EXPERIMENTAL AND THEORETICAL INVESTIGATIONS FOR BEHAVIOR OF PRECAST CONCRETE GIRDERS WITH CONNECTIONS

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Abstract

This research presents experimental and theoretical investigation of 15 reinforced concrete spliced and non-spliced girder models. Splices of hooked dowels and cast in place joints, with or without strengthening steel plates were used. Post-tensioning had been used to enhance the splice strength for some spliced girders. The ANSYS computer program was used for analyzing the spliced and non-spliced girders. A nonlinear three dimensional element was used to represent all test girders. The experimental results have shown that for a single span girder using steel plate connectors in the splice zone has given a sufficient continuity to resist flexural stresses in this region. The experimental results have shown that the deflection of hooked dowels spliced girders is greater than that of non-spliced girder in the range of (17%-50%) at about 50% of the ultimate load which approximately corresponds to the serviceability limit state and the ultimate loads is less than that of non-spliced girder in the range of (12%-52%). For other spliced girders having strengthening steel plates at splices, the results have shown that the deflection of the spliced girder is less than that of non-spliced girder in the range of (2%-20%) at about 50% of the ultimate load and the ultimate loads for spliced girder is greater than that of non-spliced girder in the range of (1%-7%). The post-tensioned concrete girders have shown a reduction in deflection in the range of (26% - 43%) at a load of 50% of the ultimate load as compared with that of ordinary girders. Moreover, post-tensioning increases the ultimate loads in the range of (70% - 132%). The results obtained by using the finite element solution showed a good agreement with experimental results. The maximum difference between the experimental and theoretical ultimate loads for girders was in the range of (3-11%).

الخلاصــــــة

تقدم هذة الدراسة بحثا عمليا وتحليليا لخمسة عشر رافدة خرسانية موصولة وغير موصولة.حيث كانت التوصيلات بواسطة الحديد المعكوف وبواسطة التقوية بصفائح الحديد اضافة الى استعمال قوى الشد اللاحق (post-tensioning) لتعزيز مقاومة التوصيلات في بعض الروافد. للتقصي التحليلي لسلوك الروافد المفحوصة مختبريا ، تم استخدام عناصر محددة ثلاثية الابعاد تتصرف لاخطيا وذلك بالاستعانة ببرنامج العناصر المحددة (**ANSYS**) لاجراء تحليل جميع الروافد. لقد تم اجراء مقارنة بين النتائج العملية والتحليلية للروافد الموصولة وغير الموصولة وقد بينت النتائج انه تحت تاثير 50% من الحمل الاقصى والذي يقترب من الحمل الخدمي للمنشا ان الهطول في الروافد الموصولة بواسطة الحديد المعكوف يزيد بنسبة (17-50%) والحمل الاقصى لها يقل بنسبة (12-50%) عما في الروافد الغير موصولة. اما الروافد الموصولة بالصفائح الحديدية فكان الهطول فيها عند 50% من الحمل الاقصى يقل بنسبة (2-20%) عما هو في الروافد غير الموصولة، اما الحمل الاقصى فيزداد بنسبة (1-7%). ان النتائج العملية اظهرت انه باستخدام قوى الشد اللاحق سوف يقل الهطول بحيث يكون عند تاثير 50% من الحمل الاقصى اقل بنسبة (26-43%) من الهطول للروافد غير الموصولة التي لم تخضع لقوى الشد اللاحق، والحمل الاقصى اكثر بنسبة (70-132%). اظهرت النتائج بشكل عام حصول توافق جيد بين نتائج العناصر المحدودة مع النتائج المختبرية وكان اكبر فرق في التحمل الاقصى للروافد يتراوح ضمن (3-11%).

Keywords: ANSYS; connections; nonlinear finite element analysis; nonlinear behavior; precast; reinforced concrete girders.

INTRODUCTION

Precast concrete construction have been getting popular and being widely applied in construction sector today. Properly detailed and constructed joints in precast concrete construction of bridges are essential to the success. The joints should be designed to transmit all forces and, furthermore, be feasible to construct under actual job site conditions. Since visible joints affect the appearance of the bridge structure, the well designed joints will enhance the structure esthetics. Connections are either wide or match cast. Depending on their width, they may be filled with cast-in-place concrete or grout. Dry match cast joints (do not employ the use of a cement-type material between the joined components) are not recommended (AASHTO, 2005).

In general precast concrete connections can be classified to continuous connections refer to connections where both moment and shear are transferred through the joints. Connections that just transfer shear act as a hinge between precast members. Continuous connections could be further divided to connections with post-tensioning tendons and those without. For connections with post-tensioning tendons, or conventional reinforcing, connections could be match-cast or non-match cast (Jimin Huang, April 2008).

Constructing concrete bridges of spans exceeding a certain length and/or weight is constrained by the contemporary capacities of precast concrete producers, as well as the shipping capacity limitations of the highways. Thus, all bridges with spans exceeding the practical limits have to be designed with structural steel plate girders. However, due to various reasons, there has been a tendency to increase precast concrete bridge spans. This presents a real challenge for researchers and designers in the field to find a technically feasible, economic, and aesthetic solution that allows for extending span capacity.

The conception, development and world wide acceptance of segmental construction in the field of prestressed concrete bridges, represents one of the most interesting and important achievements in civil engineering during the past thirty years. Instead of segmental construction method of bridge girders, splicing of precast segments can be carried out at some suitable locations especially at inflection points.

EXPERIMENTAL WORK

In the present study spliced girder models are fabricated by connecting two or three pieces to obtain the required length of the test girder. Post-tensioning is to be used to reinforce the connection between the girder segments. In addition a more rational method has also been used to reinforce the segments by using steel plates in the connection with the hooked dowels at each splice and then post-tensioning the overall girder by reinforcement.

The focus of this research is to investigate the splicing effects on behavior of precast concrete girders. The experimental program of the present study consists of testing girders. Fifteen of reinforced concrete spliced and non-spliced girder models are tested up to failure. The test girders are classified into four groups as given in Table 1. These groups differ by the following factors.

- The case of supporting
- Type of splice
- No. of Splices
- Position of Splices
- If there is or not post-tensioning

The Spliced girder connections were made with conventional reinforced and with mechanical splices. Details of the test girders are shown in Figures 1to 15.

Table 1 Description of test girders

Group	No.	Girder designation	Thickness and position of steel plate	Position of splices
1st Simply supported single splice No post-tensioning	1	B1-1	No splice (reference girder)	---
	2	B1-2	Hooked dowel only	Mid
	3	B1-3	2 mm bottom	Mid
	4	B1-4	4 mm bottom	Mid
	5	B1-5	4 mm top & bottom	Mid
	6	B1-6	4 mm Box	Mid
2nd Simply supported with two splices	7	B2-2	Hooked dowel only	Quarter
	8	B2-3	2 mm bottom	Quarter
	9	B2-4	4 mm bottom	Quarter
	10	B2-5	4 mm top & bottom	Quarter
3rd Simply supported - single splice with post-tensioning	11	B3-1	No splice (reference girder)	---
	12	B3-2	Hooked dowel only	Mid
	13	B3-3	4 mm bottom	Mid
4th Two continuous span with single splice in each span	14	B4-1	No splice (reference girder)	---
	15	B4-2	Hooked dowel only	Inflection points

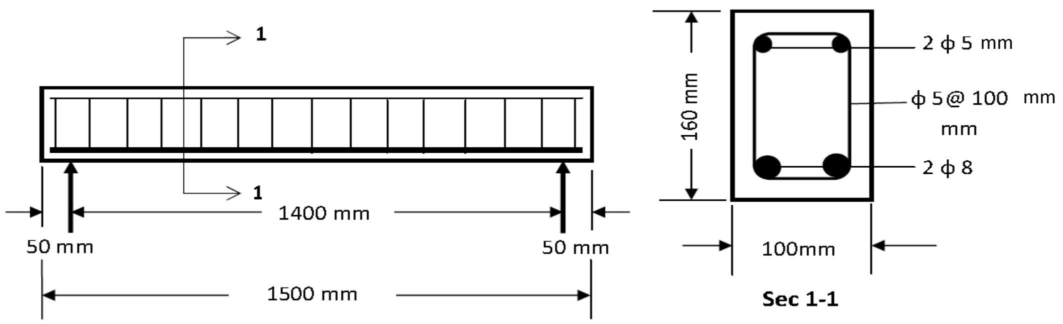


Fig. 1 Girder B1-1 details (first tested girder)

