MECHANICAL BEHAVIOUR OF COMPOSITE GIRDERS SUBJECTED TO HOGGING MOMENT: EXPERIMENTAL STUDY

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In continuous composite steel-concrete structures, cracking of the concrete slab in the hogging moment region decreases the global stiffness of composite steel-concrete structures and reduces the effect of continuity, thus making the structural behaviour highly nonlinear even for low stress levels, so special consideration is necessary. An experimental investigation on the behaviour of steel-concrete composite beams in hogging moment region is presented in this paper. A total of four specimens were tested under point load in the mid-span. Two of the composite beams with headed studs as the shear connectors, while the other two specimens are using Perfo-Bond Strips (PBLs) as the connection devices between the steel girder and the concrete slab. Ultimate load bearing capacity of composite sections in negative moment region was calculated and compared with experimental values. Crack formation, crack widths development process and strain distribution of the composite section before and after cracking were observed and presented in this study. Interface slip distribution was also given. Research results indicate that the current specifications such as AASHTO, JSCE, and EUROCODE-4 can provide appropriate values for ultimate strength of a composite girder under negative bending moment.

Key Words : *hogging moment, composite girder, experimental study, shear studs, PBL*

1. INTRODUCTION

Composite steel and concrete structures have been extensively used in civil engineering structures, such as buildings and bridge structures due to the benefits of combining the two construction materials, their higher span to depth ratio, reduced deflections and higher stiffness ratio than traditional steel or concrete beam structures. Much existing research was carried out to determine positive moment carrying capacity of the composite beams. For simply supported composite beams, the ultimate loading capacity is usually governed by either positive bending moment or shear force bearing capacity, which will be predominated by the compressive strength of the concrete and tensile strength of the joist steel. However, for composite beams in the negative moment regions, this condition may not be fully satisfied. The concrete slab is usually in tension and the lower flange of the steel girder is in compression, which generally has shortcomings in view of the durability,

loading capacity and service life of the structures.

Ultimate strength is one of the most important structural behaviors for composite structures subjected to negative bending moment, but only a limited number of experimental or analytical studies are available on this important design aspect. Lee et al.¹⁾ investigated the negative moment behavior of cold-formed steel deck and concrete composite slabs. The experimental results indicate that the steel deck contributes to the hogging moment capacity and this region exhibited a fair amount of ductility, which can be utilized for moment redistribution. Nakamoto et al.²⁾ performed an experimental study on cracking behaviour of composite girders under hogging moment. Loh et al.³⁾ developed an iterative-based analytical model to study the behavior of partial shear connection composite beams subjected to hogging bending. Lääne & Labet⁴⁾ presented a study on the behavior of slender composite plate girders under negative bending moment by using a numerical model that specifically developed for this

purpose. A simple analytical model for quantifying the slender composite beam support region ductility was proposed and verified on the basis of the parametric analysis results. Kamiya et al.^{5), 6), 7)} presented some experimental results of fatigue behaviour as well as the stress, strain and the crack distribution of composite beams under negative bending moment, and tension stiffening of concrete on stress in reinforcements was suggested to be taken into account. Ryu et al.⁸⁾ performed an experiment on a two-span continuous composite bridge to study the crack control methods in negative moment regions. Their research results show that the crack widths can be controlled appropriately within an allowable crack width in the decks and transverse joints of the composite bridges with prefabricated slabs on an interior support under service loads. Chen et al.^{9), 10)} performed a comparative experimental study of prestressed continuous steel-concrete composite beams. It was found that in hogging moment regions, the ultimate resistance of an externally prestressed composite beam would be governed by either distortional lateral buckling or local buckling, or interactive mode of these two buckling patterns. Nguyen et al.¹¹⁾ presented a numerical model based on a nonlinear mixed finite element formulation for predicting the behavior of continuous composite beams with discrete shear connectors. Nie et al.¹²⁾ presents a mechanical model based on elastic theory to investigate the stiffness of composite beams in negative bending regions by considering slips at the steel-concrete interface. And the results show that the slip effect reduces the section stiffness by 10–20% compared with that of a beam without any slip in serviceability condition. He et al.¹³⁾ conducted an experimental study on the composite girders under negative moments, and four composite beams with different types of shear connectors were used in the study.

