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Experimental Study on Steel-Concrete Composite Twin I-Girder Bridges

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Abstract

A large number of steel-concrete composite twin I-girder bridges have been built in both Europe and Japan, but the lack of redundancy has always been a concern in the U.S. and many other countries. In addition, very few experimental studies have been performed on mechanical performance of such bridges, particularly for the intact twin I-girder bridges. On this background, a steel-concrete composite twin I-girder bridge specimen was designed according to the current highway bridge design specification in Japan. The static loading tests were performed, and two loading conditions including both symmetric loading and asymmetric loading were applied. Load versus deflection relationship and strain development on steel main girders and concrete slab at key sections were measured. The flexural strains on the lateral beam were also measured and reported in this paper to confirm the load transfer between two main girders. In addition, the shear strain of shear connectors (stud, in this study) was also measured to investigate the shear force transmission on the steel-concrete interface. The theoretical values were also provided to compare with the test results from the twin girder specimen under symmetric loading condition. The experimental results indicate that the theoretical analysis can predict the behavior of the twin-girders very well in the elastic stage by considering the effective width of the slab. Performance of each structural components and load transfer paths in such bridges were also discussed.

Key Words: Twin-girder bridges; steel-concrete composite; experimental study; theoretical analysis; intact case.

Introduction

Multi-girder bridges, as shown in Fig.1 (a), are one of the most common types of medium span composite bridge, which can be used for a single span or for continuous multiple spans, and it is particularly effective where

30 construction depth is limited. In multi-girder bridges, a number of similarly sized longitudinal plate girders are
31 arranged at uniform spacing across the width of the bridge. The deck slab spans transversely between the
32 longitudinal girders and cantilevers transversely outside the outer girders (Sarraf et al., 2013). The girders are braced
33 together at supports and at some intermediate positions. Composite action between the reinforced concrete deck
34 slab and the longitudinal girders is achieved by means of shear connectors welded on the top flanges of the steel
35 girders. As twin girder bridges are classified as non-redundant structures according AASHTO specifications (2012),
36 their construction was rather limited in U.S. and many other countries. Recently, however, the two-girder steel-
37 concrete composite bridges are widely used in practice in several other countries, like Japan. In comparison with
38 the traditional multi-girder bridges, twin-girder composite bridges have the following characteristics: the use of
39 prestressed deck and composite deck makes the larger space between main girders become possible; the resistance
40 of the deck for the transverse load makes the omission of the lateral bracing become possible; less main girders and
41 lateral beams make simplification of the bridge design and significant reduction of the construction cost become
42 possible.

43 The twin-girder bridges were widely built in Europe, such as the Rudolf von Habsburg Bridge (1971~) in
44 Germany, Jassans-Riottier Bridge (1971~), Monestier Viaduct, and the Triel-sur-Seine Bridge etc. in France, whose
45 main spans were longer than 100m. In Japan, the application of twin-girder bridges is of historical inevitability. In
46 the Showa period (1926-1989 CE), due to the lack of the construction materials (particularly the structural steel)
47 and their high cost, complex design of multi-girder bridges were popular because the structural steel can be
48 efficiently used and saved. When the history moved to Heisei period (1989–present), however, the situation had
49 changed. With the significant reduction of the number of highly skilled workers, the long construction time and high
50 labor cost involved to build multi-girder bridges became a severe problem (Inaba, N., 2011). On the contrary, the
51 material cost increased little in comparison with the labor cost. It is in this context that “minority main girder”
52 bridges with simple sections became urgent demand. In 1992, a trial bridge (Shinkotoni viaduct) was built in
53 Hokkaido, and the construction cost was greatly reduced by using two main girders with less cross bracings and
54 standardized connecting plate. The research achievements were reflected in “Design Guidelines for Steel Highway
55 Bridges (Draft)”, and the bridges with less main girders were recommended for practical use and named as
56 “rationalized” girder bridges thereafter. In 1995, another twin girder bridge (the Horonai River Bridge) was built in
57 Hokkaido. The girder distance was extended from often used value (2~3m) to around 6m, by using prestress in the
58 concrete slab. Thereafter, more and more twin girder bridges were built in Japanese highway system, including the
59 Warashinagawa Bridge (awarded the JSCE Tanaka Prize, 2002), which has two main girders with girder distance

of nearly 10m.