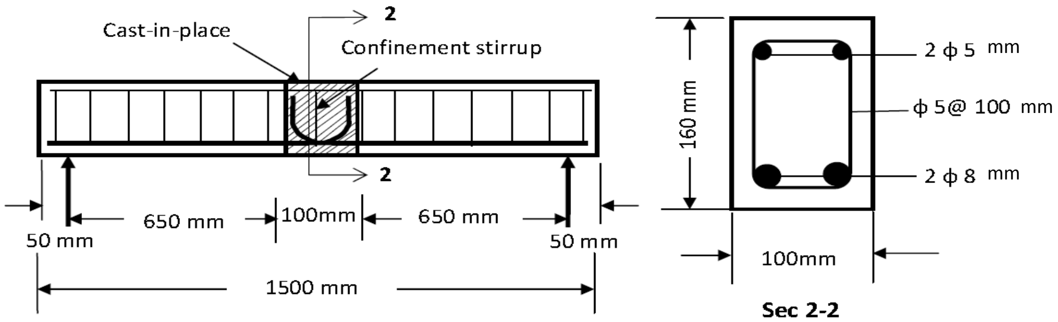


Fig. 2 Girder B1-2 details

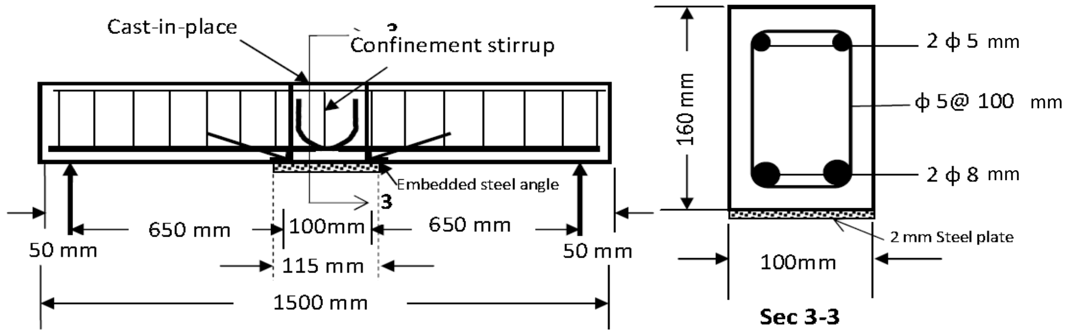


Fig. 3 Girder B1-3 details

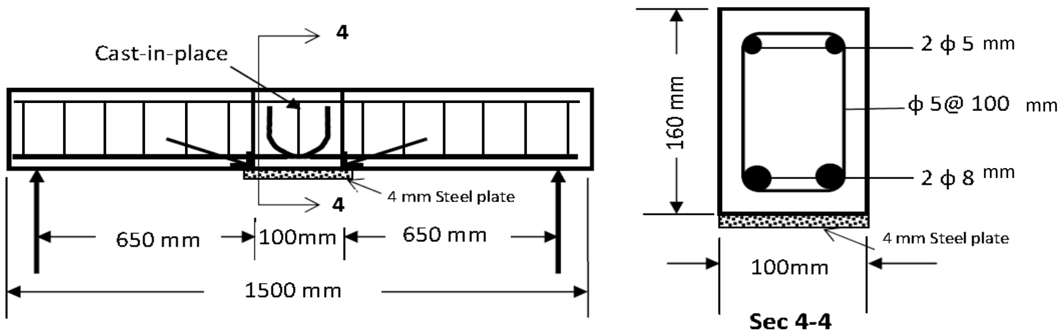


Fig. 4 Girder B1-4 details

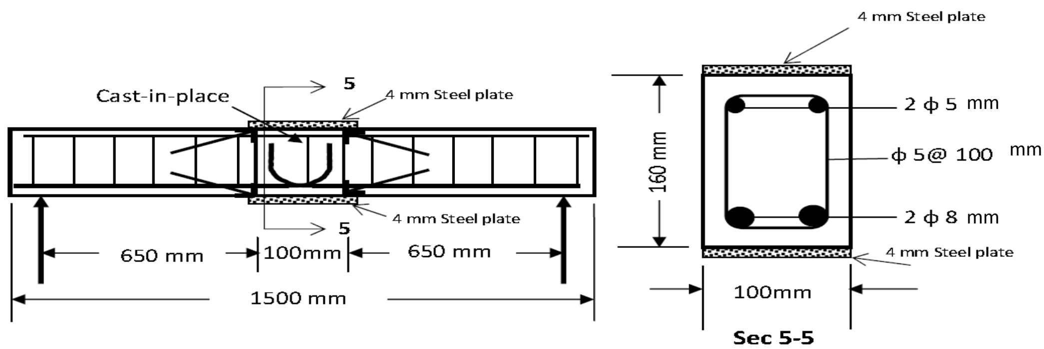
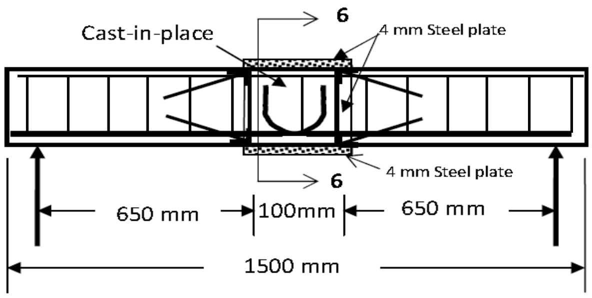


Fig. 5 Girder B1-5 details



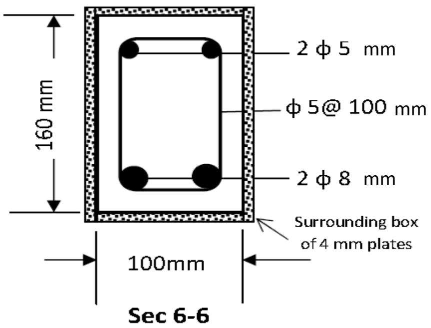
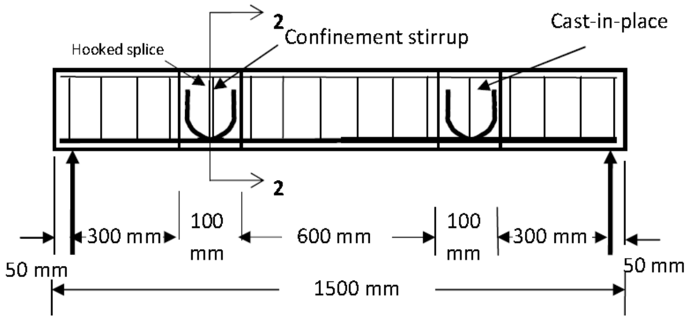


Fig. 6 Girder B1-6 details



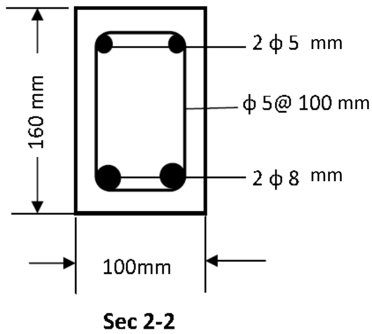
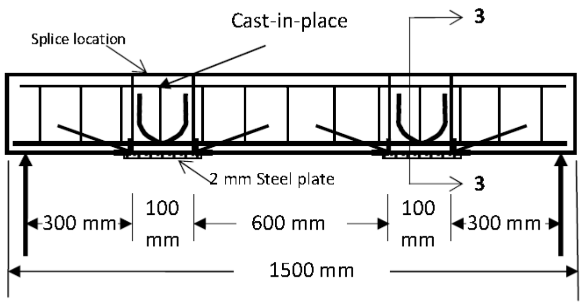


