FEASIBILITY STUDY ON CONCRETE-FILLED STEEL I-GIRDER BRIDGE

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1. INTRODUCTION

The steel I-girders are the simplest and the most commonly used structures on short to medium span bridges. For a simply supported bridge the composite girder consisting of the steel I-girder and the concrete slab is ideal. The lower part of the girder section is in tension which can be resisted by the steel girder and the upper part is in compression which is resisted by the RC slab. It is connected to the top of upper flanges with shear studs. For a continuous bridge the bending moment is positive at the span center and negative at the intermediate support. At the mid-span the steel/concrete composite girder is ideal: the steel and concrete sections resist tensile and compressive forces respectively. On the other hand, as large negative bending moments and shear forces exist at the intermediate supports, the concrete slab is in tension and does not contribute. As the lower flanges and lower parts of webs are in compression and are vulnerable to lateral-torsional buckling. Furthermore, the lower flange is wider and thicker and the web is also thicker and stiffened by vertical and horizontal stiffeners which increased the total cost of the bridge. Thus, the area around the intermediate support is the most critical part of the continuous steel girder bridges.

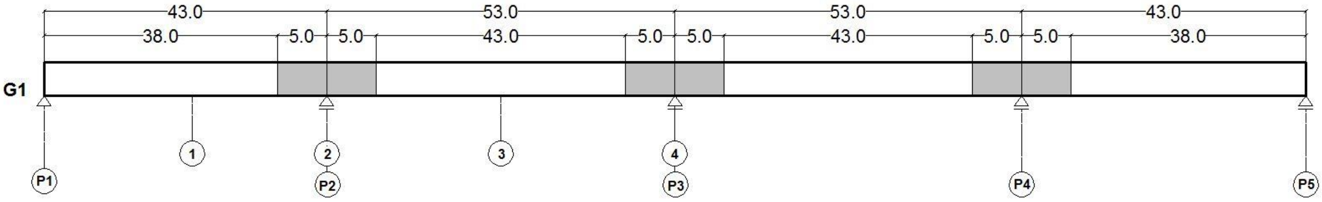

Fig. 1: Longitudinal view of Concrete filled I-Girder (CFIG) bridge (unit: m)

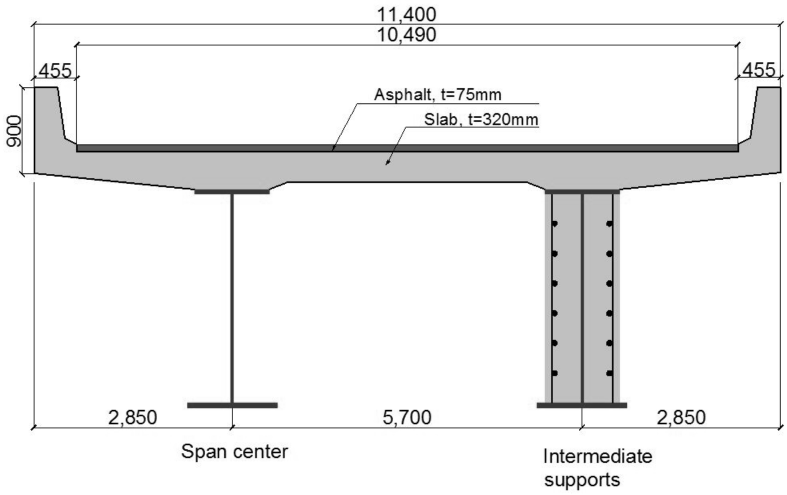

Fig. 2: Cross section of Concrete filled I-Girder (CFIG) bridge (unit: mm)

In order to improve and strengthen structural performance of continuous steel I-girder bridges under hogging bending moment at the intermediate support, the area surrounded by the flanges and the web is filled with concrete. This CFIG (concrete filled I-girder) was proposed for the intermediate support area (Nakamura, 2002). The filled concrete in the compression zone contributes to bending strength and also restricts the local buckling of lower flange and web, resulting in economical steel sections compared with the conventional continuous plate girder bridge (CCG). Furthermore, reinforcing bars are also welded to the upper and lower flanges (Fig.1, Fig.2) to restrict concrete falling down from the web and strengthen the web. This form was experimentally tested and proved by Nakamura (2003) which can be applied at the intermediate support of composite steel girder bridges. In this study, an analytical method was developed to calculate the bending strength for CFIG girder. Then, a trial design was carried out for the four-span continuous composite steel I-girder highway bridge. The result with the CFIG was compared with CCG, which confirmed advantages of CFIG. Basic structural behavior and strength of CFIG was also clarified by experiments.

