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REDUNDANCY OF CONTINUOUS TWO-GIRDER STEEL-CONCRETE COMPOSITE HIGHWAY BRIDGES

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ABSTRACT

Post-fracture redundancy of the two-girder bridges is investigated through a case study of a three-span continuous twin I-girder bridge designed in Japan. The FE analysis is employed to evaluate the load-carrying capacity after the fracture of one main girder of the two-girder bridge. The critical locations include the mid-span section of the middle span, the side span, as well as the section near the continuous pier support. Besides, damage of studs near the moment contra-flexure regions in both middle and side spans are also considered in this study. Load-deflection response of the damaged bridge was compared with the intact bridge system, and the typical failure modes were described. Besides, the direct redundancy analysis results indicated that the present continuous twin-girder steel-concrete bridge can be classified as redundant. The concrete slab is considered as the significant member for redundancy of the two-girder steel-concrete composite bridges.

Keywords: Two-girder bridge; composite bridge; redundancy; FEM; post-fracture.

1. INTRODUCTION

In recent years, there are many steel and steel-concrete composite bridge structures with fatigue cracking problems in both United States and Japan. The Hoan Bridge was temporarily closed on December 13, 2000, after two of the three support beams of the lakefront span failed, causing the north-bound lanes to buckle and sag by several feet, and leaving the span in a near collapsed state (TRB, 2005). On Aug. 1, 2007, I-35W bridge collapse in Minnesota killed 13 people and injured 145 (NTSB, 2008). In Japan, almost at the same time, a diagonal member of two steel truss bridges in national highways was fractured due to corrosion. Furthermore, a steel truss bridge over the border between Tokushima and Kagawa prefectures collapsed in Nov. 2007 (Okui et al., 2010). Those accidents promoted the researches on the redundancy of bridge structures.

AASHTO considers virtually all two-girder bridge systems as non-redundant, and each girder is considered as a fracture-critical member (FCM). In other words, the tension flange of a two-girder

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bridge system has no alternate load paths and its failure would result in full (or partial) collapse of the structure or render the bridge unfit for use (Crampton et al., 2007). On this background, this paper presents preliminary results from an analytical investigation of the redundancy of twin I-girder steel-concrete composite bridges. The goal of present study is to determine if the typical three-span continuous steel-concrete composite bridges can be classified as redundant and non-fracture critical, thus relieving concerns over selection of this bridge type by reducing fabrication and life-cycle inspection costs. The key parameters are locations of main girder damages and stud damages.

2. MODEL BRIDGE AND NUMERICAL SIMULATION

2.1. Model Bridge and Damage Scenario

Figure 1 shows a bridge model employed in the case study, which is designed according to the design code of Japan (JRA, 2002). The bridge was designed in straight as three spans, with the span length of (37.5+43+37.5) m. In the post-fracture redundancy analysis, only one member is assumed to fracture once, and the assumed fracture members are shown in Figure 1(a). The damage state considered in this study is a fracture of one of the steel I-girders and limited number of shear studs. Generally speaking, the critical location for girder damage of simple span bridges is at mid-span. In order to fully evaluate the effect of bridge continuity on redundancy, however, additional damage locations was investigated. These areas of damage include near the continuous pier support (Case-3). Also, damage of studs near the moment contra-flexure regions was also considered in this study (Case-4 and 5). The damage location was assumed 0.15L (L denotes the span length) apart from the middle support, with a total length of 2 meters, which is about 3 times of the maximum stud spacing of 0.6 m specified in JSCE(2007), and 0.8 m specified in Euro code (CEN, 2003).

