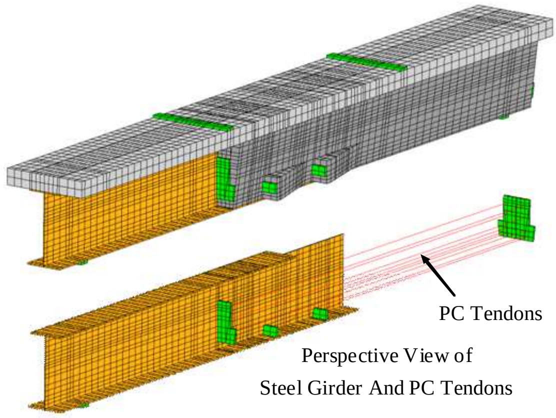


Fig.14-FEM Model

For studs, the relation of shear force and slip displacement provided in the Standard Specifications for Hybrid Structures JSCE 2009 was used. For MSFT, the following formula (11) which was approximated to envelope nearly the lowest line of the relation between shear force and slip displacement obtained by the push-out test of six specimens was input.

$$V_{sp} = 401 \times (1 - e^{-\delta})^{0.652} \qquad (kN) \qquad (11)$$

Where, V_{sp} is shear force (kN) per dowel of MFST, and δ is slip displacement (mm).

3.5 Results

3.5.1 Load-Displacement Response

Fig. 15 shows the relation between the load and the displacement at the center of the specimen. In both specimens, the load increased to twice as large as the design load almost linearly. Then, with the increase of cracks on the PC girder, increase in the load became gradual after exceeding 3160kN, which is the calculated ultimate bending moment of PC girder. When deformation exceeded 50mm, upper edge of concrete was compressively broken near the loading point of the PC girder, and the load significantly dropped. Although reduction in stiffness after the occurrence of initial cracks was slightly faster in K-1 specimen, the difference was very small. The FEM values relatively accurately