But all in all, the behavior of composite structures under negative moment at ultimate strength has not been investigated enough because only a few publications are available, thus the failure mechanism of composite girders under negative moment needs to be defined and the failure tests of model girders are necessary. This paper presents the experimental results of composite steel-concrete beams under negative moment. The objective of the present study is to investigate the failure modes, strain distribution at salient sections along the span, load-carrying ca-

capacity of composite beams in negative moment region, and to examine the effects of different kind of shear connectors on the inelastic behavior of such girders. The specimens were tested under a concentrated load applied at mid-span. Experiments have been carried out on the test beams and the results are compared with those of code specified values.

2. EXPERIMENTAL PROGRAMME

(1) Details of test specimens

Four simply supported composite steel-concrete beams CBS-1, 2 and CBP-1, 2 were designed as overturned (**Fig.1 (e) and (f)**) and tested under point load in the mid-span, as summarized in **Table 1**. CBS-1 and 2 were designed with studs as shear connectors whilst CBP-1 and 2 were designed with PBLs as shear connection devices. Each of the specimens was 4600mm in length and was simply supported at a span of 4000mm. The concrete slab thickness was 250mm with a width of 800mm. Point load was applied at the middle span. Different shear connectors, including 22 mm nominal diameter headed shear studs and PBLs were used to connect the concrete slab and the steel girder, and the number of shear connectors were designed according to loading capacities of test specimens¹⁴⁾ and JSCE specifications for steel and composite structures¹⁵⁾. The typical geometry of test specimen is shown in **Fig.1**. For the sake of simplicity, upper flange, web and lower flange are used to name different portions of the steel girder in this paper, as shown in **Fig.1 (b)**. The specimens were manufactured and tested twice. CBS-1 and CBP-1 belong to group-1, whilst CBS-2 and CBP-2 belong to group-2.

The rubber-latex mortar was usually used for steel structures to reduce the interface slip as well as the noise, to enhance the integrity and to improve the durability of the composite structures. It was also reported to have the good impact resistance, anti-corrosion resistance as well as the good constructability¹⁶⁾. In order to study the effect of rubber-latex mortar coating on structural behavior of composite girders under negative bending moment, rubber-latex mortar is sprayed on specimens of group-2, with details listed in **Table 2**.

Table 1 Sectional properties of test specimen.

Group No	Specimen	Connection device	Shear connectors spacing(mm)		Reinforcing bars spacing(mm)		Rubber latex
			Longitudinal	Transverse	Longitudinal	Transverse	
Group-1	CBS-1	Stud	150	100	100	150	No
	CBP-1	PBL	150	240	100	150	No
Group-2	CBS-2	Stud	150	100	100	150	Yes
	CBP-2	PBL	150	240	100	150	Yes

Fig.1 Dimensions of test specimen.

Table 2 Rubber-latex spraying details of Group-2.

Details	Spraying thickness (mm)		Spraying times
Locations	CBS-2	CBP-2	
Top surface of upper flange	7.7	6.6	2
Low surface of upper flange	3.0	4.8	2
Web	6.7	6.4	2
Shear connector	2.1	6.7	2
Top surface of lower flange	3.0	4.8	2
Low surface of lower flange	5.0	7.0	3

(2) Instrumentation

With the purpose of determining the neutral axis, strain gauges were used to measure the strain in the web, steel flanges as well as the concrete slab surface in key sections of test specimens. 12 linear variable displacement transducers (LVDTs) were used to measure the vertical deformation at the mid-span and support locations. Another 14 linear variable displacement transducers were employed along the

longitudinal direction to measure the slip on the interface between the concrete slab and the steel girder. Strain gauges used to measure the strain of the reinforcing bars and π -gauges for measuring the crack width on the concrete slab were also used in the tests.