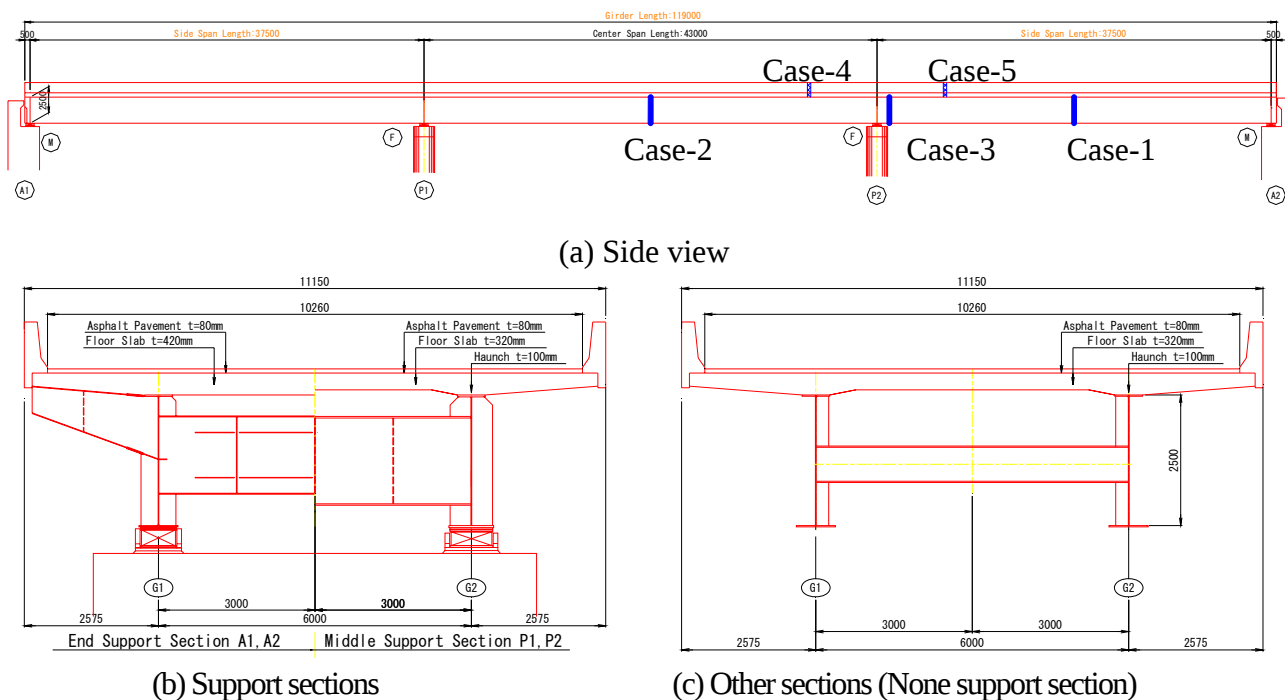


Figure 1. Model bridge

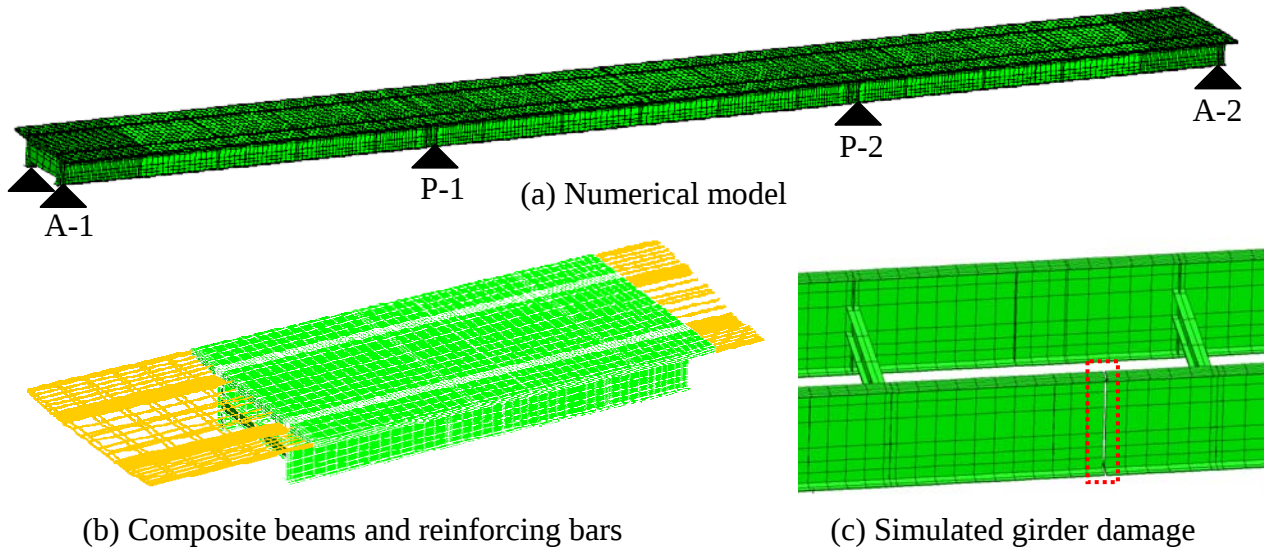


Figure 2. Numerical simulation of the damaged composite girder

2.2. Numerical Model

The modeling of each numerical model was carried out in three dimensions by using the finite-element method and the *DIANA* software, as shown in **Figure 2**. Solid elements, shell elements, spring elements were used to simulate the concrete slab, steel girder, and stud connectors, respectively. For each stud, three springs were employed for simulating the shear and axial forces in three directions. Re-bar elements were used for modeling reinforcing bars in the concrete slab. With the purpose of accounting for the slip and the composite action between concrete slab and steel girder, interface elements were employed in the numerical models. The damage is considered in the FE model by reducing the Young's modulus of a line of steel girder shell elements at the critical location to a near-zero value. Phase analysis, considering dead load in the construction phase and the live load in the service stage, was performed to evaluate the stress distributions. Besides, both physical and geometrical nonlinearity are considered in the nonlinear redundancy analysis. The validity of the present numerical method can be demonstrated by referring to the Authors' previous publications (Lin et al., 2012, 2013a, and 2013b).

The stress-strain curve due to JSCE (2002) and the experimental tension-stiffness curve proposed by Nakasu et al. (1996) were employed to simulate the compression and tension behavior of concrete, respectively. The Elasto-plastic stress-strain relationships for structural steel and reinforcing bars were adopted in numerical models. As suggested by JSCE (2002), strain hardening of the structural steel and reinforcing bars are not considered as a safe