The First International Conference of Pure and Engineering Sciences (ICPES2020)
IOP Conf. Series: Materials Science and Engineering 871 (2020) 012004
doi:10.1088/1757-899X/871/1/012004

Experimental Behavior of Continuous Hybrid Reinforced Concrete Spliced Girders

A Y Aljanabi1 and A H Hassoon2

1 Department of Civil Engineering, College of Engineering, University of Babylon, Babylon, Iraq
2 Department of Civil Engineering, College of Engineering, University of Al-Qadisiyah, Al-Diwanyah, Iraq

Abstract
This paper conducts an experimental study on the behavior of two-spans continuous reinforced concrete spliced girders strengthened using steel fiber concrete of volume fraction of 1% at splice regions and side near surface mounted carbon fiber reinforced polymer bars of 6 and 10 mm sizes. Five beams with 150 × 250 mm rectangular cross section and 2200 mm total length were cast. The first beam was cast as one unit without joints as a reference while the rest were made from assemblage of three precast segments spliced together using cast in place concrete joints of 170 mm length located at the inflection points. All girders were tested using a single concentrated force at each midspan. The studied parameters were: existence of splices, using of steel fiber concrete at splice region, and strengthening of spliced girders using side near surface mounted carbon fiber reinforced polymer bars. The results showed that existence of splices led to a decrease in the ultimate capacity of the spliced girder of about 33.1% with larger midspan deflections in comparison with the non-spliced girder. But, strengthening of spliced girders using CFRP bars could provide additional load capacity from 69.1% to 98.1% compared with the non-strengthened spliced girder with lower midspan deflections while steel fiber concrete at joints raised the ultimate load of spliced girders by 12.3%. However, the failure mode of the spliced beams were brittle if compared with the ductile failure of the control girder.

1. Introduction.
Long span girders are difficult to perform as one unit. Precast concrete jointed girders represent an economic technique to produce them owing to their high strength and quality [1]. This technique provides a solution for transportation and construction of the bridges. Also, span ranges of girders can be increased significantly which lead to a limited number of interior supports. Jointed girders consisted of one or more precast segments linked together at joints using several techniques [2]. Despite the behavior of such type of girders have been taken a great interest in last decades, research studies dealing with this problem are still limited.

Through an experimental and analytical study concerned with pre-stressed concrete girders spliced in field, Al-Mamuree [3] tested sixteen reinforced concrete spliced beams of rectangular cross section with different lengths, supports, splicing locations, and load type. Although the non-strengthened spliced girders had a load capacity higher than the non-spliced girders by (10.7 – 14.5%), using of posttensioning reinforcement with an increase ranged from (50 – 100%) could rise the ultimate capacity of the spliced girders by (11 – 16%) in comparison with the non-spliced girders having lower deflections.

Al-Quraishy [4] conducted an experimental and numerical investigation on the precast concrete girders behavior. Fifteen specimens with rectangular cross section were tested. The strengthening techniques involved using of steel plates at splice region and post-tensioning the precast segments. Four points load system was used for all girder specimens. The author concluded that strengthening using steel plates could provide a load capacity higher by (1 – 7%) of the non-spliced girders while posttensioning raised the load capacity by (70 – 132%).

Shaarbaf et al. [5] carried out an experimental study of the shear strength of precast concrete bridges. Slip between precast concrete segments and failure mode were the parameters considered. As a
conclusion, the ultimate load of the bridges increased by 16.3%, 50.7% and 60.9% for one, two and three shear keys specimens respectively in comparison with the reference flat specimen. Also, increasing the level of prestressing from 175 to 524.2 MPa increased the specimen’s capacity by about 40%.

Saibabu et al. [6] investigated experimentally a scaled simply supported precast box-girder bridge model with posttensioning. Dry and epoxy joints were evaluated under static and cyclic loading. The results showed that the maximum and failure loads of dry jointed specimens were less than those of epoxy joint by 8.6% and 16.7% owing to the high rotations and deflections at individual joints. Also, first dry joints opening load was 27% less than the epoxy joints.

The failure of dry jointed girders occurred at interfaces between precast segments while the failure was developed in the concrete adjacent to the interfaces in girders with epoxidized joints [7]. Steel fiber reinforced concrete (SFRC) could improve the shear strength and ductility of the dry joints [8].

Al-Tameemi [9] studied the behavior of RC spliced girder experimentally and numerically. The study concerned the using of carbon fiber reinforced polymer (CFRP) laminates in strengthening the spliced beams. Nineteen beams with rectangular cross section divided into two main groups depending on supports condition were tested. As a conclusion, longitudinal strengthening of simply supported girders using CFRP laminates was found to be more effective than transverse strengthening. In addition, continuous beams strengthened with horizontal or 45° inclined angle CFRP laminates at joint locations could rise the ultimate load capacity by (42 – 77%).

