Experimental Investigation of Externally Post Tension Composite Ultra High Performance Concrete-Steel Girder

Dr. AbdulMutlib I. Said
Prof.
Larah Riyadh Abdulwahed
Department of Civil Engineering
University of Baghdad/Iraq
Dr.AbdulMutlib.I.Said@coeng.uobaghdad.edu.iq
larh_riyad@yahoo.com

Abstract:-An experimental investigation was carried out to find the beneficial effect of using external prestressing technique in strengthening of an existing composite steel I-girder bridge and ultra-high performance concrete. The experimental work consists of fabricated five composite steel-concrete, non prestressed beam and four externally post-tensioned girders, classified in two groups. All girders were constructed by scaling down an existing prototype composite girder by 1/4 scale factor, were designed according to AASHTO LRFD 2012 standard specification. Each girder was tested as a simply supported beam with span of 3.90m and loaded incrementally up to failure under the action of two point loads. The prestressing force in the girders of group one was (9) Ton applied after the RC deck slab was constructed, while group two was (7) Ton. The variables include values of the eccentricity, with or without deviator and the magnitude of the applied prestressing force. Results show appreciable enhancement in load carrying capacity of the externally prestressed girders comparing with that of the unprestressed girder (reference). Group one girders showed a percentage increase in ultimate load of 36.36% and 45.45% for prestressing force of (9) Ton with eccentricity which mean that (location of prestress below bottom face are equal to 80mm and 160mm, respectively) while group two show that this percentage increase are 38.63% and 47.72% for prestress force of (7) Ton with eccentricity of (120mm and 200mm, respectively).

Keywords: Composite section, External Prestress, Ultra High Performance Concrete (UHPC)

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1. Introduction

The technique of external prestressing in strengthening of existing bridge has been used in several countries since the1950s [6]. It has been found to provide an economical and efficient solution for wide range of bridge types. The method is developing in popularity because of the minimal disruption to traffic flow and the speed of installation. The principle of pre-stressing is the application of axial load with hogging bending moment to increase the flexural capacity of beam and improve the cracking performance. It can also have beneficial effect on shear capacity [1]. Externally prestressed composite beams have been used in buildings and bridges. Composite beam prestressed with high strength external tendons have confirmed many advantages as compared with ordinary composite beam. Both continuous and simple beams can be prestressed [2]. Improvements in the science of concrete materials have led to the development of new class of cementations composite, namely ultra-high performance concrete (UHPC). The durability and mechanical properties of UHPC make it ideal candidate for use in developed new solutions about highway infrastructure deterioration, replacement, and repair [9]. The containment high-rise building, bridges of long span and marine structure can be considered as the best option for utilizing such kind of concrete. Therefore, reactive powder concrete (RPC) classified as (UHPC) [4], [5], [7], [8], & [10].UHPC refers to material with cement matrix and characteristic compressive strength of 100 MPa, possible attaining more than 200 MPa.

2. Experimental Program

2.1 Manufacturing of the models

The experimental tests were conducted on model simply supported steel – concrete composite girders. All models had similar dimensions and in fact they were selected to be 1/4 scale of the prototype composite bridge girder. The detailed calculations of some important properties of the model girder section such as the location of the elastic neutral axis (C.G. position), moment of inertia with respect to the neutral axis I.N.A. To meet the requirements needed in the experimental investigation of the present research, the properties of the model girder section are given in **Table. 1** and shown in **Fig. 1**.

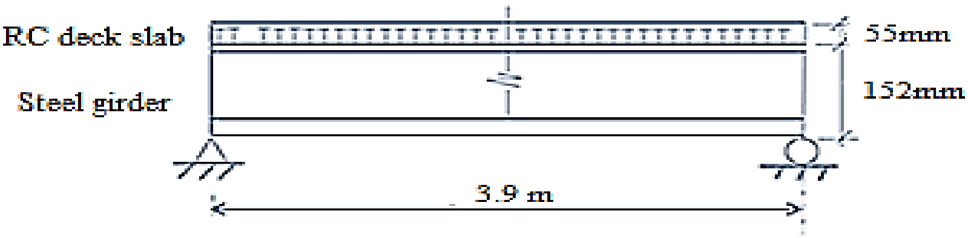
Table. 1 Properties of the Model Girders Equivalent Steel Section

member	Size (mm)	A (mm^2)	Y_b (mm) centroid to bottom	$A \times Y_b$ (mm^3)	I_o (mm^4)	d (mm) C.G to NA	Ad^2 (mm^4)
section HEA 160							
Top flange	160 × 9	1440	147.5	212400	9720	19.87462	568800.568
Web	6 × 134	804	76	61104	1203052	-51.6254	2142804.826
Bottom flange	160 × 9	1440	4.5	6480	9720	-123.125	21830198.18