2. TRIAL DESIGN

A trial design was carried out for the four-span continuous girder bridge. The bridge has 4 spans (43+2@53+43m) with11.4m width (Fig.1). The two types of girders were considered: Model I is a concrete filled steel I-girder (CFIG) and

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Model II is the conventional steel/concrete composite I-girder. The material and sectional properties of steel are assumed for this study is shown in Table 1. CCG is a conventional steel I-girder and CFIG is a concrete filled steel I-girder with steel reinforcements inside the filled concrete. This bridge model was chosen from Guidelines for performance verification of steel–concrete hybrid structures (Japanese Association of Civil Engineers, 2006). The cross section of the girder is shown in (Fig.2). The reinforcing bars with (16mm) in diameter were also vertically and horizontally at the interval of (300mm) and (355mm) are placed inside the web and flanges respectively.

2.1 Design Load

The design loads consist of pre-composite dead load (D1) due to the self-weight of girder, formworks and concrete slab and a post-composite dead load (D2) due to surface wearing, railings of the bridge and live load (L). The design live load consists of fully uniformly distributed load ($P1=3.5kN/m^2$) and equivalent concentrated load ($P2=10.0kN/m^2$) with the longitudinal width of 10m. These design live loads conform to the Japanese specification for highway bridges (Japan Road Association, 2014). Four critical cases for the live load (L_1, L_2, L_3 and L_4) are considered for both CFIG and CCG girders (Fig.3). L_1 which is applied at (19 m) from the first support produced maximum effects on the first mid-span, L_3 which is applied at the alternate spans of the girders produces a maximum effects on the second intermediate support and L_4 which is applied on the two intermediate spans of the girder applied maximum effects on the third support than L_1, L_2 and L_3.

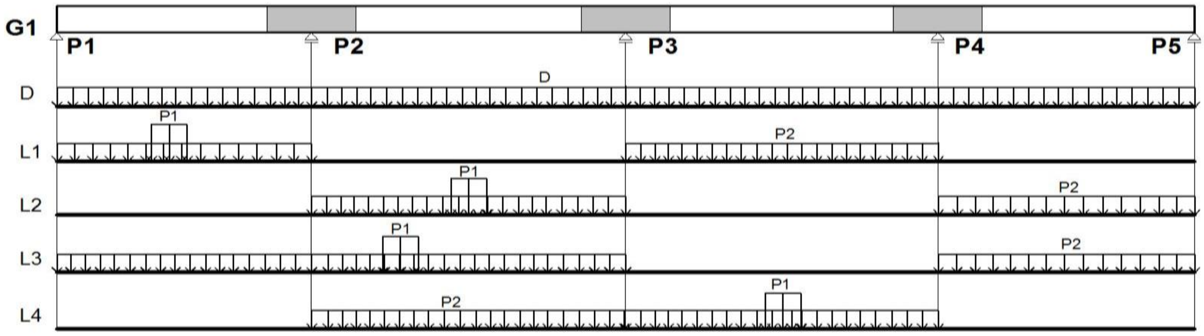

Fig. 3: Design load cases

2.2. Sectional Forces

Structural analysis was conducted for CFIG and CCG Models. Fig 4 shows the design bending moment due to dead load (D) and the maximum and minimum bending moment due to live loads (L_1-L_4) for CFIG Model. It is understood that the negative bending moment is critical at the intermediate supports and the positive bending moment is critical at the span centers.