Jiang et al. [10] concluded that the location of joints had an important effect on the shear strength of precast girders with dry joints. Also, the failure modes of them were independent of joint types.

Research studies involved using of steel fiber concrete in joint regions or strengthening by NSM FRP bars have not been investigated yet. So, the present study will investigate the effect of both techniques on the behavior of continuous spliced girders in terms of cracking load, ultimate load, load-deflection response, and crack patterns at failure.

2. Methodology

2.1. Specimens

Five continuous reinforced concrete prototypes of two spans were cast in this study to represent the experimental variables of research plan. All beams had 150 × 250 mm rectangular cross section and total length of 2200 mm. The specimens were designed according to (ACI 318-11) [11]. A control beam was cast as one unit while the others were made from assembling of three precast concrete segments using hooked dowels at splice regions located approximately at the inflection points. The splice region of the second girder was cast with normal concrete while steel fiber concrete with ordinary steel fibers of volume fraction equal to 1% was used in the splice region of the third beam [12]. The last two beams were similar to the second beam but they were strengthened at joint regions with horizontal and 30° inclined side near surfaced mounted CFRP bars, respectively. Table 1 provides the details of the spliced beams used in this study while Figures 1 and 2 show their details.

| Beam symbol | Joint length (mm) | Type of concrete in joints | Number and size of CFRP bars |
|-------------|------------------|---------------------------|-----------------------------|
| CB1         | -                | NC                        | -                           |
| CSB2        | 170              | NC                        | -                           |
| CSB3        | 170              | SFC                       | -                           |
| CSB4        | 170              | NC                        | 12 Ø 6mm (horizontal)       |
| CSB5        | 170              | NC                        | 4 Ø 10 mm (inclined by 30°) |

Table 1. Variables of the tested girders.

![Figure 1. Details of the control girder (CB1).](image-url)
2.2. Materials

Two types of concrete were used in the experimental work which are normal concrete (NC) and steel fiber concrete (SFC). For steel fiber concrete, ordinary steel fibers of 1% volume fraction were used. Table 2 provides the properties of the steel fibers.

| Table 2. Properties of the steel fiber. |
|--------------------------------------|
| Specific Density (g/cm³) | Diameter (μm) | Length (mm) | Tensile Strength (MPa.) | Elastic Modulus (GPa.) |
| 7.8            | 160          | 12          | 2700                    | 200                     |

* Supplied by the manufacturer

Two sizes of deformed steel bars were used in the present study. Bars of (Ø10 mm) size were used as longitudinal reinforcement while (Ø6 mm) bars were used for stirrups. Three specimens of each bar size were tested according to ASTM A615/A615M-09b [13]. Table 3 provides steel reinforcement properties.

| Table 3. Properties of the steel reinforcement. |
|-----------------------------------------------|
| Bar size (mm) | Actual diameter (mm) | Yield Stress (MPa.) | Ultimate stress (MPa.) | Modulus of elasticity (GPa). |
| Ø6 mm         | 5.66                  | 634.2               | 692.7                  | 200                       |
| Ø10 mm        | 9.81                  | 578.1               | 653.3                  | 200                       |

* Assumed
Also, two sizes of CFRP bars were used in strengthening of the spliced girders. Horizontal bars of (Ø6 mm) were used in strengthening of girder (CSB\textsubscript{4}) while the girder (CSB\textsubscript{5}) was strengthened by (Ø10 mm) CFRP bars inclined with 30°. The mechanical properties of CFRP bars are listed in Table 4.

### Table 4. Mechanical properties of CFRP bars.\textsuperscript{a}

| Size (mm) | Nominal area (mm\textsuperscript{2}) | Tensile strength (MPa.) | Ultimate tensile load (kN) | Tensile modulus of elasticity (GPa.) | Ultimate strain |
|----------|-------------------------------------|-------------------------|---------------------------|-------------------------------------|----------------|
| 6        | 31.67                               | 2241                    | 70.8                      | 124                                 | 1.81\%         |
| 10       | 71.26                               | 2172                    | 154.1                     | 124                                 | 1.75\%         |

\textsuperscript{a} Supplied by the manufacturer

2.3. Casting and Strengthening of the Specimens

The control girder and all precast segments were cast with normal strength concrete and cured. Then, joint regions were cleaned, prepared and cast with normal concrete and steel fiber reinforced concrete. During casting process, twenty cubes, eight cylinders, and four prisms of were sampled to evaluate the mechanical properties of concrete. Table 5 provides the laboratory test results of NC and SFC.