Recent studies on two-girder bridges mainly focused on the post-fracture redundancy and redundancy evaluation of twin-girder systems, such as those performed by Idriss et al. (1995), Tachibana et al. (2000), Park et al. (2012), Samaras et al. (2012), Kim and Williamson (2015), and Lin et al. (2013, 2016) etc. Limited studies on intact twin-girder bridges were also performed, mainly focusing on connection details of diaphragm, buckling of main girders, shear-lag effect in slabs etc. Takahashi et al. (1997) performed an experimental study to investigate the structural details of the connection of diaphragm used in the first steel twin-girder bridge "Horonai River Bridge". Four types of connection details, including the connection with triangular rib plate or connection plate, split-tee connection and endplate connection, were investigated. In 1999, Hotta et al. investigated both global and local lateral-torsional buckling of twin girder bridges during erection of main girders in the case of the incremental launching method. Dezi et al. (2006) performed an analysis to investigate the shear-lag effect in slabs of twin-girder steel-concrete composite decks due to the main prestressing techniques such as support settlements, bonded cables inside the concrete slab, and external slipping tendons. Kozy and Tunstall (2007) reported stability analysis results of a twin I-girder pedestrian bridge constructed in Minnesota, and they pointed out that the strength of non-composite twin-girder systems without lateral bracing can be controlled by the limit state of 'system' or 'global' buckling. Awall and Hayashikawa (2011) performed a parametric study on horizontally curved twin I-girder bridges, focusing on the effects of curvature on the impact factor of such bridges. Ma and Shi (2016) performed numerical analysis to investigate the parameters that affect the stability and capacities on twin-I girder systems. Parameters considered in this study include girder depths, flange width-to-thickness ratio, web depth-to-thickness ratio, number of stiffeners, and cross-beam spacing. However, experimental study on mechanical behavior of intact twin I-girder bridges was rarely performed. In order to promote the application of such bridges, fundamental studies on such structures become necessary.

On this background, studies on the mechanical behavior of twin-girder bridges were conducted. The experimental studies of twin I-girder bridges were divided into two parts: intact case and damaged case. In the intact case, the mechanical behavior of twin I-girder bridges under normal service condition is investigated to understand the serviceability, ductility and safety of such bridges. For the damaged case, mechanical behavior of such bridges under extreme conditions (member fracture etc.) was investigated, focusing on the structural redundancy and resilience. The experimental results of the intact specimen are reported in this paper.

Experimental Programme

91 ***Test Specimen***

92 A simply supported steel-concrete composite twin I-girder bridge specimen was designed according to Japanese
 93 bridge design specifications (JRA, 2012; JSCE, 2007). The specimen was designed as 4.4m in length and was
 94 simply supported at a span length of 4m, by considering the available space in the structural laboratory. The width-
 95 to-span ratio and depth-to-width ratio are taken similar to that of typical composite twin I-girder bridges used in
 96 Japan. Two I-girders with a height of 300mm were used as main girders, and the width of the concrete slab was
 97 determined as 1200mm. According to bridge design specification in Japan (JRA, 2012), the minimum thickness of
 98 concrete slab is 110mm. To also study the real behavior of studs and avoid the possible damage of the slab during
 99 transportation, the slab thickness was determined as 120mm. The influence of the deck thickness on general
 100 behavior of such structures was investigated based on the numerical analysis (Lam et al., 2017a and 2017b), and
 101 the results clearly indicate that thickness of the concrete slab as well as the concrete strength have no effect on the
 102 global behavior of such structures. Vertical stiffeners were welded at support points and other key sections at an
 103 interval of 400mm to prevent buckling failure and crippling of the web before flexural failure. Six cross beams (CB-
 104 1~6) were also used between the twin main girders for load redistribution. Vertical stiffeners on the web were also
 105 appropriately designed and arranged between cross beams. Four rows of shear stud connectors with the diameter of
 106 16mm were placed with a uniform spacing of 200mm in the longitudinal direction to ensure the full (or complete)
 107 connection between the steel girder and the concrete slab. The theoretical values of yield and ultimate bending
 108 moment capacity of the specimen considering the actual yield stress from the material tests are determined as
 109 676kN·m and 886kN·m, respectively. Size dimensions and design details of test specimen are shown in Fig.2.