(3) Specimens test set-up and loading procedure

The experiment was performed in the Structural Laboratory of Waseda University. The 5000kN loading capacity equipment of “Two Axes Large-scale Apparatus used for Performance Evaluation of Structures” was used in the experiment to apply a point load in the mid-span of the overturned composite beam. The test specimen was supported by a roller system at two ends. Shear connectors during concrete casting, rubber-latex spraying and test specimen set-up are illustrated in **Fig. 2**. After the drying shrinkage had stabilized, pre-loading was applied to check the reliability of the measuring equipments and the stability of the test specimens. The negative bending load was applied by static loading with unloading process at the levels 200,

(a) Stud and PBL shear connectors during construction

(b) Gum-latex spraying

(c) Test specimen set-up

Fig.2 Shear connectors during construction, gum-latex spraying and schematic of test setup.

400, 700 and 1300kN with loading rates of 0.001mm/s, 0.0015mm/s, 0.002mm/s, and 0.003mm/s, respectively. Displacement control was used in the tests with a loading rate of 0.004mm/s for the subsequent experiment. For unloading process in the experiments, the load was removed at the calculated initial cracking load (200 kN) and at the stationary cracking load (700 kN) to check the cracking of the concrete slab¹⁷⁾. The loading was terminated when either the maximum stroke of the jack was reached or the load capacity of the test specimen dropped significantly.

(4) Material properties

a) Concrete

Concrete cylinders of 10cm (diameter) × 20cm (height) were prepared for splitting and compressive strength tests during casting of the concrete slab for each specimen. The nominal concrete compressive strengths and tensile strength achieved after twenty-eight days of curing for CBS-1, CBS-2, CBP-1 and CBP-2 was (36.0, 2.51), (27.4, 2.09), (35.5, 2.50), and (29.1, 2.18) N/mm², respectively. Mix proportions of concrete were given in **Table 3**.

b) Structural steels

Three steel test coupons were cut out from each

component of the steel girder, including the upper flange, web, lower flange and PBL on basis of their different thickness. A universal testing machine was used to undertake the tensile tests of the steel plate. The results of yield strength, tensile strength and Young's modules were illustrated in **Table 4**. CBS-1& CBP-1 and CBS-2& CBP-2 were manufactured as two groups, so different tensile testing results for each group were given.

c) Reinforcing bars

Fourteen reinforcing bars of D19 nominal diameter were arranged into 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. Tensile tests were carried out to measure the property of the reinforcing bars and the results were presented in **Table 5**.

d) Gum-latex

Styrene-Butadiene or Styrene-Butadiene-Rubber (SBR) is a synthetic rubber copolymer consisting of styrene and butadiene. It has good abrasion resistance and good aging stability when protected by additives. The gum-latex mixing with SBR was applied for Group-2 specimens in these experiments, and its material properties were illustrated in **Table 6**.

Table 3 Mix proportion of concrete.

Group No	Design strength (N/mm ²)	Aggregate size (mm)	Slump (mm)	W/C (%)	Air (%)	Unit cement (kg/m ³)	Unit salinity (kg/m ³)
Group-1	27	<20	80±25	<55	4.5±1.0	>230	<0.3
Group-2	27	<20	80±25	<55	4.5±1.5	>230	<0.3

Table 4 Material property of structural steel.

Specimen No	Component	Thickness (mm)	Yield strength (MPa)	Tensile strength (MPa)	Young's modulus (MPa)
CBS-1&CBP-1	Upper flange	16	394.8	532.4	2.06×10 ⁵
	Web	22	372.4	527.0	2.07×10 ⁵
	Lower flange	25	349.5	521.5	2.06×10 ⁵
	PBL	12	396.9	537.5	2.04×10 ⁵
CBS-2&CBP-2	Upper flange	16	404.8	528.4	2.01×10 ⁵
	Web	22	364.9	515.9	1.99×10 ⁵
	Lower flange	25	382.2	517.6	1.97×10 ⁵
	PBL	12	416.6	535.0	2.05×10 ⁵

Table 5 Tensile test results of reinforcing bars.