### Table 5. Mechanical Properties of NC and SFC

| Type of concrete | Type of segment | Compressive strength, cube (MPa) | Compressive strength, cylinder (MPa) | Splitting tensile strength (MPa) | Flexural strength (MPa) |
|------------------|----------------|----------------------------------|-------------------------------------|---------------------------------|------------------------|
| NC               | Precast        | 42.93                            | 33.39                               | 2.69                            | 3.21                   |
|                  | Joint          | 45.1                             | 35.8                                | 2.77                            | 3.46                   |
| SFC              | Joint          | 47.6                             | 38.8                                | 4.34                            | 5.12                   |

After curing of specimens completed, the spliced girders CSB\textsubscript{4} and CSB\textsubscript{5} were strengthened by side near surface mounted CFRP bars as illustrated in Figure (2). The strengthening process were performed according to (ACI 440.2R-08) [14].

2.4. Test Procedure

All samples were cleaned, painted with white, and transported for test. Two exterior supports at 100 mm from the wright and left ends of the girders were placed. Additional third support was also placed under girder center to produce two equal spans of 1000 mm clear length. At each midspan, a concentrated load has been applied. The girders have been tested up to failure using a hydraulic machine of 500 KN ultimate capacity with a load increment of 10 KN. The deflections were measured at each midspan using two dial gauges of 0.01 mm accuracy while a micro crack meter of 24 mm range and an accuracy of 0.02 mm was used to measure the crack width of concrete during loading process. Figure 3 shows the test setup of girder specimens.

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**Figure 3.** Test setup.
3. Results and Discussion

3.1. Cracking and Ultimate Loads

During test, the deflection were recorded at each midspan then the maximum value has been taken for each load increment. First cracking load, ultimate load; and the maximum midspan deflections are listed in table 6.

| Beam symbol | Cracking load, P_cr (kN) | Ultimate load, P_u (kN) | P_u(l) / P_u(Ref.) | Maximum deflection (mm) |
|-------------|--------------------------|--------------------------|-------------------|-------------------------|
| CB_1        | -                        | 242                      | -                 | 5.51                    |
| CSB_2       | 10                       | 40                       | 162               | 0.67                    | 3.59 |
| CSB_3       | 20                       | 60                       | 182               | 0.75                    | 5.06 |
| CSB_4       | 100                      | 90                       | 274               | 1.13                    | 3.01 |
| CSB_5       | 60                       | 60                       | 321               | 1.33                    | 4.46 |

3.2. Load Deflection Response

The load deflection curves of the tested girders are presented and compared with each other as shown in Figure 4.

![Figure 4](image-url)

3.3. Crack Patterns and Failure Modes

During applying of loads, several cracks were recognized. Only flexural cracks were observed in the non-spliced control girder, while interface, flexural, and diagonal cracks were appeared in the spliced girders.

For the control girder CB_1, the first crack occurred in the negative moment region over the internal support at a load of about 70 kN started from the top of the girder while the load that flexural cracks
appeared in the positive moment region was 120 kN. When the applied load was exceeding 140 kN, the cracks extended to about 80% of the girder depth in both regions and widened. Final stages of loading showed turning of cracks to the loading points. The girder failed at a load of 242 kN owing to the yielding of steel reinforcement in tension.

First crack in the spliced girder CSB$_2$ occurred in the interface between the precast and joint segments at a load of 10 kN while the flexural crack appeared at a load of 40 kN near midspan. On the other hand, the first flexural crack in the negative moment region occurred at a load of 120 kN while the old cracks were extending and widening with increasing of load increments. The interface cracks turned toward the loading points during the last stages of loading till sudden failure of the girder at a load of 162 kN by direct shear.

Using of steel fiber concrete in the splice region for the spliced girder CB$_3$ delayed the first interface crack to a load of 20 KN. Similarly, the flexural crack that occurred in the negative moment region over support was also delayed to a load of and 50 kN. At a load of 100 kN, positive flexural crack appeared while the old cracks were extending and widening. As loading continued, the interface cracks were rapidly inclined, enlarged and extended to the loading points. Failure of this girder occurred at a load of 182 kN by incidence of direct shear.

When the spliced girder CSB$_4$, which was strengthened by horizontal CFRP bars of size of (6 mm) was loaded, the first crack occurred at a load of 90 kN, which was flexural crack approximately near midspan. Thenafter, the interface crack appeared at a load of 110 kN. As loading continued, new flexural cracks formed, extended, and widened. After the load exceeded a value of 200 KN, splitting cracks appeared through joint segments and met the interface crack forming an inclined crack that extended to the loading points from the top and the internal support from the bottom. The girder failed at a load of 274 kN owing to the direct shear and splitting.