110 ***Instrumentation***

111 The instrumentation of the loading test is shown in Fig.3. Linear variable differential transducers (LVDTs) were
 112 used to measure the vertical deformation at the span center section and support location at both ends. With the
 113 purpose of determining the neutral axis, strain gauges were used to measure the strain in the web, steel flanges, the
 114 reinforcement as well as the concrete slab surface in key sections. In addition, shear strain on stud connectors in
 115 both longitudinal and lateral directions were also measured.

116 ***Test Set-Up and Loading Procedure***

117 A testing machine with the loading capacity of 5000kN was used to apply the load in the test. The test specimen
 118 was supported by a roller system at two ends. Two loading conditions were used in this study. In the first stage,

concentrated load was applied symmetrically by placing a loading beam on the top of the concrete slab in the mid-span section, as shown in Fig.4 (a). Three loading plates in total, were used. Two of them were used between the loading beam and the concrete slab, located at right above the main girders, and the third loading plate were used on the middle of the loading beam. In this stage, the applied load was controlled less than 150kN (about 22% of the yield load of the twin girder) to avoid any possible damage of the test specimen. In the second stage, the load was applied asymmetrically on the right top of one main girder (G1), as shown in Fig.4 (b). The load was applied by static loading with unloading process. The loading will be terminated when either the maximum stroke of the jack was reached or the load level of the test specimen dropped significantly. In both symmetrical and asymmetrical loading conditions, pre-loading until 50kN was applied to check the reliability of the measuring equipment and the stability of the test specimens. Displacement control method was used in the loading process with loading rates of 0.004mm/s for the pre-loading and 0.008mm/s for following loading test. The test specimen was supported by a roller system at two ends. Set-up of the specimen in the experiment is shown in Fig.5.

131

132 ***Material properties***

Concrete cylinders of 10cm (diameter) ×20cm (height) were prepared for compressive tests during casting of the concrete slab. The concrete compressive strengths achieved after twenty-eight days of curing were 28.8, 29.5, 29.5MPa, respectively, with the average compressive strength of 29.3MPa. The structural steel of SM490 was used for the steel girder in this specimen. The web has a thickness of 6mm, and both top and bottom flanges have a thickness of 12mm. According to the tensile test results, the yield strength and ultimate strength of the structural steel for web were 450 and 577MPa, respectively. While, the yield strength and ultimate strength of structural steel for top and bottom flanges were 389 and 548MPa, respectively. The reinforcement of D10 nominal diameter was used for both longitudinal reinforcement and stirrup reinforcing bars in the concrete slab. According to the test results, the yield strength and ultimate strength of the reinforcement were 406 and 544MPa, respectively. The stud shear connectors were made of SS400, which has a nominal yield strength of 235MPa and an ultimate tensile strength of 400MPa, respectively.

144

145 **Test Results and Discussion**

146

147 ***Load-Deflection Response***

148 The load-vertical displacement curves of the test specimen under both symmetrical and asymmetrical loading were
 149 illustrated in Fig.6. The displacement was taken at mid-span section of the test specimen. Due to space limitations
 150 under the deck, the LVDTs were used on the bottom of the concrete slab but not the bottom of the main girders.
 151 The displacements on girder-1 (G1) and girder-2 (G2) are referred to as δ -1 and δ -2, respectively. For asymmetrical
 152 loading, the load was only applied on G1, as shown in Fig.4.