Specimen	Diameter (mm)	Yield strength (MPa)	Tensile strength (MPa)	Young's modulus (MPa)
CBS-1 & CBP-1	13	N.P.	N.P.	N.P.
	19	386.6	558.2	2.02×10 ⁵
CBS-2 & CBP-2	13	376.2	540.1	1.86×10 ⁵
	19	388.7	559.1	1.91×10 ⁵

Note: N.P. = not performed in the experiment.

Table 6 Material property of gum-latex.

Appearance	oyster white
Solid content	45.0±1.0%
PH	9±1
Viscosity (20℃)	<200mPa·s
Mean specific gravity	1.00±0.05
Compressive strength	32.3N/mm ²
Elastic modulus	21kN/mm ²

3. TEST RESULTS AND DISCUSSION

(1) Strength and deformation response

The relationships between load and displacement at mid-span of the girder for each specimen during the loading procedure are shown as **Fig. 3**. After the loading test, it was observed that the crack occurred throughout the slab, and the detailed crack distribution of test specimens will be described in the following section. Comparison on load-deflection relationship between specimens in Group-1 and Group-2 shows that the relationships between load and displacement are almost the same in the primary stage (< 2000kN), but the displacement of specimens in Group-2 becomes smaller than that of specimens in Group-1 under the same load until the ultimate state. This is presumably because the elastic modulus of concrete of specimens in Group-1 is larger than that of Group-2. In addition, comparison of specimens in

each group indicates that the displacements of CBP-1 and CBP-2 become a little less at around 1000kN, this is presumably because of the PBL dowels affects the rigidity of the entire girder when cracking has progressed to a certain extent.

Table 7 showed the results of the load capacity of the test specimens under initial cracking level and the ultimate state respectively. Experimental tensile strength of concrete is used for calculating the theoretical initial cracking load, and the effect of gum-latex was not considered. 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 273 kN for group-1 and 247kN for group-2, were larger than experimental values of test specimens. It seems to indicate that the initial cracking load is very small for composite beams under negative bending moment and thus making the structural behaviour highly nonlinear even for low stress levels. The comparison between the ultimate loads predicted experimentally and the code determined theoretical values are also given. The ultimate pure bending moments, $M_{u,e}$ 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 (2007)¹⁸⁾, which is almost the same with that specified in JSCE (2007)¹⁵⁾, Eurocode-4¹⁹⁾ as well as the GB50017(2003)²⁰⁾. It is found that in all cases the theoretical prediction underestimates

Table 7 Loading capacity of the test specimens.

Specimen No	$F_{u,e}$ (kN)	$D_{u,e}$ (mm)	Initial cracking bending moment (kN.m)			Ultimate bending moment (kN.m)		
			$M_{c,e}$	$M_{c,t}$	$M_{c,e}/M_{c,t}$	$M_{u,e}$	$M_{u,t}$ (kN/m)	$M_{u,e}/M_{u,t}$
CBS-1	3980.8	38.38	200	273	0.81	3980.8	3006	1.32
CBS-2	3801.4	41.67	140	247	0.57	3801.4	3104	1.22
CBP-1	4127.8	33.30	120	273	0.44	4127.8	3006	1.37
CBP-2	3998.6	40.43	120	247	0.48	3998.6	3104	1.28

Note: $F_{u,e}$ and $D_{u,e}$ =Ultimate load and corresponding ultimate displacement in the experiments; $M_{c,e}$ and $M_{c,t}$ = Initial cracking bending moment from experiments and theoretical calculation; $M_{u,e}$ and $M_{u,t}$ = ultimate bending moment from experiments and theoretical calculation, respectively.