Fig. 7 Girder B2-2 details



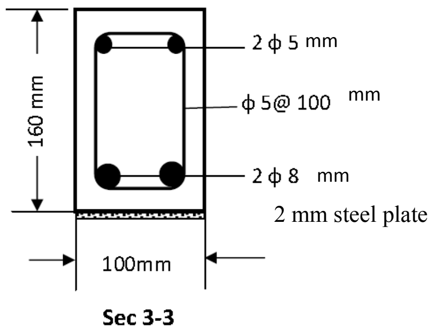
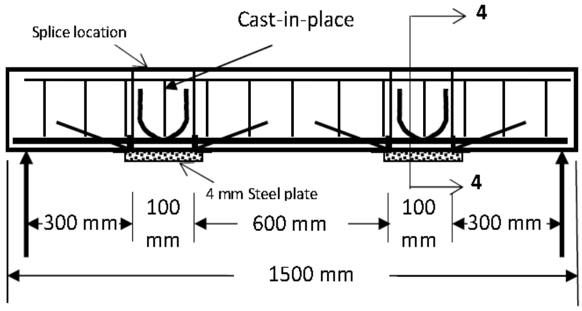


Fig. 8 Girder B2-3 details



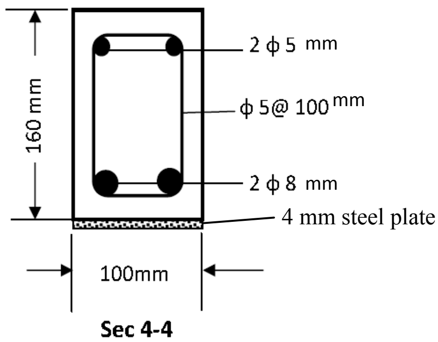


Fig. 9 Girder B2-5 details

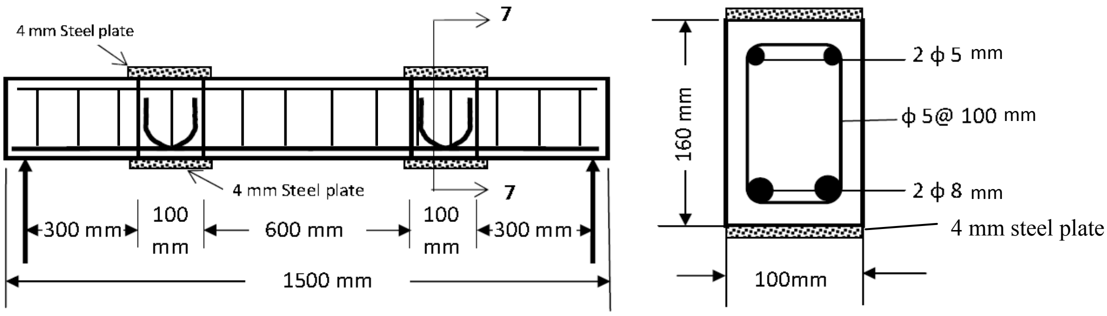
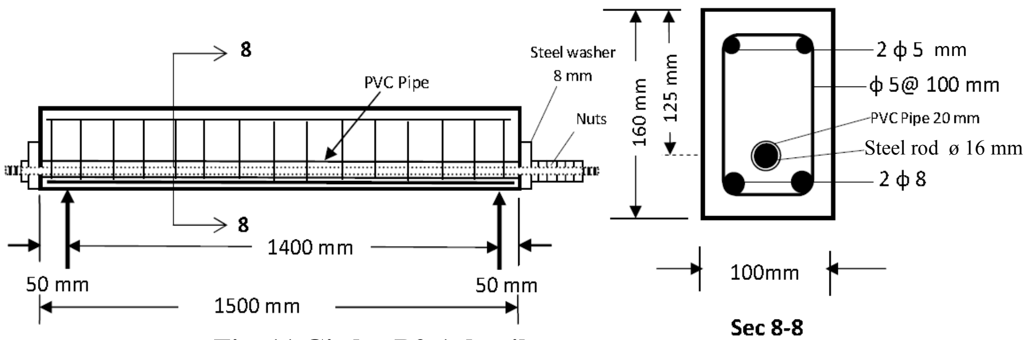


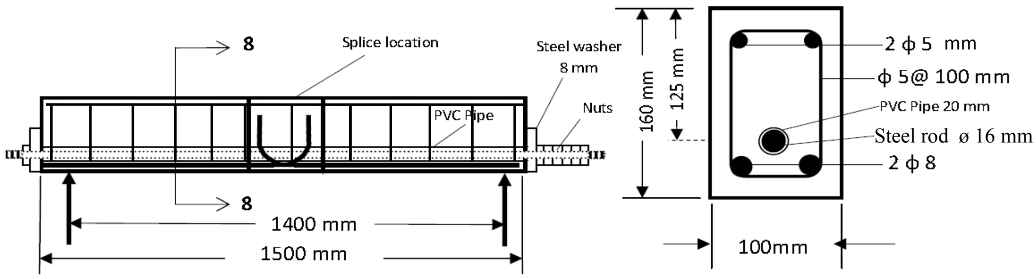
Fig. 10 Girder B2-5 details

Sec 7-7



Sec 8-8

Fig. 11 Girder B3-1 details



Sec 8-8

Fig. 12 Girder B3-2 details

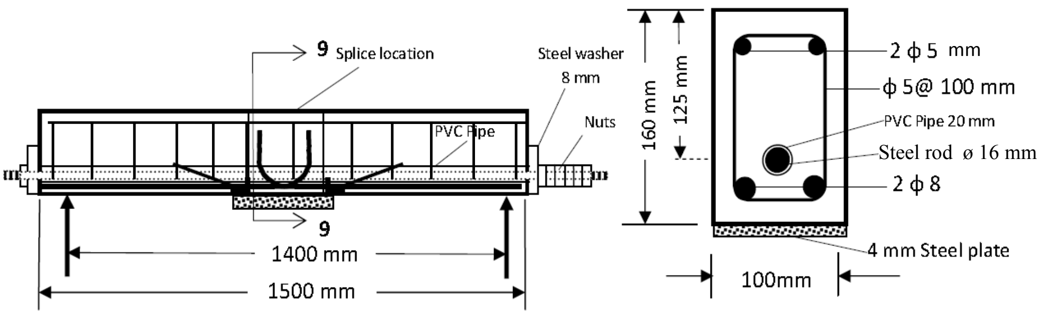


Fig. 13 Girder B3-3 details

Sec 9-9

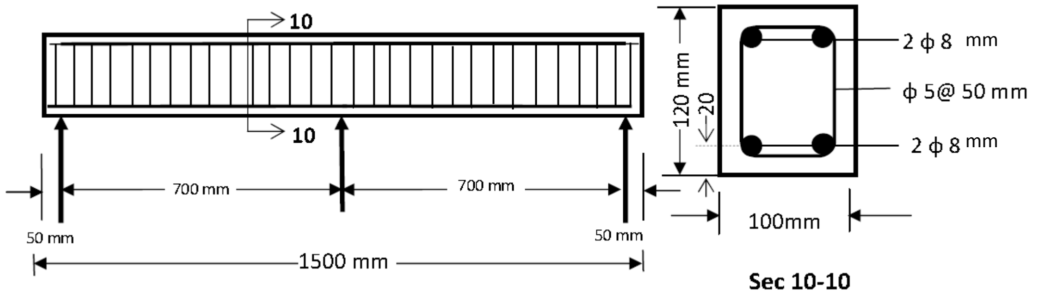


Fig. 14 Girder B4-1 details

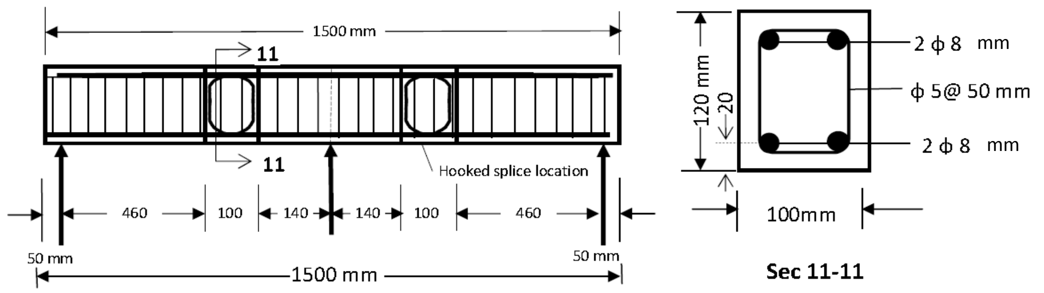


Fig. 15 Girder B4-2 details (last tested girder)

Properties of Concrete

The compressive strength test of concrete was carried out in accordance with BS1881-116 using (150mm) cubes loaded by the universal compressive machine that were used to determine the compressive strength. By using the relationships between the cubes and the cylinder strengths ($f_c^{'} = 0.8 f_{cu}$) (ACI 318m-2008)

The results are given in Table 2.