153 When subjected to a symmetrical load, the load-vertical displacement curves were shown in Fig.6 (a). It can be
 154 found that both δ -1 and δ -2 increase linearly with the increase of the applied load. Though there is small difference,
 155 the displacement of δ -1 was similar to that of δ -2, indicating that identical loads were applied on two main girders.
 156 In addition, theoretical value of the load-displacement curve of the specimen was also provided. In computing the
 157 theoretical value, all the materials were assumed as elastic and the effective width of the concrete slab was
 158 considered to account for the shear lag effects. The effective width of concrete slab in twin I-girder bridges was
 159 determined according to **Eq.(1)** according to the Specifications for Highway Bridges (JRA, 2012) in Japan. “Full
 160 connection” was assumed for the steel-concrete interface, and the transformed section method was used to determine
 161 the moment of inertia of the composite section. The theoretical values were then determined according to the classic
 162 theory for simply support girders. The comparison indicates that the theoretical results determined according to
 163 current elastic design theory agree well with the test results, demonstrating the twin I- girder bridges can be designed
 164 appropriately on the basis of the current bridge design method.

$$165 \quad \lambda = \begin{cases} b & \left(\left(\frac{b}{l} \right) \leq 0.05 \right) \\ \left\{ 1.1 - 2 \left(\frac{b}{l} \right) \right\} b & \left(0.05 < \left(\frac{b}{l} \right) < 0.30 \right) \\ 0.15l & \left(0.30 \leq \frac{b}{l} \right) \end{cases} \quad (1)$$

166 where λ : half effective width of flange (mm); b : half spacing between webs or projection width of cantilever portion
 167 (mm) (Fig.7); l : equivalent span length (mm).

168 When subjected to an asymmetric load, the load-vertical displacement relationships were shown in Fig.6 (b). In
 169 the initial loading stage, both δ -1 and δ -2 also increase linearly with the load increases. However, the δ -1 was much
 170 larger than that of δ -2, caused by the asymmetrical load applied on the side of the G1. Taking the applied load of
 171 300kN as an example, the corresponding displacement at G1 (7.884mm) was nearly 4.5 times of the displacement
 172 at G2 (1.764mm), thus the G2 carries around 18.3% ($=1.764/(1.764+7.884)$) of the load carried by the main girders.
 173 The results indicate that before crush or cracking, the concrete slab is capable of transferring a certain level of
 174 applied load from G1 to G2, but not zero as can be predicted according to the “lever principle”.

175 With the load increases to $P=387\text{kN}$, the yielding was confirmed on web-bottom flange junction of G1.

176 Thereafter, δ -1 increased much faster while δ -2 increased much slower as the increasing of the load. Even so, the
177 δ -2 kept increasing before it started to decrease when the load reached to 496kN. When the load beyond 496kN, δ -
178 1 kept increasing remarkably, while δ -2 kept decreasing and remained in the elastic stage. This phenomenon clearly
179 indicates that the G2 carries little load after the yielding of G1, and the applied load was mainly sustained by G1
180 and the concrete deck.

181 When the applied load increased to 583kN, sudden drop of the applied load was observed due to the local failure
182 of the concrete slab at the loading position. With the increase of the displacement (displacement control method
183 was used in the loading test), the applied load increased again, reflecting the rebalance at the loading point.
184 Thereafter, “turnover” behavior of the test specimen started to occur, which can be indicated from the sharp increase
185 of δ -1 and decrease (or reverse increase to upward direction) of δ -2. Thereafter, the δ -1 kept increasing quickly with
186 the increase of the load, while the downward displacement of δ -2 kept decreasing and then began the upward
187 displacement. In this stage, the load carried by G1 had been increasing while the load carried by G2 had been
188 decreasing, due to the damage of the concrete slab and insufficient load transfer or redistribution capacity of the
189 cross beams and damaged concrete deck.

190 In general, the functional limit state of a structure can be determined as the maximum vertical deflection of the
191 superstructure reaching a value equal to span length/100 (40mm for the specimen used in this study) (Lin et al.,
192 2013). In this study, however, vertical displacement of G1 had been loaded to nearly 70mm and no sudden failure
193 was observed. In this study, the failure mode of the specimen can be concluded as the local failure of concrete slab
194 at the loading point due to concrete crushing, the cracking (top surface) and the stripping of the concrete slab (bottom
195 surface) due to the torsion and lateral bending (bending moment perpendicular to the bridge axis) caused by the
196 eccentric loading, as shown in Fig.8.