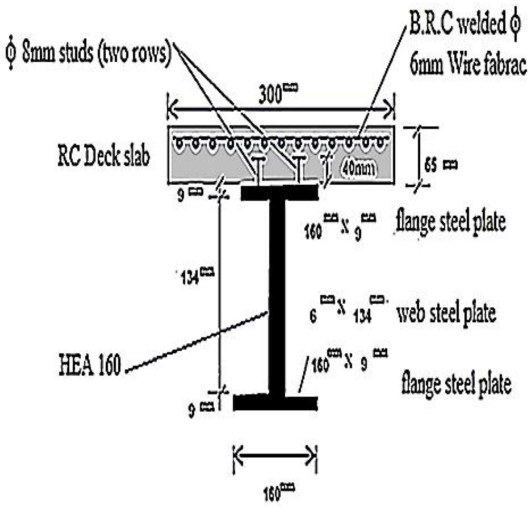
Slab concrete equivalent to steel 300/n=66.6	66.6×55	3666.3	179.5	658100.9	924213.1	51.87462	9865925.039
Summation		7350.3		938084.9	2146705.1		34407728.61

$\overline{Y}_b$=127.6254mm , I_{NA}= 36554434 mm^4 , $n = \frac{E_S}{E_C} = 4.5$, Slab Width = 300

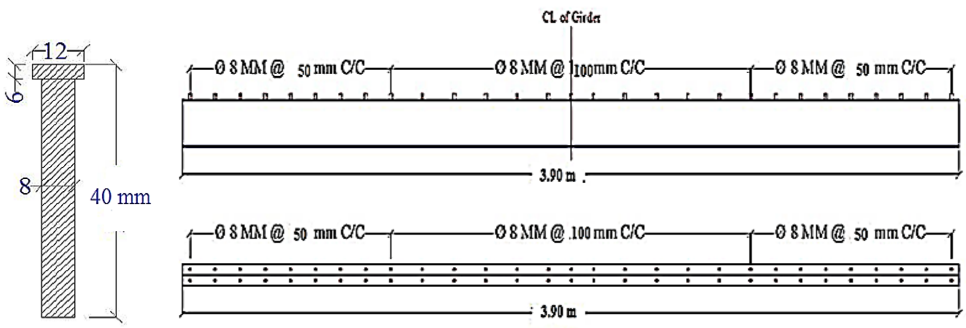
Total self-weight = 0.66 kN/m



(a) Longitudinal Section



(b) Cross Section



(c)Details and distribution of shear connector for Bridge Model

Fig. 1 Model Steel – Concrete Composite Girder Used in Experimental Tests

Each model composite test girder is a simply supported beam of span length 3.9m consisting of:

1- Steel beam HEA 160 composes of plate (160mm x9mm) for top and bottom flange and plate (134mm x6mm) for web.

2- Concrete deck slab of 300mm x 55mm cross section reinforced with welded wire fabric WWF (gauge 150mm x 150mm of 6mm diameter), with additional bars position transversely divided the distance to 75 mm and longitudinal direction and add of bars at the distance 60mm from the end bar as shown in **Fig. 1.**

3- Two rows of ø 8mm diameter studs (height 40mm) spaced at 80mm in transverse direction and

Longitudinally at direction a 50mm for1200 mm and 100mm for the next 750 mm c/c. The studs acting as shear connectors to connect the steel girder with the reinforced concrete slab. The studs are welded on the top steel beam flange such that one stud is on each side of the flange as shown **Fig. 2**.

4-Bracket (Transfer System) consist of plate with dimension of 250mm by 160mm with a thickness of 20mm and welded to perpendicular plate with dimension (140mm*160mm*25mm) with one hole. Four stiffeners were welded at each end of plates to resist deformation that may generate during strands stressing. Four brackets was designed for

levels 160mm and eccentricity of 80mm below the bottom face of flange other four bracket was at level 240mm and eccentricity 120mm as in shown **Fig. 3**.