Properties of Steel Reinforcement

Tensile test of steel reinforcement was carried out on (ϕ 8mm) hot rolled, deformed, mild steel bars employed as tension reinforcement. Also, the test included testing of (ϕ 5mm) and (ϕ 16mm) smooth mild steel bars, (5 mm) used as stirrups and (16 mm) used as post-tensioning reinforcement. Table 3 gives the results of tensile test for bars (5, 8 and 16mm).

Details of Stiffening Steel Plates

The used steel plate was of two thicknesses 2mm and 4 mm welded on angles embedded at the ends of the two spliced segments as shown in Figure 16.

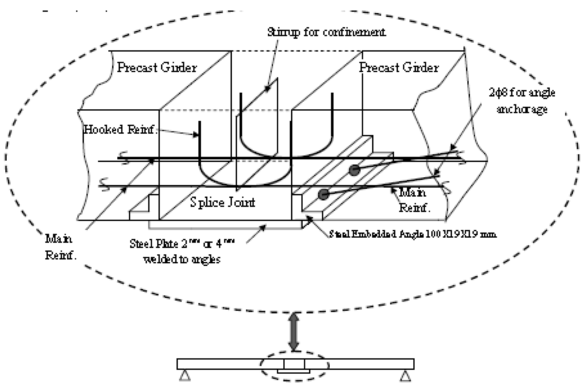


Fig. 16 Details of Steel Splices

Table 2 Compressive Strength of Concrete

Group	Girder No.	Compressive Strength*			
		f_{cu}, MPa (cube)		f'_c, MPa (cylinder)	
		Girder Pieces, 56 days (Testing Age)	Splices, 28 days (Testing Age)	Girder Pieces	Splices
	B1-1				
	B1-2				
1st	B1-3	43.9	37.5	35.1	30.0
	B1-4				
	B1-5				
	B1-6				
	B2-2				
2nd	B2-3	43.5	37.2	34.8	29.8
	B2-4				
	B2-5				
3rd	B3-1	44.3	37.7	35.4	30.2
	B3-2				
	B3-3				
4th	B4-1	42.7	36.8	34.2	29.4
	B4-2				

Table 3 Properties of Steel Reinforcement

Nominal Diameter (mm)	Measured Diameter (mm)	Yield Stress* (MPa)	Ultimate Stress (MPa)
5	5.00	282	426
8	8.08	503	719
16	16.00	346	486

*Each value is an average of three specimens (each 50 cm. length).

Note: modulus of elasticity of steel = 200 GPa (Assumed)

Post-Tensioning of Girders

No prestress bed is available in the structures laboratory; hence a proposed method of post-tensioning is suggested in the present study. The test girders of the third group (B3-1, B3-2, and B3-3) have been cast with embedded 20 mm (P.V.C) pipe. After 56 days of casting the segments, a (ϕ 16 mm) steel bar was inserted inside the (P.V.C) pipe and then was post-tensioned by a torque spanner to 0.76 f_y *of* the bar (263 MPa). An extensometer of 100mm gauge length was adopted to measure the strain in the post-tensioning bar at one of its ends.

Although this method of post-tensioning is not acceptable in practice since the conventional steel reinforcement is not adequate in pre-tensioning or post-tensioning as compared with high strength tendons. However the use of ordinary (conventional) reinforcement in the present study can be considered as an acceptable simulation for post-tensioning. This is because the post-tensioned girders were tested within minutes after post-tensioning and a measured bar strain (bar post-tension) was developed and was found to be effective in enhancing the spliced and non-spliced girders strength.

Loading

Girders (B1-1) to (B3-3) which were simply supported have been loaded with two concentrated loads at third points. While girders (B4-1) and (B4-2) which were two continuous spans have been loaded with single concentrated load at the center of each span.

EXPERIMENTAL RESULTS

During the experimental work, the load versus deflection at specified points were recorded for each test girder. Also, cracking and ultimate loads values were recorded as well as the concrete surface strains at many locations across girder depth. Results were studied in terms of:

1. Effect of Splicing Method

There are many different splicing methods for the girders in the three groups mentioned before. The load-deflection curves of spliced girders versus that of the non-spliced girders are shown in Figures 17 to 20. Deflection of the girders was measured at mid-span for each girder.

It is shown for dowels splicing method that the spliced girders (B1-2, B2-2, and B3-2) have more deflection (less stiffness) than that of the non-spliced girders (B1-1, and B3-1). At about 50% of the ultimate loads which corresponds to the serviceability limit state the deflection of the dowels spliced girders is greater than that of the non-spliced girders in the range of (17%-50%). While, in other splicing methods (except dowels method) the spliced girders have less deflection (more stiffness) than that of the non-spliced girders, and at about 50% of the ultimate load the deflection of the spliced girders is less than that of the non-spliced girders in the range of (2%-12%).