197 ***Strain on the steel main girders***

198 Six strain gauges were attached on the bottom flange of the steel girder on three key sections (section-A, section-B,
199 and section-C), denoted by S-1~S-6 respectively, as shown in Fig.2 (a). The strain results of the steel bottom flange
200 under both symmetric and asymmetric loadings were measured in the tests and illustrated.

201 When the specimen subjected to symmetric loading, the strain on the steel girder was shown in Fig.9. The
202 corresponding theoretical relationships between the applied load and the strain on the basis of the elastic theory at
203 section-A, section-B, and section-C (denoted as “Theoretical value-A”, “Theoretical value-B” and “Theoretical
204 value-C”, respectively) were also provided. Both strains on G1 and G2 increase linearly with the increase of the
205 load, and the strains on G1 were similar to that on G2. Also, the comparison shows that the theoretical results agree

206 well the measured results in all three sections, indicating that the applicability of elastic design theory in designing
207 such bridges.

208 When the specimen subjected to asymmetric loading, the strain on bottom flange of the steel girder in the mid-
209 span section was shown in Fig.10. In the initial loading stage, both strains on G1 and G2 increases linearly as the
210 load increases, as shown in Fig.10 (b). However, the strain on G1 was much larger than that on G2, which was due
211 to the different vertical displacement as discussed in previous section. Taking the applied load of 300kN as an
212 example, the corresponding strain on G1 (1184μ) was nearly 2.75 times of that on G2, indicating that the G1 carries
213 nearly 73% of the load on the main girders. In the plastic stage ($P \geq 387\text{kN}$), however, this ratio becomes smaller and
214 smaller with the load increase. Due to the local crush and cracking of the concrete slab, the load transmission
215 capacity between two main girders becomes less and less. Different from the displacement reduction confirmed in
216 Fig.6 (b), strain reduction on G2 was not confirmed. This is presumably because of the torsional deformation of the
217 girder caused by the eccentric loading.

218 When the load reached to the peak load (670kN), as strain on the bottom flange of G1 was already around 45000μ ,
219 it can be considered as the ultimate state of the composite girder. However, the corresponding maximum strain the
220 G2 was only 1618μ , indicating that the unloaded girder (G2) was still in the elastic stage. Therefore, the load
221 transformation between two main girders in the plastic stage are relatively small.

222 ***Strain on Concrete Slab***

223 Nine strain gauges were installed on the top surface of the concrete slab on three key sections (section-A, section-
224 D, and section-E), denoted by C1~C9 respectively, as shown in Fig.2 (c). For section-D and section-E, two strain
225 gauges were employed on the right top of the steel girder, and the other one was used on middle between them. For
226 sections-A, two strain gauges were used on the two sides of a loading plate, and the third gauge was used on middle
227 of the concrete slab. The detailed locations of the strain gauges were shown in Fig.2. The strain results of the
228 concrete under both symmetric and asymmetric loadings were measured in the tests and shown in Fig.11 and Fig.12,
229 respectively.

230 When two symmetric loads were applied, the normal strain on top surface of the concrete slab was shown in
231 Fig.11. The corresponding theoretical values of the normal strain according to elastic theory were also determined
232 and provided. The results indicate that the normal strain on the concrete increased linearly with the increase of the
233 applied load. For sections-A and -B, the normal strains on the right top of the main girders (C-1, -3, -4, and -6) are
234 close to the strain on the middle top of the cross section (C-2, and -5), indicating the negligible shear lag effect of
235 the concrete slab under symmetric loads. This is also confirmed in the effective width calculation for the concrete

236 slab. According to Eq. (1), the effective width of the concrete slab is determined to be 1.12m, which is very close
237 to the actual width of the slab (1.2m). Also, the comparison shows that the theoretical results agree well the measured
238 results in all three sections, indicating that the applicability of elastic design theory in designing such bridges.

