Comparison of Studs and Perfo-Bond Strips on Mechanical Behavior of Composite Girders under Negative Bending

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Abstract: The purpose of the present study is to investigate the influence of different types of shear connectors on mechanical behavior of composite steel and concrete girders under negative bending moment. Two overturned simply supported steel-concrete composite girders with different shear connectors including studs and PBLs (perfo-bond strips) were tested under point load in the mid-span. Based on the experimental observations, a three-dimensional FE (finite element) model capable of analyzing the composite girders subjected to negative bending moment was built. Load and deformation response, concrete initial cracking and composite girder ultimate load bearing capacity, strain development process of reinforcing bars before and after concrete cracking were observed in the test and compared with the numerical values. Results predicted by this modeling method are in good agreement with those obtained from the tests. Furthermore, the “crack closure” or “through crack” load were recorded by π -gauges in the tests and compared with the code-specified ultimate load.

Key words: Composite steel and concrete girder, negative bending moment, studs, PBLs.

1. Introduction

For continuous steel-concrete composite girders, the negative bending moment acting in the support regions will generate tensile stresses in the concrete slab and compressive stresses in the lower steel profile. As a result, the mechanical behavior of these girders is strongly nonlinear even for low stress levels, due not only to the slip at the beam-slab interface, but also the cracking in the concrete slab, which generally has shortcomings in view of durability and service life of the structures. Therefore, special considerations for composite girders under negative bending moment are necessary. For this reason, experimental and numerical studies about inelastic behavior of composite girders under negative bending have been done widely. Special attentions were generally given to crack control methods [1, 2], failure modes [3, 4],

fatigue behaviors [5] and long-term behaviors [6], and so on. In addition, some numerical models were also created to simulate the composite girders under negative bending. Those numerical models keep a watchful eye on the effect of slip on the steel-slab interface [7, 8], partial shear connections [9], or improved numerical methods [10, 11].

Besides, studs and PBLs (perfo-bond strips), generally known as flexible and rigid shear connectors separately, are most frequently-used shear connectors in composite girders. However, very few studies were performed particularly to investigate the effects of different shear connectors on mechanical performance of composite beams subjected to negative bending moment. For this reason, this paper presents the experimental and numerical results of two composite girders subjected to negative bending. The objective of the present study is to clarify the difference between studs and PBLs on inelastic mechanical behavior of composite girders subjected to negative bending moment.

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2. Summary of Tests

2.1 Details of Test Specimens

Two overturned simply supported composite beams of CBS (studs as shear connectors) and CBP (PBL as shear connectors) were tested under concentrated load in mid-span. Each of the specimens was 4.6 m in length and was simply supported at a span of 4 m. The concrete slab thickness was 250 mm with a width of 800 mm. Different shear connectors, including 22 mm nominal diameter headed shear studs and PBLs with the thickness of 12 mm were used to connect the concrete slab and the steel girder in CBS and CBP, respectively. The typical geometry of test specimen is

shown in Fig. 1 and the details are illustrated in Table 1.

2.2 Instrumentations and Test Setup

Measurement of strains in the reinforcements, structural steel and concrete slab were conducted by means of electric resistance gauges at sections located at mid-span, 20 cm and 60 cm from the mid-span, respectively. Vertical deflections and transverse deformations were measured by mean of deformation gauges, as shown in Fig. 1a. Measurement of slip between the concrete slab and steel beams was conducted by means of deformation gauges, and the π -gauges for measuring the crack width on the

Fig. 1 Dimensions of test specimen (mm).

Table 1 Details of test specimens.

Specimen	Connection device	Shear connectors spacing (mm)		Reinforcing bars spacing (mm)	
		Longitudinal spacing	Transverse spacing	Longitudinal bars	Stirrup
CBS	Stud	150	100	100	150
CBP	PBL	150	240	100	150

concrete slab were also employed in the test. The test specimen was supported by a roller system at two ends. After the drying shrinkage had stabilized, pre-loading was applied to check the reliability of the measure equipment and the stability of the test specimen. The test girder was loaded in four cycles with the maximum loads of 200 kN, 400 kN, 700 kN and 1,300 kN before the ultimate load was reached [12]. The loading was terminated when either the maximum stroke of the jack was reached or when the load capacity of the test specimen dropped significantly.