5- Deviator was used at mid span of all strengthened beams. The deviator was fabricated from a horizontal plate with dimension of 160 mm by 160mm and thickness of 20mm, welded to perpendicular plate with dimension (280mm*160mm*20mm) with six holes. Four stiffeners were welded at ends of plates to resist deformation that may generate during strands stressing, one hole for each strengthening case. **Fig. 4** shows all the deviator details were symmetrical as much as possible.

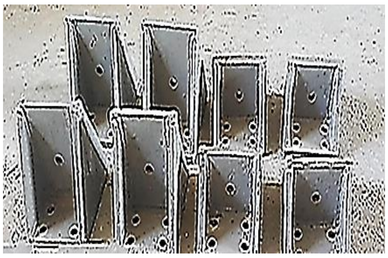


Fig. 2 Shear Connectors **Fig. 3 Type of Bracket** **Fig. 4 Deviator**

6-Grips are an important device to transfer load from the member to the strand. For each size of the strand there is special size of the grips as shown in **Fig. 5**.

7-Ply-wood sheets and bolts were used to manufacturing the mould for the reinforced concrete deck slab of the composite model beams. The mould consisted of two symmetric parts which were tied up together with the steel beam, wires and bolts to insure right position and horizontal level with the steel beam top flange. The inside of the mould was lubricated before placement of the reinforcement mesh. The mould is shown in **Fig. 6**.

Fig. 5 Anchorage device (grip) **Fig. 6 Photo of the Slab Mould**

2.2 Material Properties (Reactive Powder Concrete)

2.2.1 Cement

Ordinary Portland cement (type 1) produced in Iraq from Mass Bazian Company was used in casting of the concrete deck slab of the model composite girders.

2.2.2 Fine Aggregates

Fine aggregate is of (0.6 mm) maximum size brought from Al-Ukhaidher natural source is used for the ultra-high performance concrete

2.2.3 Admixtures

A- Silica Fume

Silica fume has become one of the necessary ingredients for making ultra-high performance concrete. Micro silica a produced by Sika Company is much more reactive than any other natural pozzolanic.

B- High Range Water Reducing Admixture

It is a new generation high performance superplasticizing concrete admixture (HRWRA sika viscocrete). Its basis is aqueous solution of modified polycarboxylate. It is primarily developed for applications in the precast concrete industries, self-compacting concrete, hot weather concrete as well as with highest water reduction up to 30 %.

C-Steel fiber

The hooked high tensile steel fiber that used in this work had a 1.05 mm in diameter and length of 50 mm, i.e. the aspect ratio of 47.6. It brought from China by Yutian Zhitai Steel Fiber Manufacturing Co., Ltd. The tensile strength is 1100 MPa.

2.3 Equivalent Loads

Before the application of the external presterssing on the girders, an important criterion was satisfied to get an exact simulation of the model composite girders with the prototype composite girder. This criterion is the consideration of the same maximum tensile stress in the prototype girder resulting from self-weight plus the superimposed dead load. Concrete block 450mm×300mm×100 will used as an additional mass to satisfy the simulation requirement of specific gravity loads (self-weight and superimposed). The girder carries a uniformly distributed dead load which is the sum of the self-weight and the additional superimposed dead load = 3.2 kN/m.

2.4 Estimation of Maximum Prestress Force

Fig. 7 shows a composite beam model subjected to externally horizontal prestressing force applied at a level of 80 and 120 mm below its bottom flange face. The location of each these two sections is at 0.45 m distance away from the nearer support. The maximum value of the applied prestress force (P_r) in this case are found from the allowable stress in concrete at top fiber at mid span of composite beam which is calculated as follows;

$$n \times f_t = -\frac{P_r}{A} + \frac{P_r \times e \times C}{I} - \frac{M \times C}{I}$$

Tensile stress f_t in the concrete is equal to $0.4\sqrt{fc'}$ according to ACI code [3] was allowed to induce in the concrete. Eccentricity (e = 80mm + Y_b') which lead that the prestress force P_r equal to (9 Ton) and for (e = 120 mm+ Y_b') the prestress force is (7 Ton).