239 When the applied load changes to asymmetric loading, the strain on top surface of the concrete slab at all
240 measured sections (section-A, section-D, and section-E) was shown in Fig.12. In the initial loading stage ($P \leq$
241 300kN), normal strains on all three sections increase linearly as the load increases, as shown in Fig.12 (b). After
242 that, the strain kept increasing until the load increased to 583kN, when the local failure of the concrete slab and
243 sudden drop of the applied load was observed. As a result, decreasing of the normal strain was observed at C-1, C-
244 4, C-7, and C-8. The strain at C-1 and C-4 kept decreasing even after the rebound of the applied load, indicating the
245 local failure of at the loading point. Also, as the normal strains of C-7 and C-8 were still smaller than the ultimate
246 strain of concrete (-3500μ according to the JSCE, 2012), the failure of the concrete was due to the combined effects
247 of the vertical compression and longitudinal compression caused by applied load and bending moment. For strain
248 gauges at other locations of the concrete slab, the strain of concrete kept increasing with the increase of the applied
249 load, indicating that the concrete slab remains effective in carrying and distributing the applied load to a certain
250 extent until the ultimate load.

251 ***Strain on cross beams***

252 Cross beams are important members in multi-main-girder bridges to prevent the buckling of the main girders and
253 for load redistribution between main girders (Lin et al., 2019). In twin girder bridges, however, the effectiveness of
254 cross beams is not clear, and their behavior is still not well defined. Because of the unclear behavior of cross beam
255 in twin girder bridges, twin girder bridges are generally considered as non-redundant. In this study, eight flexural
256 strain gauges (CBS-1~CBS-8) were attached to the two cross beams (CB-2 and CB-3), as shown in Fig.2 (a). Strain
257 gauges were attached on both top and bottom at end sections.

258 The load-flexural strain results of two cross beams are shown in Fig.13. As the flexural strain keeps increasing
259 with the increase of the load, the cross beams in twin girder bridges are functional. Also, it was found that the further
260 the cross beam located from the loading point, the larger strain can be confirmed. The deformation of cross beams
261 may consist of vertical deformation, longitudinal deformation, and twist deformation. As the vertical deformation
262 decreases with the distance from the cross beams to loading point, the strain was mainly caused by the lateral
263 deformation of the beam. Another interesting phenomenon is that the flexural strain on both top and bottom flanges
264 in cross beams are in same sign conversion, indicating that they are in either tension or compression simultaneously.
265 If the cross beams are main subjected to vertical forces (shear or bending), the flexural strain on top and bottom

flanges should have equal (or similar) strain values with different sign conversion. When subjected to asymmetrical loading, however, lateral bending (bending moment perpendicular to the cross beam axis) and the torsional moment will occur in cross beams due to the relative deformation between two main girders, which might be the major cause for this phenomenon. This is not going to happen if the cross beams are main subjected to vertical bending. Therefore, in comparison with the load carrying capacity for vertical bending, the cross beams are more functional to sustain the lateral bending (bending moment perpendicular to the cross beam axis) and the torsional moment.

Strain Results on Shear Connectors

Although existing researches provided preliminary information on the behavior of shear connectors in composite structures, most of them were generated and analyzed from the push-out tests. The specimens were short and with limited number of shear connectors, and the tests were essentially pure shear tests (Yen et al., 1997). Also, flexural strain was mostly measured instead of shear strain, in view of studies performed by Lin et al. (2014). In actual twin I-girder structures, the composite girder is subjected to bending moment, transverse shear forces and the specimens contain much more number of shear connectors. On this background, the shear strain at the foot of the studs was measured in the loading tests, as shown in Fig.14. Ten shear strain gauges in the test specimen, as shown in Fig.2 (a). Shear strain gauges of L1~L4 were used to measure the shear strain of studs in longitudinal direction on G1, while L5~L7 were used to measure that of studs on G2. Also, in order to confirm the shear strain of stud in transverse direction, another three strain gauges (T1~T3) were attached on studs in G2.