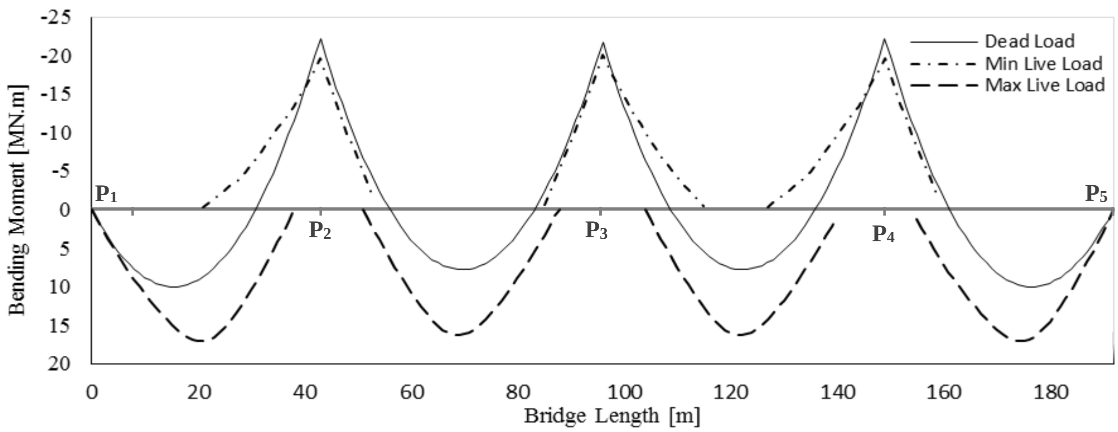

Fig. 4: Design Bending Moment

2.3 Safety Verification of CFIG and CCG Models

Table 1 shows the assumed cross-section of the girder at the mid-span and intermediate support. The SM490YB and SM570 steel grade is used at the mid-span and intermediate support of CFIG and CCG girders respectively.

Table 2 shows the safety verification for the pre-composite section of CFIG and CCG girders. The verification method conforms to “Guidelines for performance verification of steel–concrete hybrid structures” (JSCE, 2006). The design bending moment due to pre-composite dead loads (M_{d1}) is within the resisting capacity of the steel girder (M_{sud}).

Fig 5(a) shows the stress distribution of the section at the ultimate stage of CFIG at the intermediate support subjected to hogging bending moment and the neutral axis lies within the web. In addition, the compressive forces at the lower part is resisted by the steel girder, reinforcing bars and filled concrete within the web and flanges. The tensile force at the upper part is resisted by the reinforcing bars and the upper part of the steel girder. Fig.5 (b) shows the stress distribution at the span center subjected to sagging bending moment. The neutral axis lies within the concrete slab due to thinner thickness of concrete slab. The lower part below the neutral axis is tension which is resisted by the steel girder. The upper part is in compression which is resisted by RC slab.

Table 1: Sectional properties of CFIG & CCG models

Bridge girder system		*Concrete Filled I-girder(CFIG)*				*Conventional I-girder (CCG)*			
Section(mm)		*SEC 1*	*SEC 2*	*SEC 3*	*SEC 4*	*SEC 1*	*SEC 2*	*SEC 3*	*SEC 4*
Steel grade		*SM490YB*	*SM570*	*SM490YB*	*SM570*	*SM490YB*	*SM570*	*SM490YB*	*SM570*
Upper Flange	Width	400	500	400	500	500	700	500	700
	Thickness	18	25	17	25	19	35	18	35
Web	Height	2860	2845	2861	2843	2852	2816	2854	2822
	Thickness	14	15	14	17	13	16	13	22
Lower flange	Width	700	600	500	600	800	800	800	800
	Thickness	22	30	22	32	29	49	28	43
Cross sectional area (mm^2)		62,640	73,175	57,854	80,031	69,776	108,756	68,502	120,984
		(0.90)	(0.67)	(0.84)	(0.66)	(1.0)	(1.0)	(1.0)	(1.0)