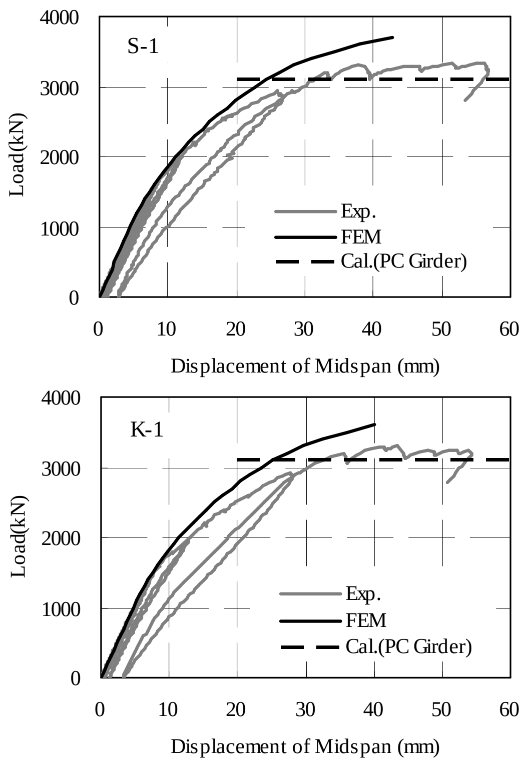


Fig.15-Load Displacement Response

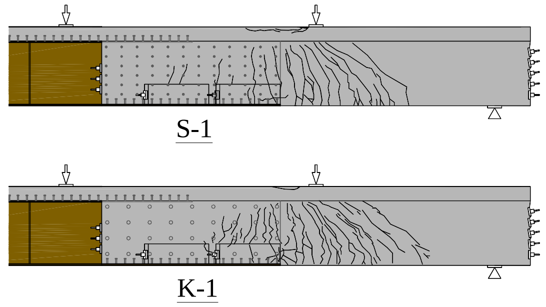


Fig.16-Cracks in Specimen

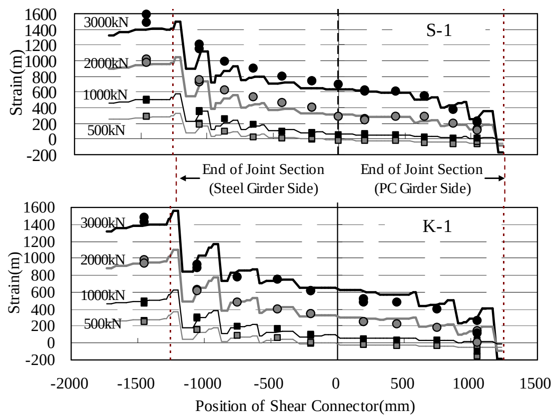


Fig.17-Strain Distribution at Web Plate

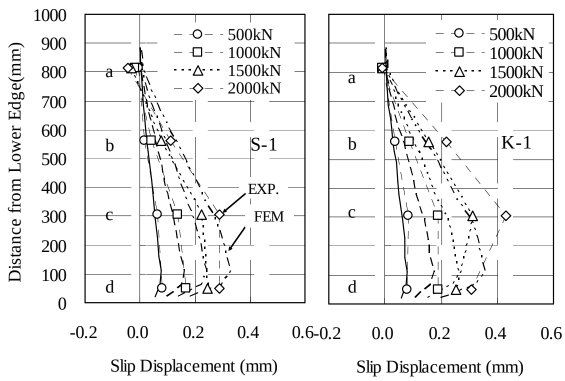


Fig.18-Slip Deformation of Concrete and Web Plate at the end of Joint

reproduced the initial stiffness and behavior after cracks.

Fig. 16 shows the condition of cracks, which started at 1600kN in S-1 specimen, and at 1400kN in K-1 specimen, in the vicinity of the lower edge of the loading point on the PC girder side. Increase of cracks at the joint became apparent both in terms of numbers and length after 2000kN was exceeded. Number of cracks that occurred in K-1 specimen was slightly larger than that of S-1 specimen. Although number of cracks at the joint of S-1 specimen was comparatively fewer, it can be pointed out that in the case of S-1 specimen, cracks occurred in wider area in the joint section.

3.5.2 Strain Distribution at Web Plate

Fig. 17 shows strain distribution in the direction of girder axis of steel girder webs according to loading levels. Tensile strain increases with the increase of the load, and the FEM values agree well with the test results. The size of strain gradient is in proportion to the shear force generated in the shear connector. This figure shows that strain gradient increases near the end of the joint, and it is expected that with this increase, the shear force generated in the shear connector also increases. On the other hand, strain gradient is relatively small near the center of the joint, and thus strain gradient generated in the shear connector is expected to be small. FEM values show the strain gradient distribution in a pulsed state. The size of the pulses is large at the points with large strain gradient, confirming that the shear force generated in the shear connector becomes large.

3.5.3 Slip Deformation of Concrete and Web Plate

Fig. 18 shows slip displacement between the steel girder and concrete at the edge face on the composite steel girder side of the joint. Four measurement points are set at regular intervals in the direction of the height of the girder, and show the average values of the two faces. Slip displacement tends to progressively increase in the lower side of the girder although near the lower flange, slip displacement tends to become smaller due to restrict of studs. In K-1 specimen, increase in slip displacement becomes apparent after 1500kN, and so the difference from the FEM values becomes slightly bigger. At the expected design load (about 800kN), slip displacement is at or below 0.2mm, which can be considered sufficiently small.

3.5.4 Shear Force of Connector at Web Plate

Shear force generated in the shear connectors was examined based on the FEM values. Fig. 19 shows distribution of shear force generated in shear connectors at web plate in the range of 500kN-2000kN. These are the values of the shear connector installed at the lowest column where the largest shear force was generated. All the values are values per shear connector. Concerning shear connectors at web plate, the values are also per connector on one face. In the same figure, 1/3 of the shear strength of studs and 1/3 of the shear strength of MFST (on one face) are also shown for reference.

At web plate, the shear force generated at the edge of the joint is large. In the case of studs of S-1 specimen, the shear force is 16kN at the design load of 800kN, while in the case of K-1 specimen equipped with steel tube dowels, the shear force is about 20kN. These values are about 53% and 30%, respectively, and thus relatively small in comparison to the reference values. This is because the shear connectors on both web plate and flanges resist the shear force applied by bending moment. Thus, it is clear that the shear force generated in the shear connectors can be restricted to considerably small if the number and the layout of the shear connectors are set by the design method proposed in Section 3.2.