Fig.3 Load-deflection relationship. **Fig.4** Strain gauge arrangement at mid-span (note: b and d layers were only applied in CBS-1 and CBP-1).

the test loading capacity, and the minimum deviation is about 22% for composite girder with studs as the connectors, and 28% for that using PBL. This could be caused by the neglect of the strain hardening effects of structural steels and reinforcement, as yield stress of structural steel and reinforcement was used in design codes when predicting the ultimate bending resistance of composite girders under hogging moment. In addition, no local buckling of steel girder was observed during the loading process in the experiments, thus the test specimens could be bending dominated failure and can be assumed as the perfect compact section until the ultimate load.

(2) Composite neutral axes

Sectional strains were measured from the longitudinal reinforcements and steel girder portions, and the arrangement of strain gauges during the loading process was shown in **Fig. 4**. **Figs. 5 to 8** illustrate the mid-span elastic linear and the plastic nonlinear strain distributions as well as the movement of the sectional neutral axis for CBS-1, CBP-1, CBS-2 and CBP-2, respectively. Elastic neutral axis (ENA) of un-cracked section and plastic neutral axis (PNA) of cracked section according to AASHTO LRFD (2007)¹⁸⁾ that with the reinforcing bars considered (PNA-1) and not considered (steel girder only, PNA-2) in the ultimate state were given to make a comparison with the movement of the composite

neutral axis of each test specimen.

It is observed that the strain distribution was linear for composite steel-concrete beams during the early stage of the tests, as shown in the figures. However, with the load increase, the strain distribution of the reinforcement has exhibited a different curvature compared with the strain distribution of the steel beam. Due to the presence of interface slip and the cracking of the concrete slab, the assumption that the plane section is plane is not satisfied and the strain distribution difference could be observed between different portions of steel girder, concrete and the reinforcing bars. Comparison also indicates that the composite neutral axes of group-2 specimens are always higher than those of the group-1 specimens, which is believed to be caused by the slip limitation of the rubber-latex mortar that sprayed for CBS-2 and CBP-2²¹⁾.

Furthermore, the movement of composite neutral axis seems can be divided into three stages, as shown in **Fig.9**. Firstly, when sectional bending moment is smaller than the cracking moment, the composite neutral axis is almost the same with the calculated elastic neutral axis. However, the test results show that the first stage is relatively small for composite girders subjected to negative bending moment. This is presumably because of the negative bending moment as well as the initial cracks in the test specimens, which will cause the structural behaviour of

Fig.5 Cross-sectional strains distribution and movement of neutral axis at mid-span of CBS-1.

Fig.6 Cross-sectional strains distribution and movement of neutral axis at mid-span of CBP-1.

Fig.7 Cross-sectional strains distribution and movement of neutral axis at mid-span of CBS-2.

Fig.8 Cross-sectional strains distribution and movement of neutral axis at mid-span of CBP-2.

Fig.9 Movement of neutral axis and crack development.

composite girders highly nonlinear and the movement of composite neutral axis even when the load is relatively small. With the load increase, the composite neutral axis change expeditiously and move towards the plastic neutral axis, which might be caused by the development of cracks in the concrete slab and can be treated as the second stage. In the third stage, with the load increase the cracks on the concrete slab become stable and the cross-sectional stress of composite girders will mainly sustained by the reinforcing bars and the steel girder, and the composite neutral axis begin to keep as an constant at around the plastic neutral axis. Additionally, the results indicate that the plastic neutral axis always higher than the results that only considering the steel girder (PNA-2), which means the composite section of the negative moment region was not an entirely cracked section, but some section could be effective due to the tension stiffening of the concrete.

(3) Crack formation and distribution

Initial crack formation, crack widths development and distribution during the loading process were recorded by using 14 π -gauges on the top surface of the concrete slab. Numbers and locations in measurement of π -gauges in the experiments were shown in **Fig.10** and **Fig.11**.