side consideration. SM570, SM490Y, SD345, were used for main structural steel members and reinforcing bars depending on the locations. In the numerical study, the shear studs were modeled by 3D nonlinear spring elements. For each stud, three springs are used, two in the horizontal direction and one in the vertical direction. The spring is simulated by using the constitutive relationship suggested by Ollgaard et al. (1971). In this study, the experimental-based bond-friction interface model was employed to simulate the composite action

between the steel girder and the concrete slab. The maximum bond stress f_{bo} and corresponding slip was taken as 0.5 N/mm^2 and 0.06mm (DÖRR et al, 1980), respectively.

2.3. Live load conditions

For each redundancy analysis, the dead load and live load (P_2) are considered, and the live load is applied so as to maximize the deformation of the assumed fractured member in the intact bridge system. The primary live load used for the analysis was a P_2 standard design load (JRA, 2002) with a distributed load of 3.5kN/m^2 excluding the impact loading. In this study, unsymmetrical load was used for redundancy evaluations, as shown in **Figure 3**.

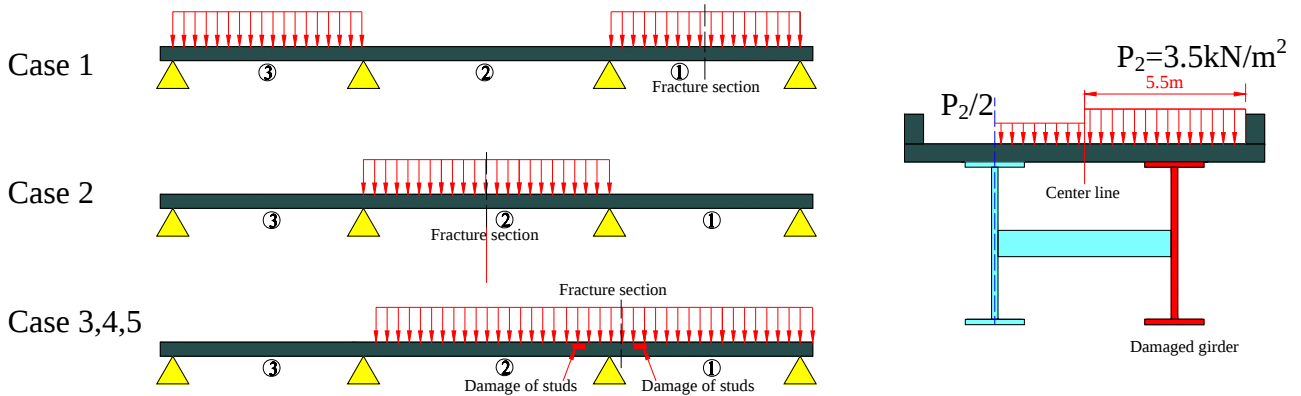


Figure 3. Live load conditions

3. DISCUSSION OF NUMERICAL RESULTS

3.1. Load-deflection response

The load factor-deflection relationship of the present bridge was compared with the corresponding intact bridge system, as shown in **Figure 4**. As mentioned above, the phase analysis was performed in this study, and the live load was applied after the dead load. For all the present extreme cases, the bridge structure can withstand the dead load without collapse. Therefore, in the load-displacement curves, live load factor was employed to present the relationship between the applied design live load and the vertical displacement.