3. Numerical Simulation of Experiment

3.1 Model Building

The modeling of each numerical model was carried out in three dimensions using the finite-element method and the DIANA software, as shown in Fig. 2. Solid elements, shell elements and spring elements were used to simulate the concrete slab, steel girder and shear studs, respectively. Also, in order to account for the slip between concrete slab and steel girder, interface elements were employed in the numerical analysis.

3.2 Material Properties

3.2.1 Concrete

Material property of concrete was determined from material test. The nominal concrete compressive strengths and tensile strengths achieved after 28 days of curing for CBS and CBP was (35.5, 2.50) and (29.1, 2.18) N/mm², respectively. The often quoted stress-strain curve due to Hognestad [13] was employed to simulate the compression behavior of concrete, as shown in Fig. 3. Experimental tension-stiffness curve proposed by Nakasu et al. [14] are used to simulate the tension softening behavior of concrete.

3.2.2 Structural Steel and Reinforcing Bars

The uniaxial stress-strain relationships for structural steel and reinforcing bars shown in Fig. 4 were

adopted in numerical models based on the material tests. Three steel beam coupons were cut out from different components of the upper flange, web, lower flange and PBL with different nominal thickness of 16 mm, 22 mm, 25 mm and 12 mm, respectively, and the tensile tests were performed. Fourteen reinforcing bars of D19 nominal diameter were used for two layers as longitudinal reinforcing bars in the concrete slab, whilst D13 stirrup reinforcing bars were used as torsional reinforcement with a nominal diameter of 13 mm.

3.2.3 Interface Modeling

In order to account for the effect of the composite action between the steel girder and the concrete slab, interface element was used in the numerical models. As for the material properties of the interface, generally speaking, the shear force-slip response of interface or shear connectors depending on the slab-steel surface treatment, so there are no specified values reported. Simplified interface property by considering the bond-slip model can be described as shown in Fig. 5, which was used in this study to simulate the composite action between the concrete slab and steel girder. Based on previous comparison studies between experimental and numerical studies [15, 16], the maximum shear bond stress was taken as 2.4 N/mm² due to the application of rubber-latex on the interface and the corresponding initial slip Δu_0^0 was taken as the standard value of 0.06 mm suggested by Dörr et al. [17].

Fig. 2 FE model of test specimen.

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Fig. 3 Stress-strain curve for concrete.

Fig. 4 Multi-linear relationship for steel and re-bars.

3.2.4 Shear Connector

PBLs are simulated by shell elements in the finite element model, and the material property is illustrated in Fig. 4. The shear studs were modeled by using nonlinear spring elements. For each stud, three springs are used, two in horizontal direction and one in vertical direction. The constitutive relationship of the spring shown in Fig. 6 is given by Eq. (1) [18], in which the ultimate shear force loading capacity of studs was specified by JSCE specifications [19].

$$Q = Q_u (1 - e^{-0.7S})^{0.4} \quad (1)$$

$$Q_u = \min \left(\frac{{}^{31}A_{ss} \sqrt{(h_{ss}/d_{ss}) f_{cd}} + 10,000 / r_b}{A_{ss} f_{ss} / r_b} \right) \quad (2)$$

where, S is slip of the shear stud (mm), A_{ss} is area of the shank of the stud (mm^2), d_{ss} is diameter of the shank of the stud (mm), h_{ss} is height of the stud (mm), f_{cd} is design compressive strength of concrete (N/mm^2) ($= f_{ck} / \gamma_c$), f_{ck} is the characteristic compressive

Fig. 5 Material properties of interface.

Fig. 6 Constitutive relation of shear stud.

strength of concrete (N/mm^2), f_{ss} is design tensile strength of the stud (N/mm^2) ($= f_{sk} / \gamma_s$), f_{sk} is characteristic tensile strength of the stud (N/mm^2), γ_c is material factor of concrete ($= 1.3$), γ_s is material factor of stud ($= 1.0$), γ_b is member factor ($= 1.3$).