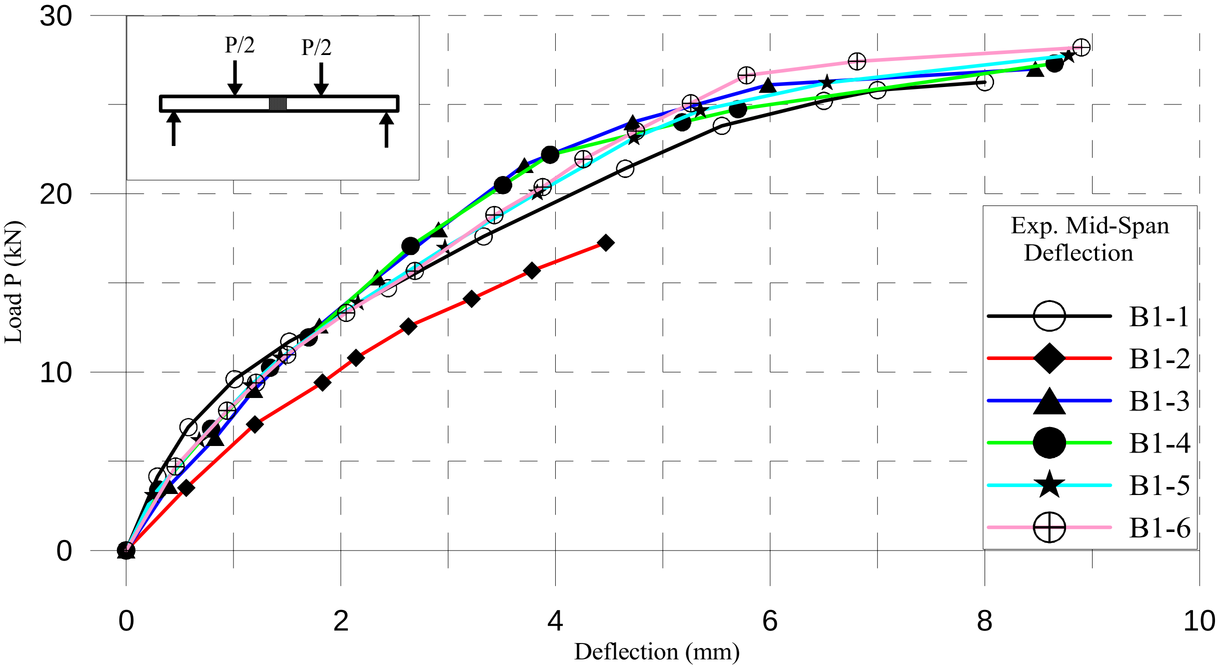


Fig. 17 Load- Deflection Relationship at mid-span for girders first group

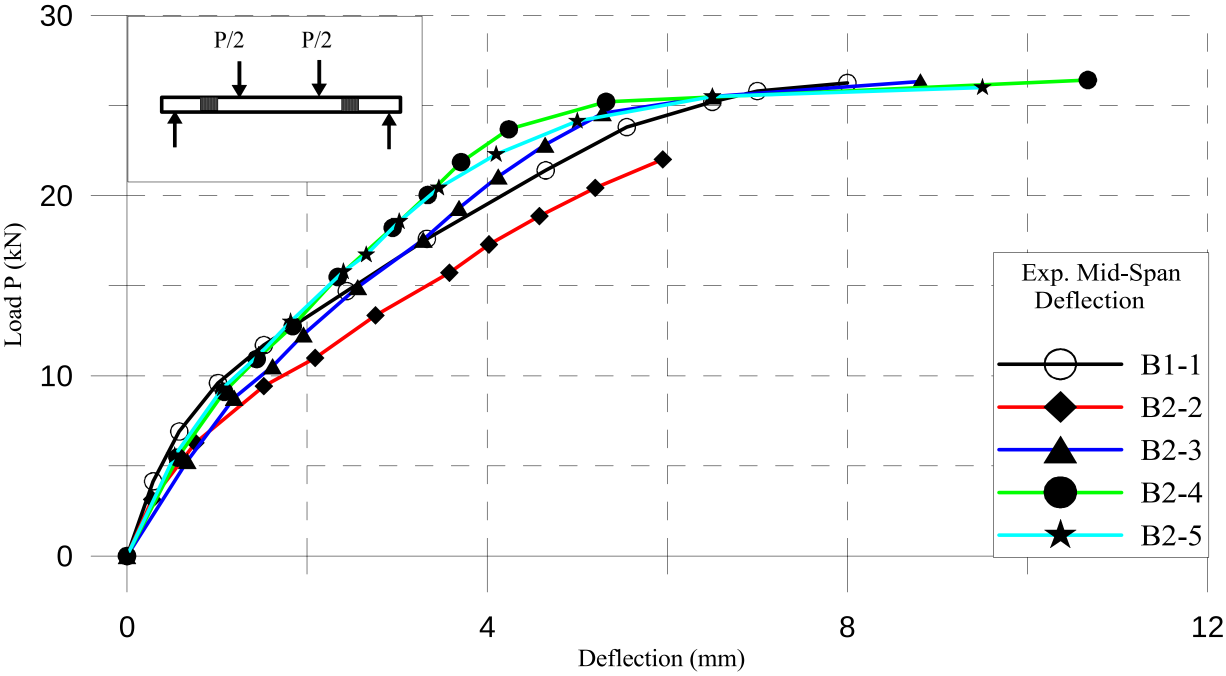


Fig. 18 Load- Deflection Relationship at mid-span for girders of secondgroup

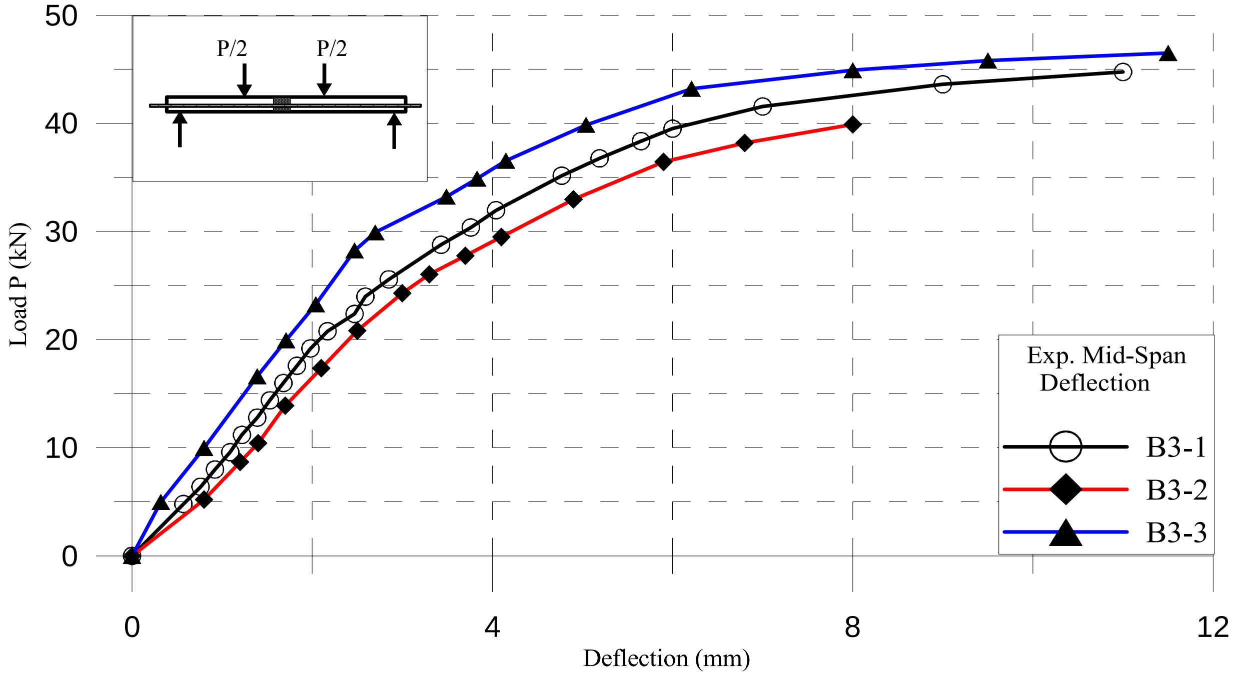


Fig. 19 Load- Deflection Relationship at mid-span for girders of third group

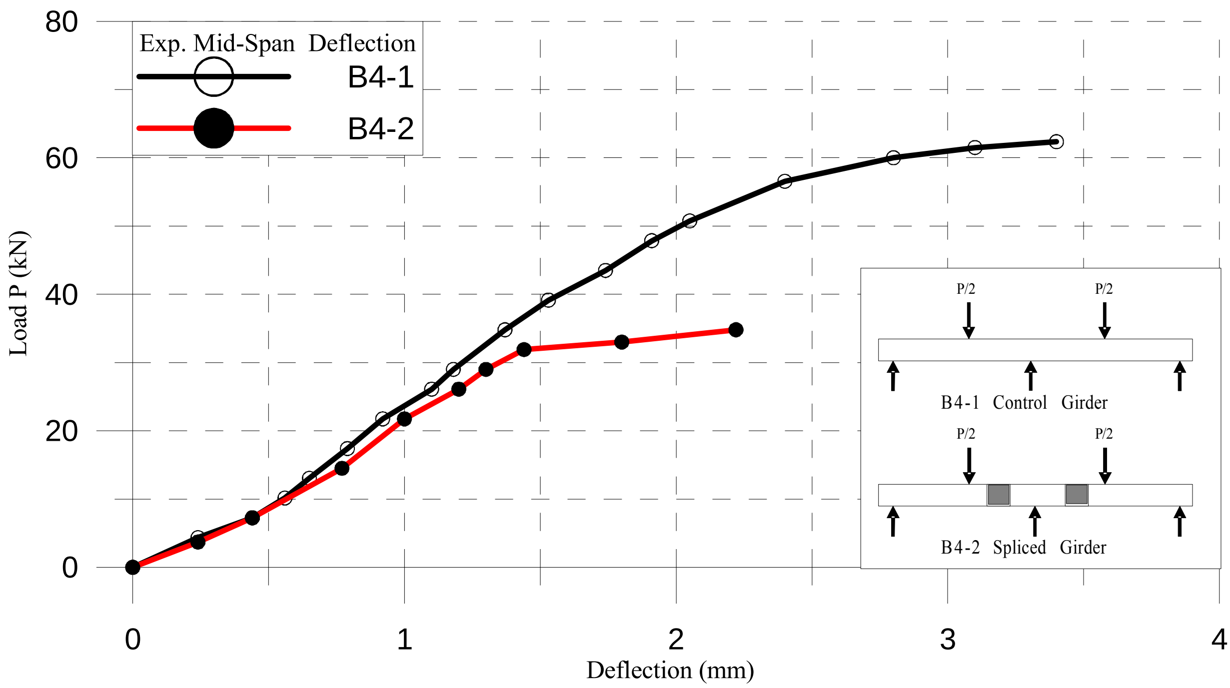


Fig. 20 Load- Deflection Relationship under load point for girders of fourth group

2.Effect of Splices Number

Girders (B1-2) to (B1-5) are two pieces spliced girders and (B2-2) to (B2-5) are three pieces spliced girders. It found from the experimental results that the deflection of spliced girder at 50% of the ultimate value for the two pieces girders did not differ by more than 10% from that of the three pieces girders.

The ultimate load for the dowels spliced two pieces girder (B1-2) is less than that of the three pieces girder (B2-2) by 27%. The reason of this was that the maximum moment of (B2-2) is not near the splice location as in (B1-2). The ultimate loads for the spliced girders stiffened by steel plates did not differ by more than 8%. This indicated that the number of pieces has slight effect on the ultimate load, Figures 21 to 24.

