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

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## 9 Abstract

10 A large number of steel-concrete composite twin I-girder bridges have been built in both Europe and Japan, but the 11 lack of redundancy has always been a concern in the U.S. and many other countries. In addition, very few 12 experimental studies have been performed on mechanical performance of such bridges, particularly for the intact 13 twin I-girder bridges. On this background, a steel-concrete composite twin I-girder bridge specimen was designed 14 according to the current highway bridge design specification in Japan. The static loading tests were performed, and 15 two loading conditions including both symmetric loading and asymmetric loading were applied. Load versus 16 deflection relationship and strain development on steel main girders and concrete slab at key sections were measured. 17 The flexural strains on the lateral beam were also measured and reported in this paper to confirm the load transfer 18 between two main girders. In addition, the shear strain of shear connectors (stud, in this study) was also measured 19 to investigate the shear force transmission on the steel-concrete interface. The theoretical values were also provided 20 to compare with the test results from the twin girder specimen under symmetric loading condition. The experimental 21 results indicate that the theoretical analysis can predict the behavior of the twin-girders very well in the elastic stage 22 by considering the effective width of the slab. Performance of each structural components and load transfer paths 23 in such bridges were also discussed.

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

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# 26 Introduction

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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

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

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

60 of nearly 10m.

Recent studies on two-girder bridges mainly focused on the post-fracture redundancy and redundancy evaluation 61 of twin-girder systems, such as those performed by Idriss et al. (1995), Tachibana et al. (2000), Park et al. (2012), 62 Samaras et al. (2012), Kim and Williamson (2015), and Lin et al. (2013, 2016) etc. Limited studies on intact twin-63 girder bridges were also performed, mainly focusing on connection details of diaphragm, buckling of main girders, 64 shear-lag effect in slabs etc. Takahashi et al. (1997) performed an experimental study to investigate the structural 65 details of the connection of diaphragm used in the first steel twin-girder bridge "Horonai River Bridge". Four types 66 of connection details, including the connection with triangular rib plate or connection plate, split-tee connection and 67 endplate connection, were investigated. In 1999, Hotta et al. investigated both global and local lateral-torsional 68 buckling of twin girder bridges during erection of main girders in the case of the incremental launching method. 69 Dezi et al. (2006) performed an analysis to investigate the shear-lag effect in slabs of twin-girder steel-concrete 70 composite decks due to the main prestressing techniques such as support settlements, bonded cables inside the 71 concrete slab, and external slipping tendons. Kozy and Tunstall (2007) reported stability analysis results of a twin 72 I-girder pedestrian bridge constructed in Minnesota, and they pointed out that the strength of non-composite twin-73 girder systems without lateral bracing can be controlled by the limit state of 'system' or 'global' buckling. Awall 74 and Hayashikawa (2011) performed a parametric study on horizontally curved twin I-girder bridges, focusing on 75 the effects of curvature on the impact factor of such bridges. Ma and Shi (2016) performed numerical analysis to 76 investigate the parameters that affect the stability and capacities on twin-I girder systems. Parameters considered in 77 this study include girder depths, flange width-to-thickness ratio, web depth-to-thickness ratio, number of stiffeners, 78 and cross-beam spacing. However, experimental study on mechanical behavior of intact twin I-girder bridges was 79 rarely performed. In order to promote the application of such bridges, fundamental studies on such structures 80 become necessary. 81

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.

87 The experimental results of the intact specimen are reported in this paper.

# 91 Test Specimen

A simply supported steel-concrete composite twin I-girder bridge specimen was designed according to Japanese 92 bridge design specifications (JRA, 2012; JSCE, 2007). The specimen was designed as 4.4m in length and was 93 simply supported at a span length of 4m, by considering the available space in the structural laboratory. The width-94 to-span ratio and depth-to-width ratio are taken similar to that of typical composite twin I-girder bridges used in 95 Japan. Two I-girders with a height of 300mm were used as main girders, and the width of the concrete slab was 96 determined as 1200mm. According to bridge design specification in Japan (JRA, 2012), the minimum thickness of 97 concrete slab is 110mm. To also study the real behavior of studs and avoid the possible damage of the slab during 98 transportation, the slab thickness was determined as 120mm. The influence of the deck thickness on general 99 behavior of such structures was investigated based on the numerical analysis (Lam et al., 2017a and 2017b), and 100 101 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 interval of 400mm to prevent buckling failure and crippling of the web before flexural failure. Six cross beams (CB-103 104  $1 \sim 6$ ) were also used between the twin main girders for load redistribution. Vertical stiffeners on the web were also appropriately designed and arrange between cross beams. Four rows of shear stud connectors with the diameter of 105 16mm were placed with a uniformly spacing of 200mm in the longitudinal direction to ensure the full (or complete) 106 connection between the steel girder and the concrete slab. The theoretical values of yield and ultimate bending 107 moment capacity of the specimen considering the actual yield stress from the material tests are determined as 108 676kN m and 886kN m, respectively. Size dimensions and design details of test specimen are shown in Fig.2. 109