Table 2 Safety check for the Pre-composite section of CFIG and CCG girders

Section		Design Bending Moment	Design Bending Moment Capacity		$(1.1M_{d1}/M_{sud})*\Upsilon_i \leq 1$	
		$1.1M_{d1}$	$M_{sud,t}$	$M_{sud,c}$	T-Side	C-Side
Section 1	CCG	6,185	22,918	7,065	0.30	0.96
Section 2	CCG	-14,439	24,624	-21,320	0.65	0.74
Section 3	CCG	5,761	21,983	6,772	0.29	0.94
Section 4	CCG	-14,650	-50,504	-43,726	0.32	0.37
Section 1	CFIG	7,057	17,979	7,778	0.43	1.00
Section 2	CFIG	-15,544	33,480	-24,643	0.51	0.69
Section 3	CFIG	5,487	15,341	6,676	0.39	0.90
Section 4	CFIG	-15,194	-39,050	-26,161	0.43	0.64

Table 3: Safety check for the post-composite section of CFIG and CCG girders

Section			Section 1	Section 2	Section 3	Section 4
Pre-Composite Dead Load	$1.1M_{d1}$	CCG girder	6,185	-14439	5,761	-14650
Post-Composite Dead load	$1.2M_{d2}$	CCG girder	3,164	-6,124	2,686	-6,200
Live load	1.98L	CCG girder	17,103	-17,660	17,452	-18,452
Design Bending Moment	M_d [kN.m]	CCG girder	26,452	-38,223	25,899	-39,302
Resistance Bending moment	M_{ud} [kN.m]	CCG girder	43,863	100,919	43,050	72,809
Pre-Composite Dead Load	$1.1M_{d1}$	CFIG girder	7,057	-15,544	5,487	-15,194
Post-Composite Dead load	$1.2M_{d2}$	CFIG girder	2,958	-6,690	2,285	-6,436
Live load	1.98L	CFIG girder	17,109	-19,602	16,327	-20,117
Design Bending Moment	M_d [kN.m]	CFIG girder	27,124	-41,836	24,098	-41,747
Resistance Bending moment	M_{ud} [kN.m]	CFIG girder	53,461	65,288	47,366	69,527
Υi			1.1			
$\Upsilon i*Md/Mud \leq 1.0$		CCG	0.66	0.42	0.66	0.59
		CFIG	0.56	0.70	0.56	0.66

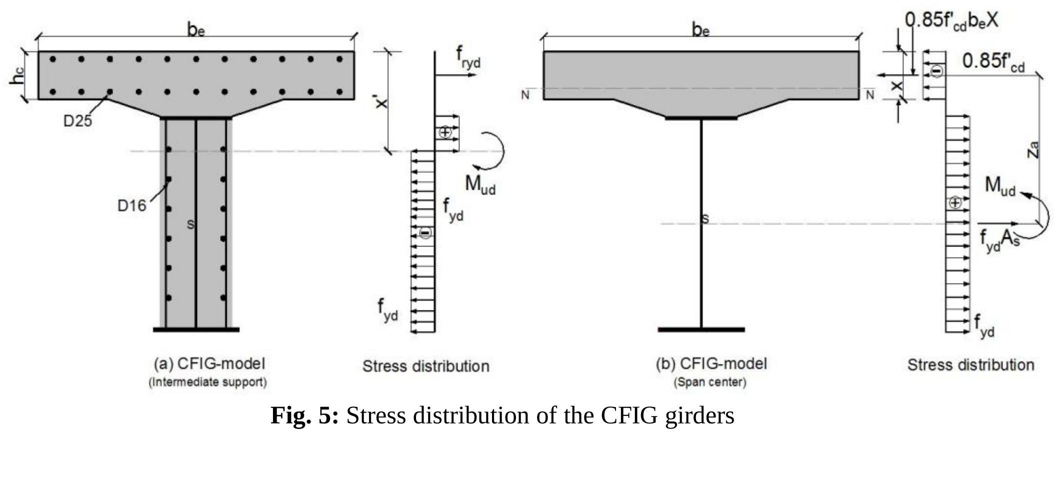

Fig. 5: Stress distribution of the CFIG girders

Table 3 shows the safety verification for the post-composite section. The load factor and the structure factor (Υ_i) conform to "Guidelines for performance verification of steel–concrete hybrid structures" (JSCE, 2006). The design bending moments (M_d) is within the resistance bending moment capacity (M_{ud}), which confirms that the assumed cross section is appropriate and safe. It is found from this trial design that the thickness of the flanges and web of CFIG can be less than 70% of CCG at the intermediate support. Also, those of CFIG can be less than 10% at the span center.