4. Results and Discussion

4.1 Load and Deflection Response

The load-displacement curves obtained from the numerical analyses are compared with the experimental data, as shown in Fig. 7. Both numerical and experimental displacements are taken from the vertical deflection at the bottom mid-point of the composite beam, which is the bottom point of the concrete slab. The loads corresponding to the theoretical yield and full plastic moments are plotted as the horizontal lines indicated by P_y and P_p , respectively. The ultimate pure bending moments, $M_{u,t}$

Fig. 7 Load-deflection relationship.

is to be calculated following the procedure for computing the plastic bending moment of composite sections under negative moment specified in Appendix D6.1 of AASHTO LRFD [20].

It is found that in the linear region, the load-displacement curves from numerical studies agree well with the measured results. However, in the nonlinear region after girder yielding, the rigidity of FE model is a little bit stronger than the test girder. This is presumably because the residual stress of steel girder is not considered in the finite element analysis, which may result in slightly larger stiffness of FE model than that of the test girder. At the final stage, the concrete can not sustain any increase in applied loading and eventually fails by through crack in the mid-span. The numerical results and experimental results are summarized and compared in Table 2.

The initial cracking load calculated with reference to the slab exhibiting maximum negative moment of composite section under elastic state has been estimated to be 247 kN for both CBS and CBP, were close to numerical results but larger than experimental values. It seems to indicate that the initial cracking load is very small for composite beams under negative

bending moment and thus making the structural behavior highly nonlinear even for very low stress levels. Regarding to the ultimate bending moment, as strain hardening of the steel girder and reinforcing bars were not considered in the theoretical analysis, resulting in that the experimental results was approximate to numerical results but much larger than the theoretical values. Moreover, comparison of load-deformation response of each specimen indicates that the displacement of CBP becomes a little less at around 1,000 kN than those of CBS, which is presumably because the PBL dowels affect the rigidity of the entire girder when cracking has progressed to a certain extent. Both experimental and numerical results indicate that the ultimate bending moment of CBP is larger than that of CBS.

4.2 Strain of Reinforcements

The strains of longitudinal reinforcements in the mid-span near the loading point were measured in the tests. Fig. 8 shows gauges' number (gauges 13, 15 and 17 were arranged for the upper layer reinforcements, and gauge 43, 45 and 47 were employed for the lower layer reinforcements) and locations. Strain development of the reinforcement before theoretical girder yielding load as well as the strain results during the whole loading process were illustrated separately. During the whole loading process, no obvious difference of numerical strain results was found between different reinforcements in the same layer, so average values for both upper and lower layers were given separately to compare with the test results. The strain increased linearly and slowly before the concrete slab cracking.

Table 2 Loading capacity of the test specimens.

Specimen	Initial cracking moment(kN·m)			Crack closure moment (kN·m)	Ultimate bending moment (kN·m)		
	$M_{c,e}$	$M_{c,t}$	$M_{c,f}$		$M_{u,e}$	$M_{u,t}$	$M_{u,f}$
CBS	220	247	260	3,500	3,801	3,104	3,720
CBP	120	247	224	3,203	3,999	3,104	3,900

Note: $M_{c,e}$, $M_{c,t}$ and $M_{c,f}$ = Initial cracking bending moment from experiments, theoretical calculation and numerical values; $M_{c,c}$ = Crack closure bending moment from experiments; $M_{u,e}$, $M_{u,t}$ and $M_{u,f}$ = ultimate bending moment from experiments, theoretical calculation and numerical calculation, respectively.

After the initial cracking, the strain near the point where the crack has occurred increased rapidly (often referred to as strain jump). Similar phenomenon was also confirmed in the numerical studies, shown in Figs. 9a and 10a. In regard to the initial cracking moment, results in Table 2 indicate that the numerical results agree well with the theoretical values, but larger than the test values. It demonstrates that for continuous composite steel and concrete structures, the concrete slab will crack and thus making the structural behavior highly nonlinear even for low stress levels (smaller than theoretical predictions).