2.5 Instrumentation

After the superimposed dead load was applied (through the use of concrete blocks uniformly distributed on the top of the model along its full length) the two point loads were then applied on the model girder in successive increments, up to failure.

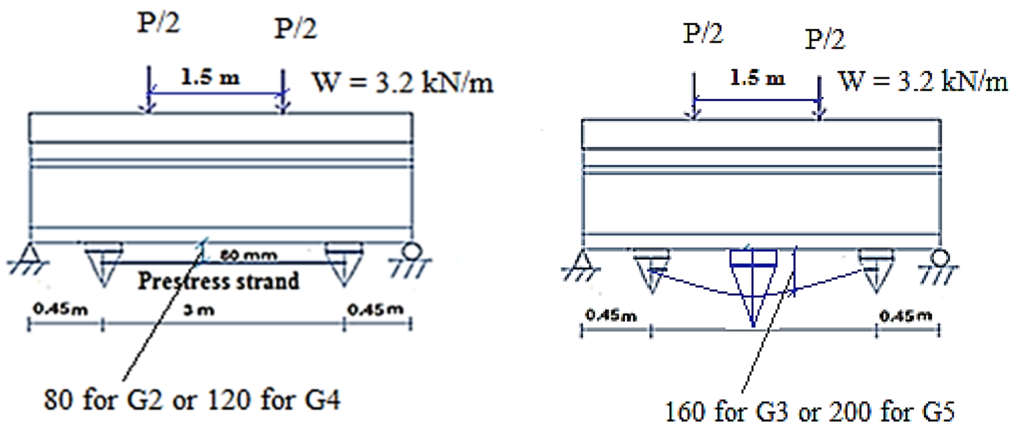


Fig. 7 Location of the External Prestress Tendon

At each loading increment, the measurements were documented, as follows:

1- Load was applied by a hydraulic jack of 500 kN capacity, with an increments of 5 kN.

2- The deflection was measured using a dial gauge as shown in **Fig. 8** at mid–span as well as under point load. The deflection readings of these two dial gauges were taken each 5 kN loading increment.

2.6 Testing Procedure

Before testing, the model girder specimen was placed on the supports of the testing machine, and then the superimposed dead load (represented by the added concrete blocks) was applied as uniform load on the girder and then the prestress tendon (in all model girders except the reference girder) were post–tensioned to the specified force required for the test as shown **Fig. 9**. Thereafter two point loads were applied and the results of dial gauges were obtained. After that the model girder specimen was loaded with a constant rate of loading of approximately (5 kN/min.). Each load level, measurements were taken for the deflections at mid – span and under point load. The loading was continued up to the stage of failure (ultimate stage).

Fig. 8 Dial Gauge

Fig. 9 Prestress Post-Tension

3. Results and Discussion

Table. 2 gives a full detailed description of the experimental results for all the tested composite girders models of the present investigation.

Table. 2 Experimental Test Results of the Tested Models

Group	Symbol	Designation	e @ distance 45cm from end span	e @mid span by deviator	P (kN)	Δu mid span (mm)	Δu Under point load (mm)
Reference 1	G1	GR1	-	-	220	55.68	45.78
1	G2	GP9S-e80	80	80	300	43.04	37.42
	G3	GP9D-e160	80	160	320	48.31	40.03
2	G4	GP7S-e120	120	120	305	43.97	35.98
	G5	GP7D-e200	120	200	325	49.55	42.69

* P = Total applied on two point load (P = 2V)

e80 = the distance below bottom face of flange

Where:
G = Girder, **P9** = the value of the external prestressing force was 9 Tons, **P7** = the value of the external prestressing force was 7 Tons, **S =** Strand of prestress represents as straight line,**D =** Strand of prestress represents by deviator

3.1 Reference Girder (G1)

This girder carried the symbol G1 and designated as GR1. It was a composite steel–concrete girder with compressive strength of the deck slab concrete fc'=90 MPa and no prestressing force was applied to it. The ultimate two point loads carrying capacity of the beam was P = 220 kN and this maximum value of load gave a vertical deflection of 55.68and 45.78mm at mid–span and under point load respectively. The load–deflection behavior of this reference girder is shown in **Fig. 10**. As the applied load was increased mor than (170 kN), the state of stress in the steel beam changed from linear to nonlinear and the compressive stress block in the concrete deck slab became inelastic; this can be concluded from the shape of the P-Δ curve which is showing a noticeable change in shape . A sign of flexural failure was shown when the ultimate load was reached when the concrete of the deck slab crushed at mid span due to excessive compressive stress on it resulting from the development of high bending moment. **Fig. 11** shows the mode of failure of the reference beam GR1.