The live load factor versus displacement of Case-1 is shown in **Figure 4 (a)**. The through-crack is assumed at the middle section of side-span (through-crack is only on the steel girder, but not the concrete slab). Under the dead load, both the steel girder and the rebar are still in the elastic stage. Under the live load, when the live load factor increased to 9.08, the calculation was stopped because of the local buckling on the web. The location is close to the pier support, and the buckling could be caused by the negative bending moment and the shear force. As a comparison, the live load factor of the intact system can keep increasing until 19.5. The through-crack was assumed at the center of mid-span in Case-2, and the load-displacement curve is shown in **Figure 4 (b)**. Similar to Case-1, both the steel girder and the rebar are still in the elastic state under the dead load. Under the live load, when

the live load factor increased to 10.3, the calculation was stopped because of the local buckling on the web. The maximum live load factor for the intact bridge system under the same load condition was predicted to be 19.9, as show in **Figure 4 (b)**.

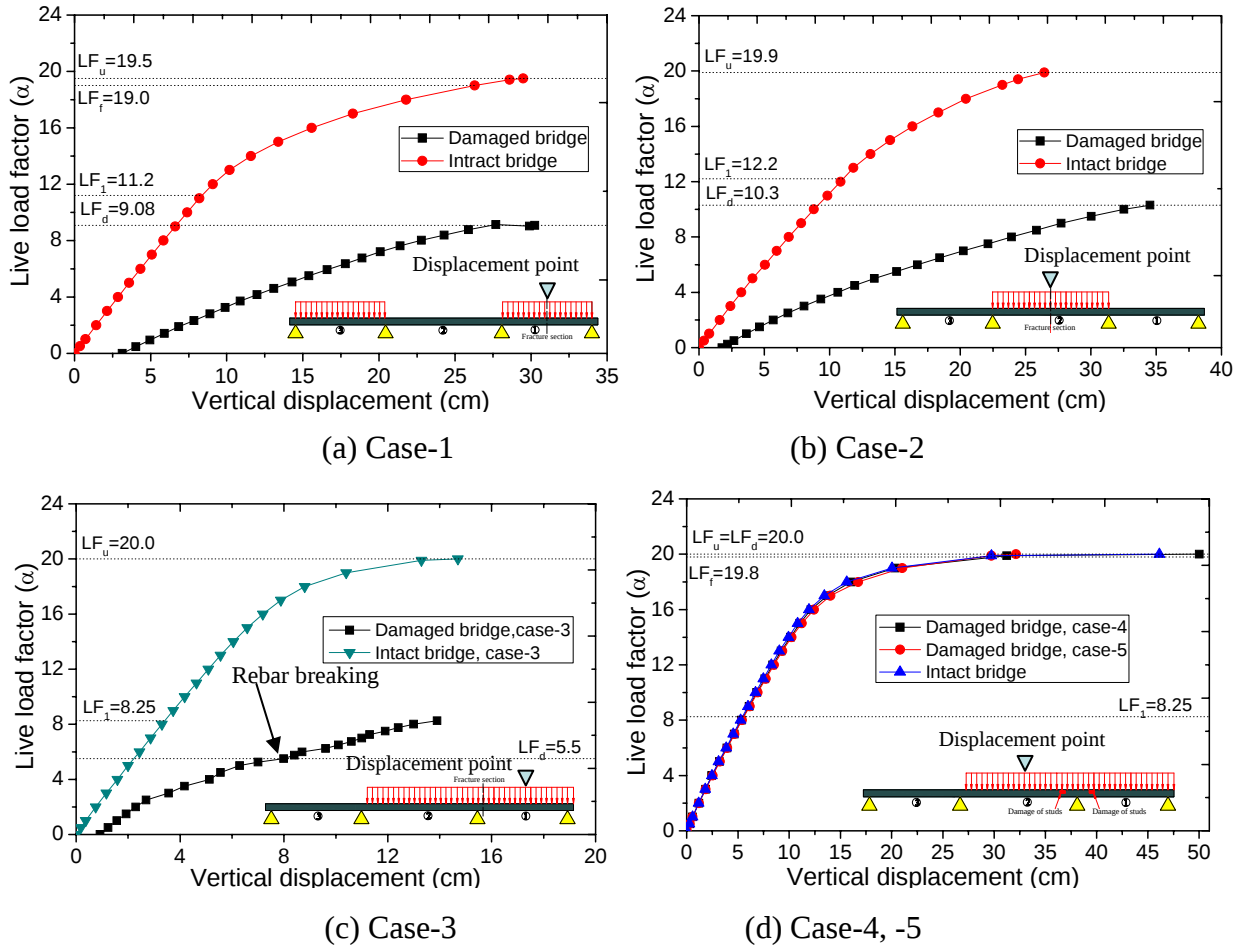


Figure 4. Live load factor-vertical displacement response of different extreme cases

For Case-3, the through-crack was assumed near the continuous pier support, and the load-vertical deformation response is shown in **Figure 4 (c)**. As the rebar strain is believed to begin breaking (rebar strain is larger 20%), the LF_d is taken as 5.5 although the calculation could still continue. The maximum live load factor for the intact bridge system of Case-3 is 20. Damage of studs near the moment contra-flexure regions are considered in Case-4 and -5. The live load factor versus displacement shown in **Figure 4(d)** indicates that both LF_d and LF_u are around 20. This comparison demonstrates that the damage of limited number of shear connectors will not affect on the global behavior of the bridge structure.

3.2. Failure modes and classifications