As the behavior of crack occurrence and crack width development with the load increase was similar to each other between different test specimens, so results of CBS-1 was presented as an example. As mentioned earlier, loading, unloading and reloading process were applied in this experiment, thus, recorded results during the loading and reloading process of 0-400kN, 0-700kN, 0-1300kN and final reloading process until the ultimate load were presented in **Fig.12**. **Fig.12 (a)** indicates that an initial cracking occurred at the top of concrete slab near loading position when load is about 200kN. The cracking load was slightly smaller than the calculated value. With increasing of the load, the cracks propagated and distributed. At a crack, the re-bar strain is maximum and the concrete strain is zero. In between cracks, the re-bar strain is minimum and the concrete strain is maximum. If, under increasing load, the concrete strain reaches the limiting tensile

Fig.10 Arrangement of π -gauges (100mm square meshes on the surface of the concrete slab).

Fig.11 Arrangement of π gauges in the experiments.

Table 8 Comparison of “Crack closure”load and ultimate load.

Specimens	CBS-1	CBP-1	CBS-2	CBP-2
$F_{c,c}$ (kN)	3824	3440	3500	3203
$F_{u,t}$ (kN)	3006	3006	3104	3104
$F_{u,e}$ (kN)	3981	4128	3801	3999

Note: $F_{c,c}$ = “crack closure” load from the test; $F_{u,t}$ = Theoretical ultimate load; $F_{u,e}$ = Experimental ultimate load.

strain, an intermediate crack forms between two previously formed cracks. It is observed that when new crack occurs, there is always some small “crack jump” for existing cracks. When initial cracking (for example, when load is 200kN in **Fig.12(a)**) or new cracks occurs between existing cracks, the crack width jumps up or down suddenly, which is referred to as crack jump herein. In the crack formation of composite girders, there are three stages before yielding of the composite section: Firstly, an un-cracked section stage before cracking; secondly, cracks developed after cracking; and lastly, a crack stabilizing stage continued until the beam yielding.

In addition, a very interesting phenomenon of “crack closure” was observed in the test, and the crack width was found suddenly to become small and keep as constant for all large cracks before the ultimate load was reached, shown as **Fig.12 (d)** and the details was illustrated in **Fig.12 (e)**. Similar phenomenon was also observed in the tests of CBP-1, CBS-2 and CBP-2. This might be because the sectional through crack was generated and the bond effect between the concrete slab and the steel girder

Fig.12 Crack formation and development in loading and reloading process of CBS-1.

was out of service, thus the concrete slab lost its loading capacity and the crack width does not change anymore but remain stable due to the perfect slip on the composite interface.

Due to the strain hardening of the steel girder, the composite girder could continue to sustain more loads after the “crack closure” was observed. However, as the concrete was out of service, the girder stress as well as its deformation increased rapidly until the ultimate load was reached. This phenomenon was observed in the experiments of the entire test specimens, therefore it comes to the conclusion that the lost of composite action in composite structures can be experimentally observed as “crack closure”. Furthermore, as this experimental behavior is similar to those assumptions for the calculation of

ultimate load in current specifications such as AASHTO, JSCE, Eurocode-4 and GB-2003, thus “crack closure” load of each test specimen was presented and made a comparison with theoretically and experimental ultimate load, listed in **Table 8**. The results indicate that the “crack closure” load is always slightly larger than the code specified ultimate strength but smaller than the experimentally ultimate loads, which seems to indicate that the current codes for ultimate loading capacity of composite girder under negative bending moment is suitable for practical design because “crack closure” always happen after the theoretically loading capacity was reached. Moreover, the results indicate that the “crack closure” loads of PBL specimens are smaller than those of STUD specimens.

Fig.13 Cracks distribution on the surface of concrete slab after loading test (Note: The horizontal coordinate represents the distance from the loading point, the grid in blue stands for the place of reinforcement, and the line colored in red shows the cracks.).

Table 9 Crack spacing of test specimens.

Test crack spacing(mm)		CEB-FIP crack spacing (mm)		JSCE maximum crack spacing L_{max} (mm)		
Maximum	Average	Maximum	Average	JSCE(2002) ²⁵⁾	JSCE(2007) ¹⁵⁾	MILT(2009) ²⁶⁾
200	150	265.9	177.3	240	217	204

Fig.14 Position of reinforcement strain gauges.