# 110 Instrumentation

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

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

A testing machine with the loading capacity of 5000kN was used to apply the load in the test. The test specimen 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-119 span section, as shown in Fig.4 (a). Three loading plates in total, were used. Two of them were used between the 120 loading beam and the concrete slab, located at right above the main girders, and the third loading plate were used 121 on the middle of the loading beam. In this stage, the applied load was controlled less than 150kN (about 22% of the 122 yield load of the twin girder) to avoid any possible damage of the test specimen. In the second stage, the load was 123 applied asymmetrically on the right top of one main girder (G1), as shown in Fig.4 (b). The load was applied by 124 static loading with unloading process. The loading will be terminated when either the maximum stroke of the jack 125 was reached or the load level of the test specimen dropped significantly. In both symmetrical and asymmetrical 126 loading conditions, pre-loading until 50kN was applied to check the reliability of the measuring equipment and the 127 stability of the test specimens. Displacement control method was used in the loading process with loading rates of 128 0.004mm/s for the pre-loading and 0.008mm/s for following loading test. The test specimen was supported by a 129 roller system at two ends. Set-up of the specimen in the experiment is shown in Fig.5. 130

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#### 132 Material properties

133 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, 134 29.5MPa, respectively, with the average compressive strength of 29.3MPa. The structural steel of SM490 was used 135 for the steel girder in this specimen. The web has a thickness of 6mm, and both top and bottom flanges have a 136 thickness of 12mm. According to the tensile test results, the yield strength and ultimate strength of the structural 137 steel for web were 450 and 577MPa, respectively. While, the yield strength and ultimate strength of structural steel 138 for top and bottom flanges were 389 and 548MPa, respectively. The reinforcement of D10 nominal diameter was 139 used for both longitudinal reinforcement and stirrup reinforcing bars in the concrete slab. According to the test 140 results, the yield strength and ultimate strength of the reinforcement were 406 and 544MPa, respectively. The stud 141 shear connectors were made of SS400, which has a nominal yield strength of 235MPa and an ultimate tensile 142 strength of 400MPa, respectively. 143

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# 145 Test Results and Discussion

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# 147 Load-Deflection Response

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

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

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

165

where  $\lambda$ : half effective width of flange (mm); *b*: half spacing between webs or projection width of cantilever portion (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  $\delta$ -1 and  $\delta$ -2 also increase linearly with the load increases. However, the  $\delta$ -1 was much 170 larger than that of  $\delta$ -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=387kN, the yielding was confirmed on web-bottom flange junction of G1.

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

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

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

# 197 Strain on the steel main girders

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

When the specimen subjected to symmetric loading, the strain on the steel girder was shown in Fig.9. The corresponding theoretical relationships between the applied load and the strain on the basis of the elastic theory at section-A, section-B, and section-C (denoted as "Theoretical value-A", "Theoretical value-B" and "Theoretical value-C", respectively) were also provided. Both strains on G1 and G2 increase linearly with the increase of the load, and the strains on G1 were similar to that on G2. Also, the comparison shows that the theoretical results agree well the measured results in all three sections, indicating that the applicability of elastic design theory in designingsuch bridges.

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

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

### 222 Strain on Concrete Slab

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

When two symmetric loads were applied, the normal strain on top surface of the concrete slab was shown in Fig.11. The corresponding theoretical values of the normal strain according to elastic theory were also determined and provided. The results indicate that the normal strain on the concrete increased linearly with the increase of the 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 close to the strain on the middle top of the cross section (C-2, and -5), indicating the negligible shear lag effect of the concrete slab under symmetric loads. This is also confirmed in the effective width calculation for the concrete slab. According to Eq. (1), the effective width of the concrete slab is determined to be 1.12m, which is very close to the actual width of the slab (1.2m). Also, the comparison shows that the theoretical results agree well the measured results in all three sections, indicating that the applicability of elastic design theory in designing such bridges.

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

#### 251 Strain on cross beams

Cross beams are important members in multi-main-girder bridges to prevent the buckling of the main girders and for load redistribution between main girders (Lin et al., 2019). In twin girder bridges, however, the effectiveness of cross beams is not clear, and their behavior is still not well defined. Because of the unclear behavior of cross beam in twin girder bridges, twin girder bridges are generally considered as non-redundant. In this study, eight flexural 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 gauges were attached on both top and bottom at end sections.

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

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.

### 272 Strain Results on Shear Connectors

273 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 274 limited number of shear connectors, and the tests were essentially pure shear tests (Yen et al., 1997). Also, flexural 275 strain was mostly measured instead of shear strain, in view of studies performed by Lin et al. (2014). In actual twin 276 I-girder structures, the composite girder is subjected to bending moment, transverse shear forces and the specimens 277 contain much more number of shear connectors. On this background, the shear strain at the foot of the studs was 278 measured in the loading tests, as shown in Fig.14. Ten shear strain gauges in the test specimen, as shown in Fig.2 279 (a). Shear strain gauges of L1~L4 were used to measure the shear strain of studs in longitudinal direction on G1, 280 while L5~L7 were used to measure that of studs on G2. Also, in order to confirm the shear strain of stud in transverse 281 direction, another three strain gauges (T1~T3) were attached on studs in G2. 282

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:

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$$\gamma = \frac{VQ}{GIt} \tag{2}$$

where  $\gamma$  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  $\gamma$  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'.

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

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

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

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.

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# 330 Concluding Remarks

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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
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span.

355	Data Availability
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357	Some or all data, models, or code generated or used during the study are available from the corresponding author
358	by request (list items).
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# **List of Figure Captions**

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











(a)



(b)





