3. EXPERIMENTS OF THE PARTIAL CONCRETE-FILLED STEEL I-GIRDER MODEL

Bending tests were performed with two models (Fig.6). The model BS was the steel plate girder. The web was 900 mm high and 6 mm thick, and the flanges were 200 mm wide and 12 mm thick. The web was stiffened by intermediate vertical stiffeners at interval of 375 mm and the models were laterally restrained from lateral movements at the support. The BC model is the concrete encased girder. The steel reinforcing bars with 10 mm in diameter were places vertically at intervals of 200 mm and welded to the upper flanges and the horizontal reinforcing bars were also placed horizontally and connected with vertical bars by wires. The steel plate I-girder was placed flat and then poured with concrete to the one side after other by concrete hardened. This encased composite girder is expected to be not only useful for new girders but also useful for repair and rehabilitation of the damaged or old girders. The steel I-girders of these two models had the same dimensions and were fabricated from the same steel plate. It was found from the test samples that the yield and tensile strength of the steel plate was 372.3 and 511.4 MPa, respectively, and the compressive strength of the encased concrete was 55.0 MPa for BC. The specified yield strength of the deformed reinforcing bars is 290 MPa. The models were 3600 mm long and loaded at the two edge points of the pure bending moment zone with a length of 600 mm.

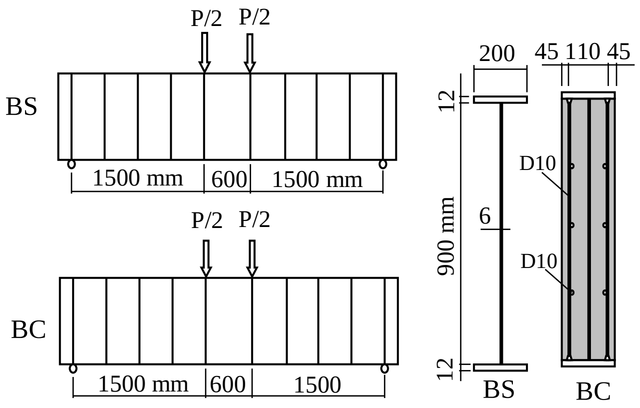

Fig. 6: Models for bending tests

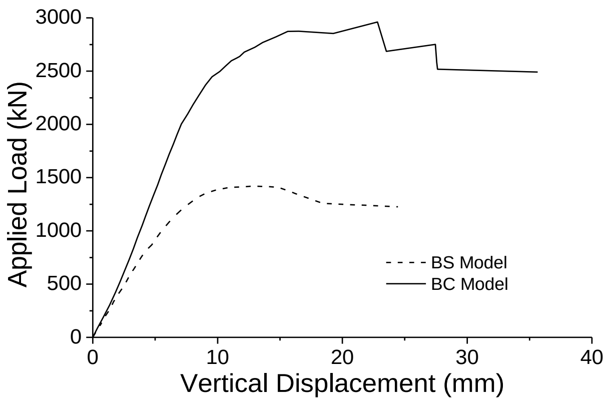

Fig. 7: Measured vertical displacement

Fig.7 shows the measured vertical displacements with the applied load at mid-span. The applied load first increased linearly, reached the maximum point and then collapsed when the web and upper flange buckled in the BS model. In the BC model the applied load increased sharper than BS and in the linear part and the relation became non-linear. However, the girder showed a good ductile property. It is noted that the maximum bending strength of the filled concrete model BC was 2.08 times the steel model BS.

4. CONCLUSIONS

A new steel/composite girder was proposed and applied to continuous girder bridge. The area surrounded by the flanges and the web is filled with concrete CFIG (concrete filled I-girder) at the intermediate support. A trial design was carried out for the four-span continuous highway bridge with span of 43+53+53+43m. The result with the CFIG was compared with CCG (conventional composite girder), which showed that the thickness of the flanges and web of CFIG can be less than 70% at the intermediate support and 10% at the mid span of the bridge of CCG. Bending tests were performed with CFIG and CCG. The maximum bending strength of the filled concrete model was 2.08 times the steel model.

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