Fig.13 shows the cracks on the concrete slab surface of each specimen after the loading test. It was observed that in the ultimate stage, the cracks mainly distributed in a direction perpendicular to the bridge axis, which means the cracks were mainly dominated by bending moment. With the entire specimens, crack spacing ranges from 100 to 200mm, with an average value of 150mm, coinciding with the spacing between the reinforcement stirrups. Also, it was found that the maximum experimental crack spacing is almost twice the minimum, which is consistent with the theoretical findings²²⁾. Furthermore, no obvious difference about crack distribution was observed between specimens with studs and PBLs.

The test crack spacing was compared with maximum crack spacing specified by CEB-FIP^{23), 24)} and JSCE, summarized as **Table 9**. The comparison indicates that code specified values are slightly larger and can be regarded as appropriate values when compared with the test results.

(4) Strain results of the reinforcement

Strain gages for measuring reinforcing bars were arranged on longitudinal bars around the mid-span section, where the maximum tensile stress are probable occur for simply supported girders under concentrated load in the span center. Sudden increase of reinforcing bars strain was usually observed in gauges close to the large cracks in the concrete slab at the load level when these cracks occurred, which was often referred as strain jump and was used to judge the initial cracking of the concrete slab in this paper.

Results of CBS-1 was taken as an example and presented herein. **Fig.14** shows gauges' number (gauges 13, 15, 17 were arranged in the upper layer reinforcements, and gauges 43, 45, 47 were for the lower reinforcements) as well as their positions, and the load-strain relationships were depicted in **Fig.15**. The strain increased linearly and slowly before cracking. After the start of cracking, the strain near the point where the crack has occurred increased rapidly (often referred to as strain jump). Until the failure of the specimen, the reinforcement was continuously deformed and the tension stiffening effect of the reinforcements was confirmed 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. Taking the load-displacement curve shown in **Fig.3** into consideration, it seems to indicate that the strength increase after girder yielding was mainly contributed by the strain hardening of the steel girder but not reinforcements. Also, strain

(a) Loading process from 0 to 400kN

(b) Reloading process from 0 to ultimate load

Fig.15 Strain development of reinforcement in loading and reloading process of CBS-1.

Table 10 Re-bar yielding moment.

Specimens	CBS-1	CBP-1	CBS-2	CBP-2
M_{ve} (kN)	2922	2497	2953	2505

hardening effect of reinforcement for composite girders in negative bending moment region seems can be ignored due to the slip between the reinforcement and surrounded concrete.

Furthermore, re-bar yielding moments of test specimens were illustrated in **Table 10**. Results indicate that re-bar yielding moments of STUD specimens are much larger than those of PBL specimens. By considering the crack closure load shown in **Table 8**, it seems to indicate that the PBL specimens have much severer cracks on the concrete slab in the span center than those of STUD specimens, resulting in re-bar yielding load of PBL specimens much smaller than those of STUD specimens. Thus it comes to the conclusion that different types of shear connectors may affect on the cracking pattern of the concrete slab and re-bar yielding load.

(5) Interface slip distribution

Generally speaking, in typical steel and concrete composite structures that designed with complete shear connection, the behavior of shear connectors have a negligible effect on the flexural behavior of composite beams in elastic state because the full connection as well as the friction effects on the shear stiffness are considerably high. However, in the plastic stage the slip behavior of the shear connectors becomes very important because it will effect on cross-sectional stress and strain distribution in steel girder, concrete slab and reinforcing bars.

The interface slip between the concrete slab and the top steel flange was measured and presented in **Fig.16**. These measurements were taken along the longitudinal direction of the test composite beams, where measured sections were shown as **Fig.17**. 14 linear variable differential transducers (LVDTs) were used in the experiments to measure the slip on the interface. Arrangement of slip measurement is

shown in **Fig.18**.