Two cracks occurred simultaneously at a load of 60 kN during the test of girder CSB$_5$ which was strengthened by inclined CFRP bars of size of (10 mm). The first was interface while the second was flexural in the positive moment region. Then, a flexural crack in the negative moment region appeared at a load of 90 kN over support. Additional flexural cracks formed with increasing of load increments while the old cracks were extending and widening. The interface cracks inclined towards the loading points a load values over 220 kN. The failure load of this girder was 321 kN by flexural shear. The load – crack width relations are shown in Figure 5 while Figure 6 captures the crack patterns with failure modes for all tested girders.

![Figure 5](image_url)  
**Figure 5.** Load-crack width relationships of the tested girders.
3.4. Effect of Study Parameters

The main parameters that were considered in the present study are: existence of splices, type of concrete in the splice region; and strengthening by CFRP bars. The effects of the above parameters on the experimental results is discussed in details in this section.

3.4.1. Effect of Existence of Splices

Splice regions are the most fragile parts of the spliced girder owing to the high stresses developed during loading process. From the experimental results, the ultimate load of the continuous spliced girder
CSB₂ was smaller than that of the control girder CB₁ by 33.1% with higher midspan deflections. In addition, first flexural cracking load in the girder CSB₂ was smaller than that of the girder CB₁ by 42.9%. The existence of splice regions can be considered as the main reason for the decrease of the ultimate load of the spliced girder CSB₂ which was also mentioned in the previous work [4, 9]. Additionally, the failure mode of the girder CSB₂ was brittle by direct shear in comparison with the ductile failure of the control girder CB₁. The load deflection response of the girder CB₁ (without splices) and the spliced girder CSB₂ are shown in Figure 7.

![Figure 7. Load-deflection curves of girders CB₁ and CSB₂.](image)

3.4.2. Effect of Steel Fiber Concrete (SFC)

In the spliced girder CSB₃, steel fiber concrete (SFC) has been used in the splice regions instead of the normal concrete (NC) that used in girder CSB₂ to enhance the structural behavior of the spliced girder. Experimental test results indicate that using of steel fiber concrete (SFC) in the splice regions showed an increase in the ultimate load of the spliced girder of 12.3% if compared with normal concrete (NC). The first flexural crack was also delayed to a load of 60 kN in the splice girder CSB₃, while the first flexural cracking load in the spliced girder CSB₂ was 40 kN. However, the failure mode of the girder CSB₃ still brittle. Figure 8 shows the load deflection response of girders CSB₂ and CSB₃.

![Figure 8. Load-deflection curves of girders CSB₂ and CSB₃.](image)
3.4.3. Effect of Strengthening by CFRP Bars

The spliced girders CSB4 and CSB5 have been strengthened using side near surface mounted carbon reinforced polymer CFRP bars. The spliced girder CSB4 was strengthened using twelve horizontal CFRP bars of size (Ø 6mm) while four CFRP bars of size (Ø 10 mm) inclined by 30° were used to strengthen the spliced girder CSB5. This technique was used to improve the capacity of the spliced girders. Experimentally, the results declared that the ultimate load of the spliced girders CSB4 and CSB5 increased by 69.1% and 98.1%, respectively, in comparison with the non-strengthened spliced girder CSB2 with lower deflection values. Furthermore, the first flexural crack was delayed to a load of 90 kN for the spliced girder CSB4, likewise; the first flexural crack appeared in the spliced girder CSB5 at a load of 60 kN while the non-strengthened spliced girder CSB2 had a flexural cracking load of 40 kN. Despite the great increase in the ultimate load for the spliced girders CSB4 and CSB5, the failure mode of both girders still brittle owing to the direct shear and the nature of the FRP reinforcements. Figure 9 shows the load deflection response of the spliced girders CSB2, CSB4, and CSB5.

Figure 9. Load-deflection curves of girders CSB2, CSB4, and CSB5.

4. Conclusions.

The current study presents an experimental investigation of the behavior of the continuous spliced girders. Based on the obtained results, the following conclusions are presented:

- Splicing of continuous girders without strengthening decreased the ultimate load capacity of the girder by about 33.1% with higher midspan deflections.
- Using of steel fiber concrete of 1% volume fraction at joints raised the spliced girder load capacity by 12.3% if compared with normal concrete.
- Strengthening of a spliced girder using twelve horizontal side near surface mounted CFRP bars increased the load capacity of the spliced girder by 69.1% with lower deflections.
- Strengthening of a spliced girder using four side near surface mounted CFRP bars inclined with 30° increased the load capacity of the spliced girder by 98.1% with lower deflections.
- In the non-strengthened spliced girders, interface cracks were wider than flexural or shear cracks and happened earlier.
- Strengthening of a spliced girder using side near surface mounted CFRP bars delayed the formation of the interface and flexural cracks.
- Despite the additional capacity provided by the strengthening, the failure mode of the spliced girders was brittle.
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