Until the theoretical girder yielding load of the specimen, the reinforcements were continuously deformed and the strain of upper layer reinforcements was found larger than those of lower reinforcement. Comparison between numerical and theoretical results

indicates that numerical strain results of reinforcements had a similar variation trend but a little bit smaller than test values, which might be caused by the initial cracks in the concrete slab that can not be considered in the numerical simulation. Beside, as the concrete in tension zone are usually suggested to be ignored for ultimate strength calculation, resulting in

Fig. 8 Position of reinforcement strain gauges.

Fig. 9 Strain development of reinforcing bars of CBS.

Fig. 10 Strain development of reinforcing bars of CBP.

theoretical yielding loads is smaller than the test values, as shown in Figs. 9b and 10b. The tension stiffening effect between cracks should be considered in order to evaluate the more exact flexural stiffness of the composite section under negative bending moments.

Furthermore, as perfect bond was assumed between reinforcing bars and surrounded concrete, resulting in smaller strain during the loading process and deferred re-bar yielding in the numerical analysis. Also, strain hardening of the reinforcements was confirmed in the experiment for some reinforcements. However, for some other reinforcements, the strain was declined before strain hardening was obtained, which could be caused by the bond effect failure between the concrete and the reinforcement. Therefore, it seems that the strain hardening effect of reinforcement for composite girders in negative bending moment region can be ignored.

4.3 Crack Formation and Development on Concrete Slab

Initial crack formation and crack width development during the loading process were recorded by using 14 π -gauges on the surface of the concrete slab. Numbers, locations as well as measurement of π -gauges in the experiments were shown in Fig. 11. Crack width calculated by considering the maximum crack spacing (taken as 217 mm according to standard specifications for steel and composite structures [19]) and re-bars strains from

numerical models was illustrated as numerical results, shown in Figs. 12 and 13, in which the average experimental values of crack width were also given to make a comparison.

The crack width jump shown in Fig. 12a indicates that the initial cracks occurred at the top of concrete slab near loading position when applied load reached to 220 kN, which is a little bit smaller than the numerical and theoretical results, as shown. Fig. 12b also demonstrates that the proposed numerical model can simulate the un-cracked section well and give similar results to experimental values. However, after initial cracking, experimental cracking width becomes larger than numerical values. The results shown in Fig. 12b demonstrates that before re-bar yielding, the assumption of perfect bond between re-bars and surrounded concrete will not affect on crack width and the numerical simulation has similar results in comparison with test values. Also, the results indicate that the smeared crack model that usually used in commercial software is capable for simulating the crack development process. However, after re-bar yielding, non-ignorable slip is produced between re-bars and concrete, resulting in larger difference between numerical and experimental results.

In addition, an interesting phenomenon of “crack closure” was observed in the test for both CBS and CBP, and the crack width was found suddenly to become small and to keep constant for all large cracks before the ultimate load was reached, as shown in

Fig. 11 Arrangement of π -gauges (100 mm square meshes on the concrete slab).

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Fig. 12 Crack development process of CBS.

Fig. 13 Crack development process of CBP.

Figs. 12b and 13b. This might be because the sectional through crack was generated and the bond effect between the concrete slab and the steel girder was failure, thus the concrete slab lost its loading capacity and the crack width just remains stable. Due to the strain hardening of the steel girder, the composite girder could sustain more loads after the “crack closure” was observed. However, as the concrete had through cracks, the girder stress as well as its deformation increased rapidly until the ultimate load was reached. Nevertheless, similar phenomenon can not be found in the numerical analysis, which was due to perfect bond assumption between reinforcing bars and surrounding concrete.

5. Conclusions

The results indicate that both PBL and stud connectors are effective shear connective devices for

composite girders subjected to negative bending moment, and no obvious differences were found except that PBL connectors could slightly improve the rigidity of the composite girder in both the serviceability limit state and the ultimate limit state in comparison with stud connectors. Stud specimens have relatively better mechanical behavior in regard to concrete slab cracking behavior, such as initial cracking and crack closure. Besides, the proposed numerical models can simulate the test specimens and can be served as a basis for the design of composite bridges under negative bending moment. Perfect bond assumption for concrete-reinforcement interface was proved suitable before re-bar yielding.

Acknowledgments

The authors would like to thank to Ministry of Land, Infrastructure, Transport and Tourism of Japan for the

financial support for the research.

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