He et al.¹³⁾ pointed out that the maximum interface slip occurred at around 1/4 span for simply supported girders under negative bending moment. The results from tests also showed this tendency, and similar distribution behavior of interface slip was observed for CBS-1, CBS-2 and CBP-2. It might be caused by two reasons. Firstly, an effect of the friction forces at beam ends by reaction frame restriction will limit the slip development on the interface. Secondly, cracking of the concrete slab influenced the slip distribution. For composite girders in positive bending moment region, concrete is always in compression and can be treated as the continuous materials, so experimental measurements of interface slip is mainly caused by the relative displacement between the concrete and steel girder. However, for composite girders under negative bending moment region, measurement results in the experiments will be also affected by the serious cracking of the concrete, which will impact the interface slip distribution. Moreover, the interface slip in the span center (section 4-4' in **Fig.17**) was also believed to be caused by the cracking of the concrete slab. However, by considering the results of CBP-1, interface slip distribution seems rough-and-tumble and it is hard to judge where the maximum slip may occur. The authors believe that the interface slip distribution for composite girders under negative bending moment was closely related to the crack pattern of the concrete slab, and more detailed studies are necessary on this aspect.

Additionally, average slip and maximum slip corresponding to AASTTO specified ultimate load of test specimens were generated from experiments and summarized in **Table 11**. As expected, the overall slip measurements from the composite steel-concrete beams with PBLs were much lower than those with studs. Under AASHTO specified ultimate load, the relative interface slip is 0.37mm and 0.22mm for PBLs compared with that of 0.06mm

Fig.16 Interface slip distribution of test specimens.

Fig.17 Locations of linear variable displacement transducers (LVDTs).

Fig.18 Arrangement of the linear variable differential transducers (LVDTs) in the test.

Table 11 Interface slip under theoretical ultimate load (mm).

Specimen	Average Slip	Maximum Slip
CBS-1	0.60	0.71
CBS-2	0.59	0.93
CBP-1	0.37	0.51
CBP-2	0.22	0.43

Note: Average slip= Average slip of measured sections except section 4, the slip of which is believed to be caused by the concrete cracking.

and 0.59mm for stud connectors. Besides, by comparing with the previous studies on the shear force-slip relationship of shear connectors (Ollgaard et al., 1971), most of the shear connectors were already in the plastic state.

4. CONCLUDING REMARKS

Detailed experimental investigations were carried out on composite girders subjected to hogging moment. Composite beams with different shear connectors such as shear studs and PBLs were loaded to the ultimate state. From the present results on the ultimate strength behaviors of test composite beams under negative bending moment, the following conclusions and recommendations deserving priority are made:

- (1) The present experimental results indicate both PBLs and Stud connectors are effective as shear connective devices. The PBL connectors could slightly improve the rigidity of the composite girder under both the serviceability limit state and the ultimate limit state in comparison with stud connectors according to the experimental load-displacement relationship. Stud specimens have relatively better mechanical behavior in regarding to initial cracking, girder yielding as well as the crack closure loads of test specimens.
- (2) The initial crack load calculated on the basis of the maximum negative moment of composite section was found larger than the test results, which means that composite beams under negative bending moment will crack and then make the structural behaviour highly nonlinear even in low load levels. Ultimate loads calculated were similar to test results, and current specifications based on the plastic analysis were proved appropriate for design on the safe side.
- (3) It is considered that the location of composite neutral axis is affected by the rub-latex and it moves between the cross-sectional elastic neutral axis and the plastics neutral axis. The location of the composite neutral axis was found to be the same as un-cracked section before cracking. After cracking, tension stiffening of the concrete is suggested to be considered when determining the location of the neutral axis.
- (4) Strain hardening effect of reinforcements was suggested to be ignored. The phenomenon of “crack closure” was observed for all test specimens, and thereafter the crack width was found keep as a constant. Additionally, average crack spacing on concrete slab was found

mainly dependent on the transverse reinforcement spacing, and the maximum crack spacing specified by current specifications was proved to be appropriate values when compared to the test results.

- (5) Slip distribution along the steel-concrete interface were given in this study, and the interface slip distribution was found closely related to crack pattern of the concrete slab for composite beam under hogging bending moment. Support conditions were considered to be another influencing factors.

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