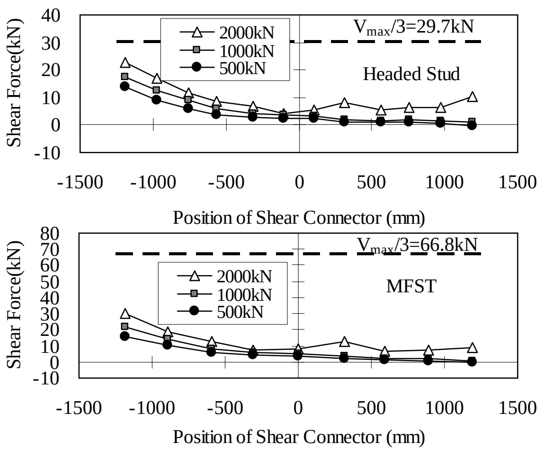


Fig.19-Shear Force Generated in Shear Connector at Web Plate

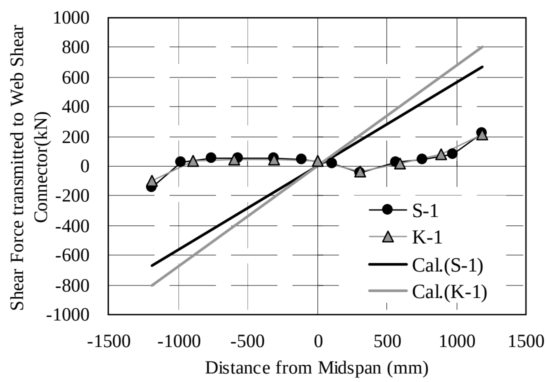


Fig.20-Comparizon of Shear Force transmitted to Shear Connector at Web with Design Values

3.5.5 Shear Force of Connectors Originated from Rotation of Web Plate

Fig. 20 shows the comparison of the shear force generated in the shear connectors at web plates in the vertical direction at 3000kN and the design values. The generated shear force was the sum of the shear force generated in the shear connectors installed in one vertical row (S-1: 6 points, K-1: 4 points). Design values are the sum of the shear force of each row, calculated on the basis of the method proposed in Section 3.2. Concerning the symbols attached to the shear force, the direction of shear force acting on the shear connectors when the web plate is pushed down is set to be positive.

There is a tendency that shear force generated in the shear connectors in vertical direction becomes larger at both end of the joint. As positive and negative of these values reverse at both ends of the joint, they seem to be generated by rotation of the web plate. The generated shear force converges to nearly zero immediately after entering inside of the joint.

The size of the generated shear force is considerably smaller than the values envisaged in the design. Possible reasons include the fact that the length of the joint of the specimen is twice as long as the height of the girder and that studs of the upper and lower flanges also resist the rotation, and also that axial tensile stiffness of studs contributes to resisting the force in the direction of flanges coming off from concrete. On the basis of these results, it is demonstrated that the joint can be safety designed by the method proposed in this study, against the rotation of web plate at the joint, by limiting the shear force generated in the shear connectors in vertical direction. Contributions of flange studs and bearing force of flanges that resist the rotation of the web plate shall be studied in more detail as our subject of future investigation.

3.5 Conclusion

With regard to the joint structure of 2 main girder type composite steel girders and PC girders based on the shear connecting method, we proposed a design method of shear connectors and demonstrated the following matters by performing the loading test on a 1/2 scaled model of the actual structure designed by the above-mentioned method, and FEM analysis:

1) We proposed a design method of simple shear

connectors assuming that upper and lower flanges resist to bending force acting on the joint and that shear connectors at web plate resist to shear force and pre-stress. We also proposed additional installation of shear connectors at web plate to resist the rotation of web plate at the joint.

2) As results of various tests, it was confirmed that destruction is caused by bending destruction of the PC girder, and also that shear force generated in the shear connectors can be limited to considerably small if the shear connectors are designed by the method mentioned in 1).

3) It was confirmed that the behavior of the joint can be reproduced in detail by 3-D FEM analysis in which the relation between the shear force and slip displacement is modeled by non-linear spring elements, based on the push-out test results. Validity of the simple design method was proven by the analysis of the shear force generated in the shear connectors.