3.Effect of Post-Tensioning

Girders (B1-1) to (B1-3) and also girders (B3-1) and (B3-3) are of spliced and non-spliced types, but (B3-1) to (B3-3) contain one post-tensioned ordinary mild steel bar. It is found from experimental results that post-tensioning reduce the deflection in the range of (26%- 43%) at a load of 50% of the ultimate value. Moreover, post-tensioning increase the ultimate loads in the range of (70%-132%). This indicated that the post-tensioning has a great effect on the strength of the girders especially for that of the dowels splice type, Figures 25 to 27.

Figure 28 shows that the load-deflection curve of non-spliced not post-tensioned girder (B1-1) compared with the dowels spliced post-tensioned girder (B3-2). Result of deflection at 50% of ultimate value for (B3-2) was less than that for (B1-1) by about 24% and the ultimate load was increased by about 50%.

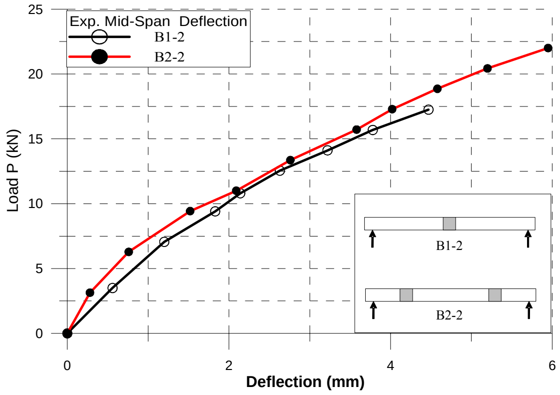


Fig. 21 Load- Deflection Relationship at mid-span for girder (B1-2) and (B2-2)

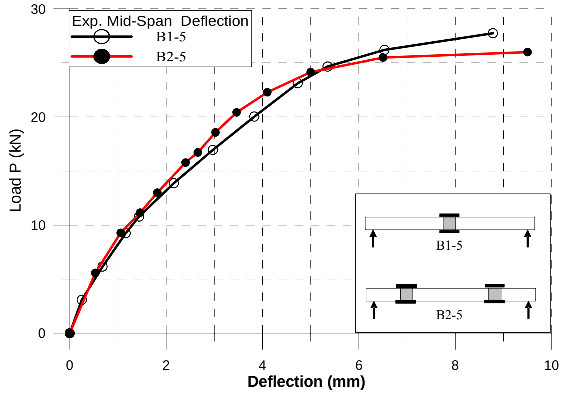


Fig. 24 Load- Deflection Relationship at mid-span for girder (B1-5) and (B2-5)

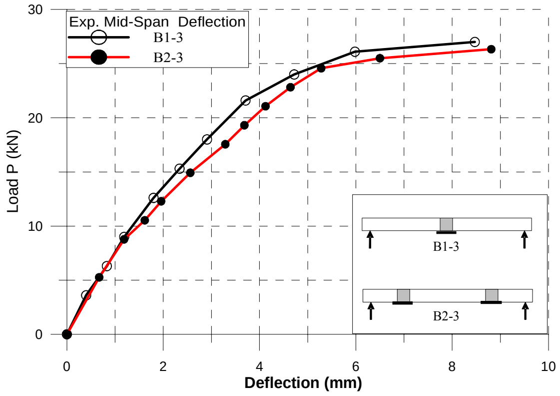


Fig. 22 Load- Deflection Relationship at mid-span for girder (B1-3) and (B2-3)

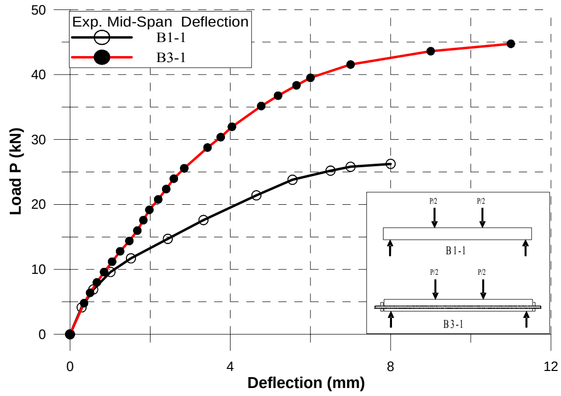


Fig. 25 Load- Deflection Relationship at mid-span for girder (B1-1) and (B3-1)

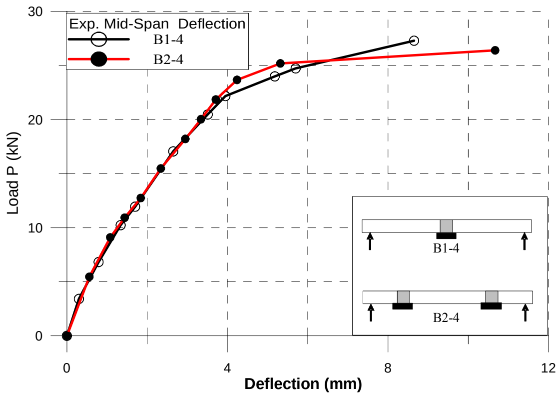


Fig. 23 Load- Deflection Relationship at mid-span for girder (B1-4) and (B2-4)

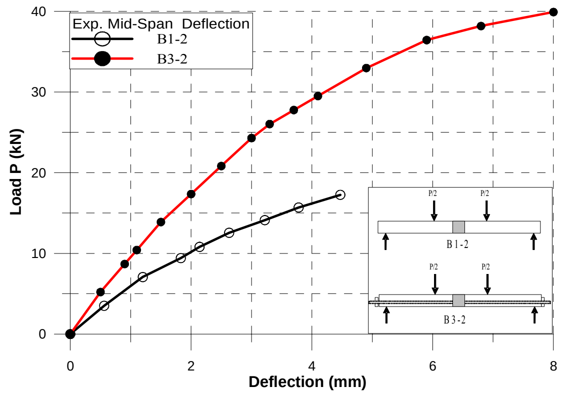


Fig. 26 Load- Deflection Relationship at mid-span for girder (B1-2) and (B3-2)

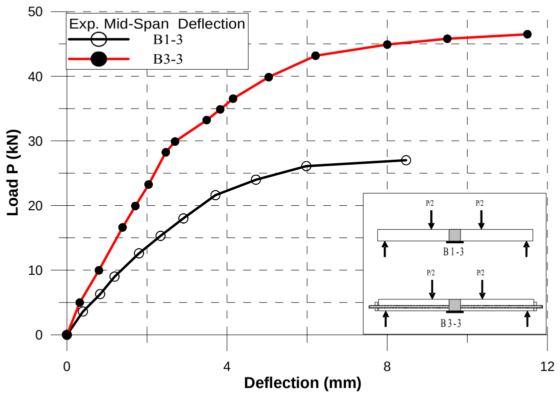


Fig. 27 Load- Deflection Relationship at mid-span for girder (B1-3) and (B3-3)

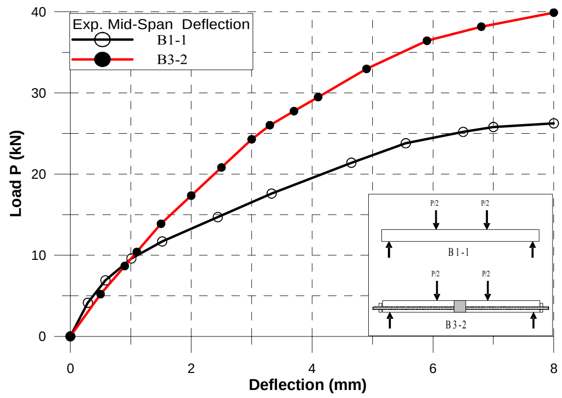


Fig. 28 Load- Deflection Relationship at mid-span for girder (B1-1) and (B3-2)

ANSYS Computer Program

The tested girders are modeled in ANSYS 11-2006 software using the element types (SOLID65, SOLID45, LINK8, SHELL63, CONTA173, TARGE170, and COMPIN39. Due to the advantage of symmetry only a quarter or a half of the girder was modeled and analyzed. This depends on the presence of the post-tensioning force. The girders have two planes of symmetry; one plane of symmetry is the x–y plane cutting girder in halves longitudinally and the other plane of symmetry is the y–z plane cutting girder in halves transversely. Figure 29 shows the adopted quarter of control girder, other girders were modeled by the same procedure. Due to symmetry, only quarter portion of the girder is analyzed and symmetric boundary conditions are placed along the two symmetric planes for groups 1, 2, and 4.