According to the numerical results, the failure modes of the present bridge structure in different extreme cases can be classified into two types: steel girder buckling and concrete crash, and shear failure of the concrete slab and rebar, as shown in **Figure 5**.

For Case-1 and 2, as the through-crack is assumed at the mid-span, the failure of the damaged

bridge system generally can be described as: serious cracking is generated on the concrete slab in both cracked locations and mid-support regions under the dead load, but the steel beam and the reinforcing bars are still in the elastic state. With increasing of the live load factor, the rebar in the through-crack location firstly yields, and relatively large deformation is produced in the cracked location. The deformation increases the concrete compression at the through-crack location, concrete cracking as well as the steel girder compression at the mid-support regions. Concrete crash at the through-crack and the steel beam buckling occurs when the live load factor increases to a certain level. Also, concrete crash would promote mutually with steel buckling as the load factor increases. Finally, collapse occurs because the damaged bridge system becomes unstable and cannot withstand any increase of the live load, as shown in **Figure 5 (a)**.

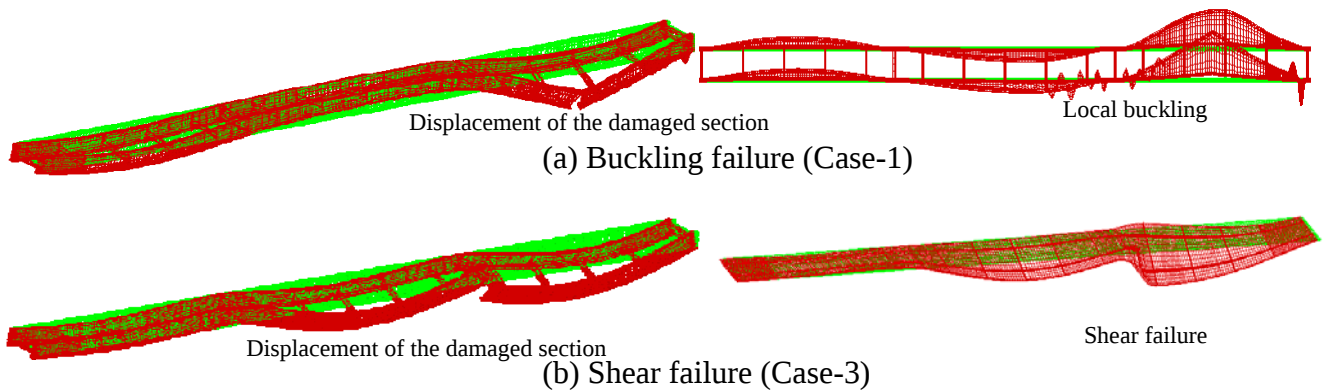


Figure 5. Typical failure modes in extreme cases

Failure modes of Case-3 can be classified as shear failure in the through-crack section, as shown in **Figure 5(b)**. Different from the Case-1 and 2, the strain of the rebar in the damage section is relatively large under the dead load and in the plastic range. Any increase of the live load factor will greatly increase the strain in the reinforcing bars. Through-crack of the concrete slab in the damaged location will appear when the live load factor is still relatively small. Finally, as the strain in the reinforcing bars beyond the maximum strain of 20%, the calculation is stopped because the rebar is assumed to begin breaking and the damaged system could not bear any increase in the live load.