Shear strain results of studs in the loading tests subjected to symmetric load are shown in Fig.15. Also, the theoretical prediction of the longitudinal shear strain at the measured location based on elastic theory was also provided. “Complete connection” was assumed on the steel-concrete interface, which means the slip on the interface was not considered. The theoretical value of longitudinal shear strain was determined by using the following equation:

$$\gamma = \frac{VQ}{GIt} \quad (2)$$

where γ is the shear strain at foot of the stud, V is the shear force, G is the shear modulus of the stud, I is the second moment of cross sectional area (twin I-girder section considering effective width of the concrete slab), t is the actual width of the section at the position where γ is being calculated (total width of top flanges of the twin girder in this study). $Q = A'\bar{y}$, where A' is the area of the top portion of the cross-sectional area above the measured location, and $\bar{y}$ is the distance from the neutral axis to the centroid of A' .

The longitudinal shear strain of studs on G1 and G2 were shown in Fig.15. Zigzag increase of shear strain on

295 studs was observed, which might be caused by shear force transmission at the initial loading stage. Theoretically
296 speaking, the shear strain at different sections should be the same due to the same shear forces under symmetric
297 loading applied at the mid-span. However, the strain results in Fig.15 (a) and Fig.15 (b) indicate that the shear strain
298 on studs near the 1/4 span was larger than those on studs near the girder end section or mid-span section, which was
299 similarly to the results observed in previous tests (Lin et al., 2014), and the fraction forces on the interface caused
300 by reaction forces at the supports and the applied loads are considered as the main reasons. In addition, the
301 comparison indicates that the theoretical results agree well with the results with the measured shear strain at L1 and
302 L2, but slightly larger than the results at L3 and L4. Therefore, it can be confirmed that the slip on the steel-concrete
303 can be ignored at the elastic stage, and “full shear connection” can be used for designing the twin I-girder bridges
304 with “full connection”. On the other hand, the results in Fig.15 (c) indicates that the shear strain of studs in the
305 lateral direction have similar increasing trend with those in longitudinal direction as shown in Fig.15 (a) and Fig.15
306 (b).

307 When subjected to an asymmetric load, the shear strain increase on studs versus applied load relationship was
308 shown in Fig.16. The results in Fig.16 (a) show that shear strains on all measured studs increase linearly with
309 increase of the applied load at the linear stage. After that, shear strains increase significantly as the applied load
310 increase, which might be caused by two reasons: upward movement of the neutral axis and breaking of the bonding
311 on the interface. After the yielding of the bottom flange, the neutral axis on G1 moves from the elastic neutral axis
312 (in the web of the steel girder) to the steel-concrete interface which caused the higher shear force transmission on
313 the interface. On the other hand, the failure of the chemical bond or friction on the steel-concrete interface cause
314 the shear force transmission from the chemical bond to the stud shear connectors. In addition, the comparison of the
315 shear strain on different studs on G1 indicates the similar conclusion observed in Fig.15, the maximum shear strain
316 is more likely to occur on studs near the 1/4 span. Approximately equal shear strain according to the constant shear
317 force distribution was not confirmed at the measured studs on G1. The shear strains on studs near the girder end
318 and near the mid-span are likely to be smaller, which might be caused by the friction forces on the interface due to
319 the reaction force and the applied load at girder end and mid-span, respectively. The shear strain shown in Fig.16
320 (a) indicates that the shear studs on G2 undertake very small shear force under the concrete crush, indicating that
321 little shear force was transmitted from G1 to G2. When the applied load increased to 583kN (local crush of concrete),
322 significant shear strain increase was observed on L5 and L7, but little increase was observed in L6. However, all
323 the shear strain are relatively smaller than those observed in studs on G1. It can be concluded that for asymmetric
324 load applied right top of the one main girder, the shear force transmission on the other main girder (un-loaded girder)

can be ignored. Fig.16 (c) shows the transverse shear strain at the foot of studs on G2. It can be seen that the transverse shear strain of shear studs on G2 was much larger than their longitudinal shear strains in both elastic and plastic stage. Therefore, the results clearly indicate that the combination of shear strains at both longitudinal and transverse directions should be considered in the design of studs in twin I-girder steel-concrete composite bridges.