While only one symmetry plane is allowed for one half of the girder in group 3.

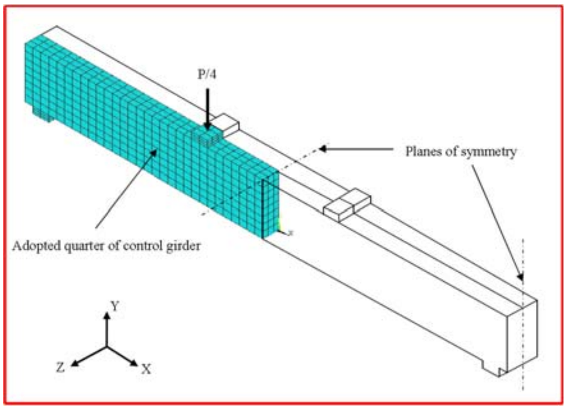


Fig. 29 Quarter of Control Girder

(Group 1 and 2)

1. Load-Deflection plots

The experimental and theoretical load deflection plots for the four groups are presented and compared in Figures 30 to 44. In general, it can be noted from the load-deflection plots that the finite element analyses agree well with the experimental results throughout the entire range of behavior.

2. Ultimate Loads

COMPARISON BETWEEN EXPERIMENTAL AND THEORETICAL RESULTS

Tables 4 to 7 show the comparison between the ultimate loads as obtained from tests and from finite element analysis. The ultimate loads obtained from numerical model agree well with the corresponding values of the experimental (tested) girders. Results of numerical model (FEM) are higher than that of experimental by range within a (11 %).