For Case-4 and 5, the failure mechanism is close to that of Case-1 and 2, and can be classified as type-1 failure mode. However, as the damage of the shear connectors does not greatly affect on the mechanical behavior of the bridge structure, their failure modes are close to the intact system. The live load factor is relatively high and the crash of the concrete in the mid-span section can be treated as the beginning of the bridge failure.

3.3. Direct Redundancy Analysis results

Referring to NCHBR 458 Report (Liu et al. 2001), the following calculation was used to evaluate the redundancy of the present bridge structure. The system factor relating to ductility and redundancy of **Eq. (1)** for the structural components of a superstructure shall be calculated from the results of an incremental analysis.

$$\phi_s = \min\left(\frac{R_u}{1.3}, \frac{R_f}{1.1}, \frac{R_d}{0.5}\right) \quad (1)$$

where:

R_u , R_f , and R_d = system reserve ratio for the ultimate limit state, functionality limit state, and the damage condition, respectively;

$$R_u = \frac{LF_u}{LF_1}, \quad R_f = \frac{LF_f}{LF_1}, \quad R_d = \frac{LF_d}{LF_1}$$

LF_u , LF_f , LF_d and LF_1 are obtained from the incremental analysis, where:

LF_u = the vertical live load factor that causes the failure of the superstructure

LF_f = the vertical live load factor that causes the maximum vertical deflection of the superstructure to reach a value equal to span length/100.

LF_d = the vertical live load factor that causes the failure of a damaged superstructure

LF_1 = the vertical live load factor that causes the first member of the intact superstructure to reach its limit capacity

Table 1 Direct Redundancy Analysis Results

Case	LF_1	LF_u	LF_f	LF_d	$R_u/1.3$	$R_f/1.1$	$R_d/0.5$	ϕ_i	ϕ_s
1	11.2	19.5	19.0	9.08	1.34	1.54	1.62	1.34	
2	12.2	19.9	—	10.3	1.25	>1.48	1.69	1.25	
3				5.5			1.33	1.33	1.25
4	8.25	20	19.8	20	1.86	2.18	4.85	1.86	
5				20.1			4.87	1.86	

Notes: ϕ_i denotes partial redundancy factor of different extreme cases; “—” means the data of LF_f is not available because $LF_f > LF_u$ according to the numerical results.

Direct redundancy analysis results are given in **Table 1**, and it indicates that the present three-span continuous twin-girder steel-concrete composite bridges can be classified as redundant. Besides, it was found that the live load factor of the mid-span is higher than that of the side-span. Damage near the continuous pier support has the lowest live load factor in comparison with other damage scenarios because of the cantilever condition of the survived girder. However, the redundancy factors for different damage conditions are close to each other and larger than recommended values. It should also be noted that the real redundancy factor of the present bridge is larger than that given herein, because the tension stiffening of the structural and the reinforcing bars was ignored in the numerical analysis.

4. CONCLUSIONS

A case study of the post-fracture redundancy on a three-span two-girder steel-concrete composite bridge is carried out. Based on the nonlinear redundancy analysis, it was found that the live load factor of the mid-span is higher than that of the side-span. Damage near the continuous pier support

gives the lowest loading capacities in comparison with other damage scenarios. The damage of studs near the moment contra-flexure regions was also considered in this study. And it was found that the damage of limited number of shear studs would not affect obviously on the loading capacity of the bridge structures. Direct redundancy analysis was described and performed in this study, and the results demonstrated that the present three-span continuous twin-girder steel-concrete composite bridge can be classified as redundant.

The slab systems, including the concrete deck and the reinforcing bars, have substantial effects on the post-fracture redundancy. Even though one main girder is fractured, or limited number of shear connectors was damaged, the concrete slab continues to support the load, and the collapse of the bridge can be avoided in many ways.

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