Concluding Remarks

Static loading tests were performed on an intact steel-concrete composite twin-girder bridge model. Detailed static loading tests involving load-displacement response and load-strain relationships on the steel girder, concrete slab, stud shear connectors, and cross beams were reported and examined carefully in this paper. Two loading conditions including both symmetrical loading and asymmetrical loading were applied in the loading test. From the results presented herein, the following conclusions and recommendations deserving priority are made:

- (1) The theoretical results determined according to the current elastic design theory, including both displacements on different sections and strains on different members, agree well with the test results. Therefore, twin I-girder bridges can be designed appropriately according to the current bridge design method.
- (2) The unloaded girder needs to carry a certain amount of load applied on the right top of the other main girder in the elastic stage. Concrete slab is the key member in load transmission and distribution between two main girders. Load distribution capacity of deck decreases significantly after concrete failure.
- (3) Failure of the steel-concrete twin I-girder bridges is governed by the concrete crush. Under asymmetric loading, failure mode of the specimen was due to the local failure of concrete slab at the loading point due to crush, the cracking on top surface and the stripping on bottom surface of the concrete slab due to induced torsion and lateral bending.
- (4) Cross beams in the twin girder bridges are functional in load transmission and deformation compatibility between two main girders. Cross beams contribute more on constraining the longitudinal and torsion deformation than redistributing the vertical load between the two main girders.
- (5) In twin-girder bridges, the combination of shear strain at both longitudinal and transverse directions in stud shear connectors was confirmed in the loading test, and it should be considered in the design of studs in such bridges. Also, maximum shear strain of stud shear connectors is more likely to occur on studs near the 1/4 span.

Data Availability

Some or all data, models, or code generated or used during the study are available from the corresponding author by request (list items).

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List of Figure Captions

- Fig.1** Traditional multi-girder composite section and new structural form of twin girder composite section: (a) Multi-girder composite section; (b) Twin girder composite section.
- Fig.2** Size dimensions of the connection joint after strengthening (unit: mm): (a) Plane view of steel girder and shear strain gauge on studs; (b) Side elevation; (c) Plane view of concrete deck and reinforcement; (d) Cross-section of steel girder; (e) Composite section; (f) Stud distribution.
- Fig.3** Instrumentation in the loading test ((a) LVDT for displacement, (b) Strain gauges on main girder, (c) Strain gauges on stud)
- Fig.4** Loading conditions ((a) Symmetric load, (b) Asymmetric load)
- Fig.5** Loading test set-up ((a) Symmetric load ($P < 150\text{kN}$), (b) Asymmetric load)
- Fig.6** Load-displacement relationships ((a) Symmetric load ($P < 150\text{kN}$), (b) Asymmetric load)
- Fig.7** Effective width of flange
- Fig.8** Failure mode of the test specimen ((a) Top view of the concrete slab, (b) Bottom view of the concrete slab)
- Fig.9** Strain on bottom flange (Symmetric load)
- Fig.10** Strain on bottom flange (asymmetric load) ((a) Whole loading stage, (b) Initial loading stage)
- Fig.11** Normal strain on top surface of the concrete slab (symmetric load) ((a) Section-A, (b) Section-D, (c) Section-E)
- Fig.12** Normal strain on top surface of the concrete slab (asymmetric load) ((a) Section-A, (b) Section-D, (c) Section-E)
- Fig.13** Normal strain on cross beams ((a) CB-2, (b) CB-3)
- Fig.14** Shear strain gauge on shear stud ((a) Shear strain gauge location, (b) Stud in the test specimen)
- Fig.15** Longitudinal and transverse shear strain on shear studs (symmetric load) ((a) Longitudinal shear strain, G1; (b) Longitudinal shear strain, G2; (c) Transverse shear strain, G2)
- Fig.16** Longitudinal and transverse shear strain on shear studs (asymmetric load) ((a) Longitudinal shear strain, G1; (b) Longitudinal shear strain, G2; (c) Transverse shear strain, G2)

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