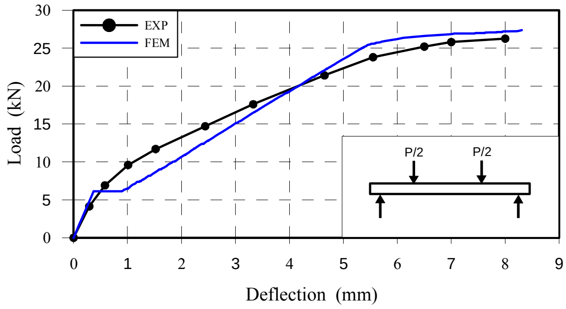


Fig. 30 Girder B(1-1), Load - Deflection varying at mid-span

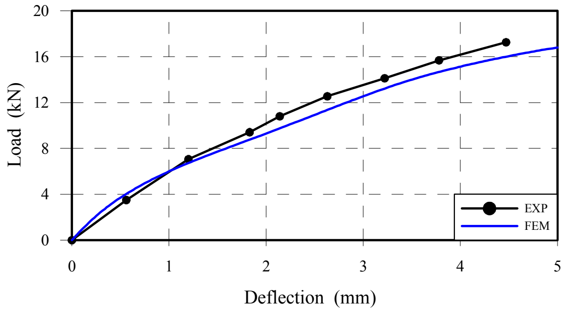


Fig. 31 Girder B(1-2), Load - Deflection varying at mid-span

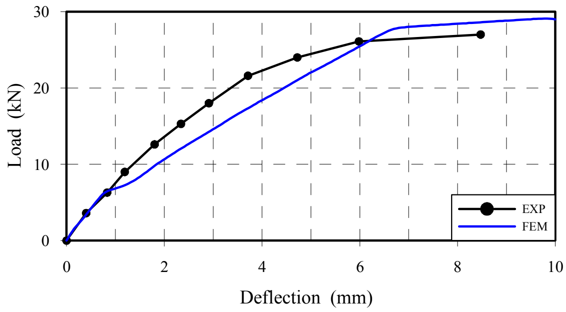


Fig. 32 Girder B(1-3), Load - Deflection varying at mid-span

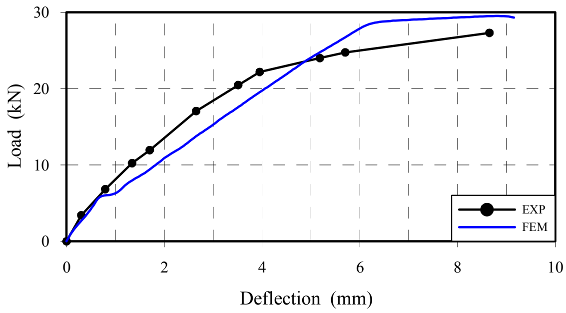


Fig. 33 Girder B(1-4), Load - Deflection varying at mid-span

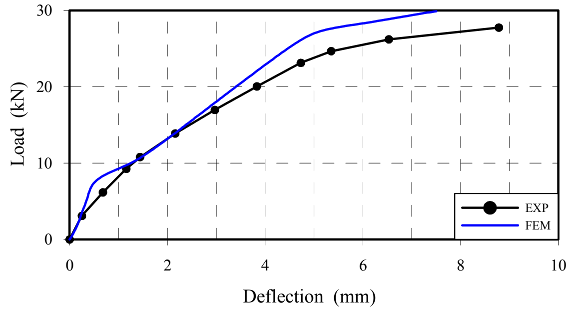


Fig. 34 Girder B(1-5), Load - Deflection varying at mid-span

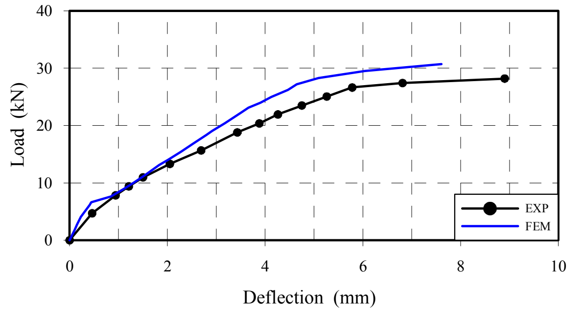


Fig. 35 Girder B(1-6), Load - Deflection varying at mid-span

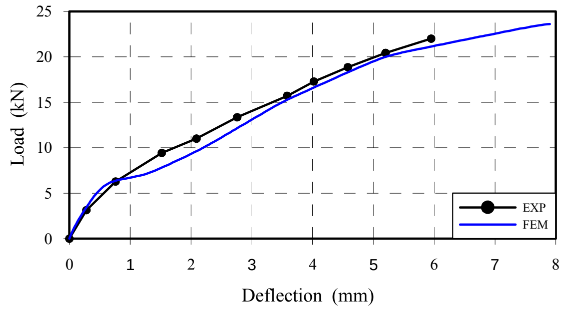


Fig. 36 Girder B(2-2), Load - Deflection varying at mid-span

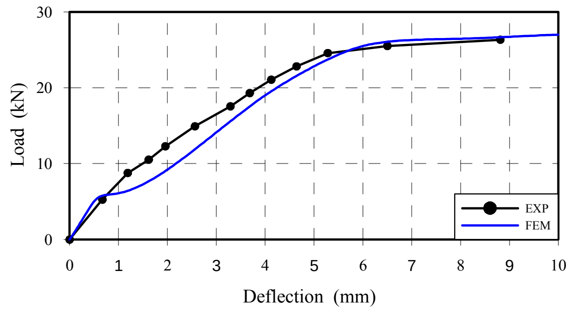


Fig. 37 Girder B(2-3), Load - Deflection varying at mid-span

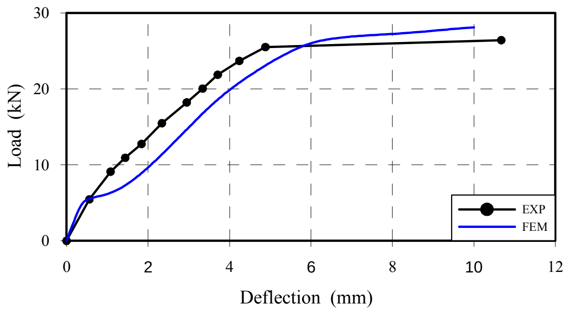


Fig. 38 Girder B(2-4), Load - Deflection varying at mid-span

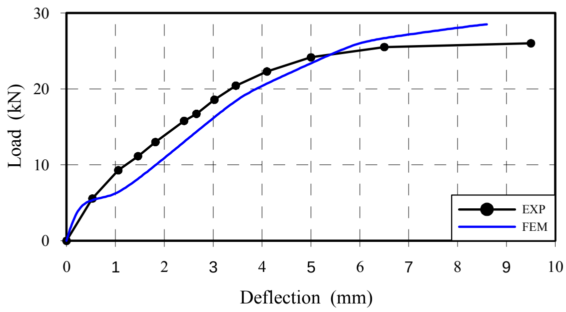


Fig. 39 Girder B(2-5), Load - Deflection varying at mid-span

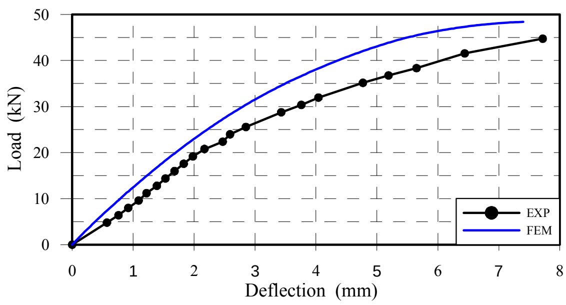


Fig. 40 Girder B(3-1), Load - Deflection varying at mid-span

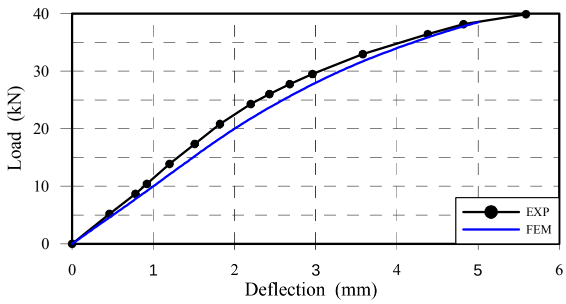


Fig. 41 Girder B(3-2), Load - Deflection varying at mid-span

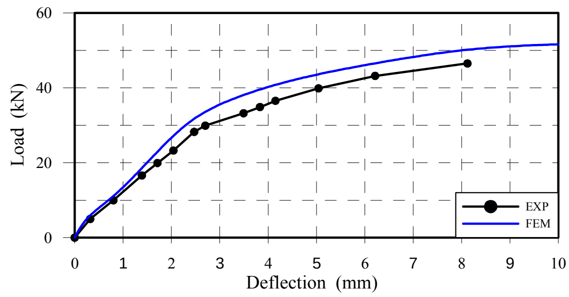


Fig. 42 Girder B(3-3), Load - Deflection varying at mid-span

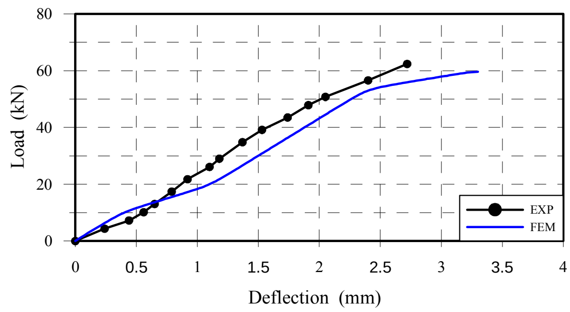


Fig. 43 Girder B(4-1), Load - Deflection varying at load point

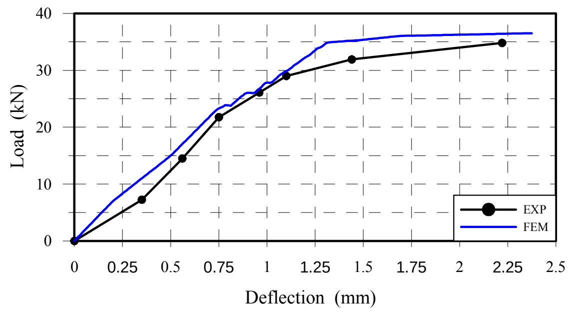


Fig. 44 Girder B(4-2), Load - Deflection varying at load point

Table 4 Comparison between Exp. and FEM Ultimate Loads for First Group

Girder No.	Ultimate Load (kN)		$\frac{(P_u)_{FEM}}{(P_u)_{EXP.}}$
	$(P_u)_{EXP.}$	$(P_u)_{FEM}$	
B1-1	26.25	27.80	1.06
B1-2	17.25	16.78	0.97
B1-3	27.00	29.00	1.07
B1-4	27.30	29.30	1.07
B1-5	27.75	29.90	1.08
B1-6	28.20	30.73	1.09

Table 5 Comparison between Exp. and FEM Ultimate Loads for Second Group

Girder No.	Ultimate Load (kN)		$\frac{(P_u)_{FEM}}{(P_u)_{EXP.}}$
	$(P_u)_{EXP.}$	$(P_u)_{FEM}$	
B1-1	26.25	27.80	1.06
B2-2	22.00	23.60	1.07
B2-3	26.33	27.02	1.03
B2-4	26.41	28.10	1.06
B2-5	26.00	28.50	1.10

Table 6 Comparison between Exp. and FEM Ultimate Loads for Third Group

Girder No.	Ultimate Load (kN)		$\frac{(P_u)_{FEM}}{(P_u)_{EXP.}}$
	$(P_u)_{EXP.}$	$(P_u)_{FEM}$	
B3-1	44.74	48.40	1.08
B3-2	39.90	38.50	0.96
B3-3	46.50	51.60	1.11

Table 7 Comparison between Exp. and FEM Ultimate Loads for Forth Group

Girder No.	Ultimate Load (kN)		$\frac{(P_u)_{FEM}}{(P_u)_{EXP.}}$
	$(P_u)_{EXP.}$	$(P_u)_{FEM}$	
B4-1	62.35	59.63	0.96
B4-2	34.80	36.50	1.05

CONCLUSIONS

The following conclusions can be drawn from the present study:-

1. The percentage changes in mid-span deflections of the spliced girders as compared to the corresponding values of non-spliced girders were in the range of (-20%) - (+50%) at 50% of the ultimate load. The lower bound corresponds to the post-tensioned-single splice girder, spliced at mid-span with steel plate. The upper bound in the above ranges corresponds to the hooked dowel-single splice girder.

At the ultimate load the percentage change ranged between (-44%) and (+32%). The lower bound in the above ranges corresponds to the hooked dowel single splice girder and the upper bound corresponds to the girder with two splices by steel plate.

2.The percentage changes in the ultimate loads of the spliced girders as compared to the corresponding values of non-spliced girders were in the range of (-34%) – (+7%). The lower bound of this range corresponds to the hooked dowel single splice girder, while the upper bound corresponds to the single splice girder spliced with box of plates.

3. The ANSYS nonlinear analysis software proved its accuracy in obtaining results. The discrepancies in deflections between the experimental and ANSYS analysis results were in the range of (3.0% - 20.0%) among the complete load-deflection relationships. The discrepancies in the ultimate loads were in the range of (3.0% - 11.0%).

4. The experimental results have shown that for a single span girder using steel plate in a splicing joint has given a full continuity to resist the flexural stresses in this region. Also using only a single plate at the bottom of splice is quite enough for continuity and strength purposes.

5. The post-tensioning has improved the behavior of hooked splice girder. The post-tensioned concrete girders have shown a reduction in deflection in the range of (26% - 43%) at a load of 50% of the ultimate load as compared with that of ordinary girders. Moreover, post-tensioning increases the ultimate loads in the range of (70% - 132%).

REFERENCES

AASHTO LRFD, Bridge Design Specifications, SI Units, Third Edition, 2005.

ACI 318m-08, American Concrete Institute,(2008) Building Code Requirements for Reinforced Concrete, American Concrete Institute, Farmington Hills, Michigan.

ANSYS Manual, Version (11.0). 2006.

Jimin Huang, “Designing Durable, Reliable Precast Concrete Connections”, April 2008.

Method for determination of compressive strength of concrete cubes, (BS1881-116), British Standards Institute, London, 1983.

NOTATIONS

f_c' cylinder compressive strength of concrete (MPa)

f_{cu} cube compressive strength of concrete (MPa)

f_y yield strength of steel (MPa)

GPa Giga Pascal (GN/m^2)

MPa Mega Pascal (MN/m^2)

P applied force (kN)

P_u ultimate load (kN)

ϕ diameter of reinforcement bar (mm)