4) It was confirmed that the joint structure using MFST is feasible based on the bearing force of the specimen where MFST are used as shear connectors at web plate and also the behavior of the joint. Adoption of this type of MFST can help reduce the number of onsite welding work and also the number of shear connection points.

4. OTHER APPLICATION

4.1 Joint of Precast Web and Slab of PC Box Girder using MFST as Connection Key

Fig. 21 shows the butterfly web box girder bridge that eliminates the installation of re-bars in the web and reduces the weight of the girders, by using concrete reinforced by high-strength steel fiber in the pre-cast web. MFST are applied to the joint of this pre-cast web and the slab. The reasons behind the adoption of this connector include that it is difficult to install a shear key at the web plate in structural terms, and that there is a concern that the steel bars become overcrowded only by using shear connectors. When applying this approach, it was concerned that sufficient shear strength cannot be obtained because fracture splitting cracks may occur as the cover concrete of the MSFT is small. Therefore, the effects of the size required for covering and the amount of steel fiber mixed into concrete on the shear strength were studied by performing a push-out test. As a result of the test, it was revealed that fracture splitting cracks will not occur if covering is at or more than 125mm, and the amount of steel fiber is at or 0.4%vol, and that the shear strength can be estimated by the shear strength evaluation formula proposed in Section 3.2.2. Moreover, a simple model to evaluate the reduction of shear strength by the generation of fracture splitting cracks was developed and it was demonstrated that the experiment results can be validated with high accuracy.

Fig.21-Butterfly Web Box Girder Bridge (Ashizuka et al., 2012)

Fig.22-MFST Setup in Butterfly Web

4.2 Composite Main Tower of Extra Dosed Bridge using Re-Bar Connector

We studied the application of shear connectors equipped with steel bars inserted in the perforated steel plates (hereafter “re-bar connector”) to the anchoring system for PC cables of the main tower in extra dosed bridges. The main tower consists of two concrete piers on both side edges serving to anchor PC cables, and a steel plate installed at the center. PC cables are installed in parallel to both side of the steel plate. Re-bar connectors are used for connecting the concrete columns and steel plate. It was confirmed by the push-out test performed on the re-bar connector in advance that the shear strength can be evaluated by the theoretical pure

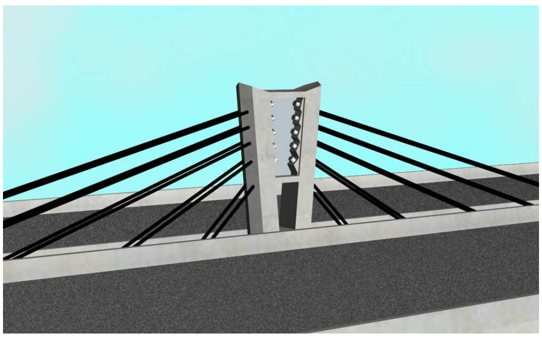


Fig.23-Conceptional Drawing of Composite Main Tower of Extra Dosed Bridge

shear strength of the steel bars. Loading test was performed on the real scale model of the joint which was designed based on the above strength. Based on the results of the experiment, it was demonstrated that the behavior of the joint is within the range of elasticity at the tensile force applied on the PC cables when the joint is actually put into service as envisaged in the design, and also that destruction does not occur at the joint even if large load equivalent to the tensile force of the PC cables is applied.

Fig.24-Re-Bar Connector

4. CONCLUSIONS

The new type shear connectors that we developed have sufficient shear strength even compared with conventional shear connectors. Slip displacement and residual slip deformation at $V_{max}/3$ were also sufficiently small. Thus this shear connector can be put into practical use.

Studs have been playing the major role as a shear connector for steel and concrete for very long time. With recent development of various types of composite structures, perfo-bond strips and block dowels have appeared, enabling us to choose suitable shear connectors according to the construction conditions. MCY and MFST that we developed in this study are produced for the purpose of overcoming the challenges posed by perfo-bond strips which also utilize perforated steel plates. As these techniques show sufficient performance compared to perfo-bond strips, they have huge potential that they can be applied even in the locations where perfo-bond strips are difficult to be used. We expect that application of these techniques will expand to increasingly various types of joints.

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