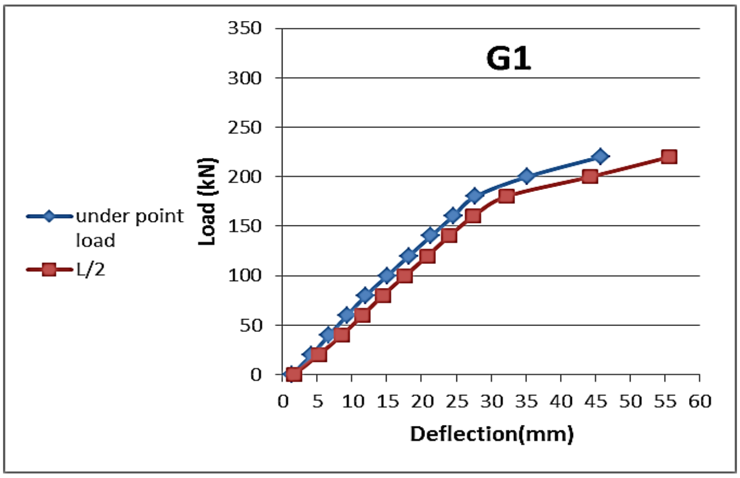


Fig. 10 Load–Deflection Behavior of Girder G1 (designated as GR1)

Fig. 11 The Mode of Failure of the Steel Girder (GR1) G1

3.2 Model Girders of Group (1)

This group consists of two steel – concrete composite model girders G2, G3 which were designated as GP9S-e80 and GP9D-e160. Each girder in this group consisted of a steel beam subjected to an external prestressed force of value 9 Ton. The prestressing force was applied after the reinforced concrete deck slab was cast on top and the superimposed dead load is applied.

The values of the ultimate two loading points for girder GP9S-e80 and GP9D-e160 were respectively 300kN and 320kN corresponding to deflections at mid–span of 43.04mm, and 48.31mm and deflections under point load was 37.42mm and 40.03mm respectively. The load – deflection cures for these girders are shown in **Fig. 12**. The two girders of group 1 failed by flexural mode of failure characterized by full yielding of the steel girder and crushing of the deck slab concrete at mid–span for (G12) and under point load for (G13)as shown in **Figs. 13 and 14**, respectively.

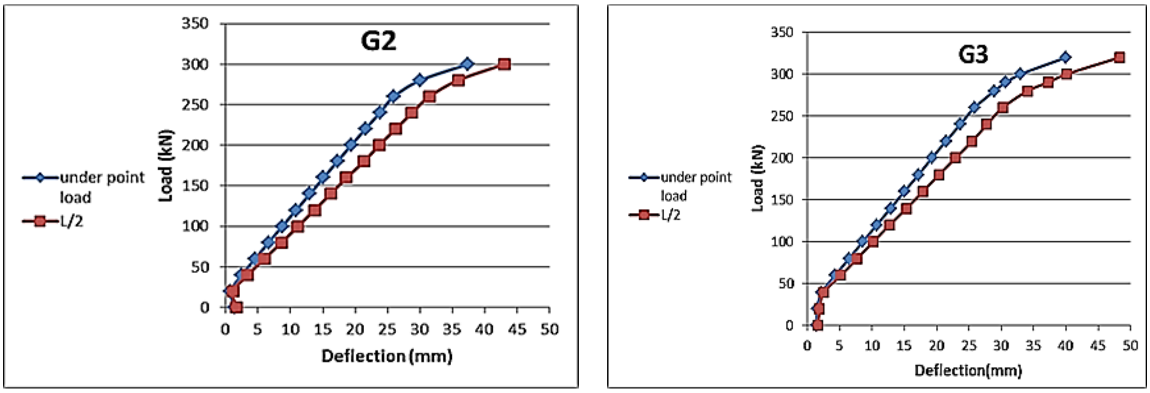
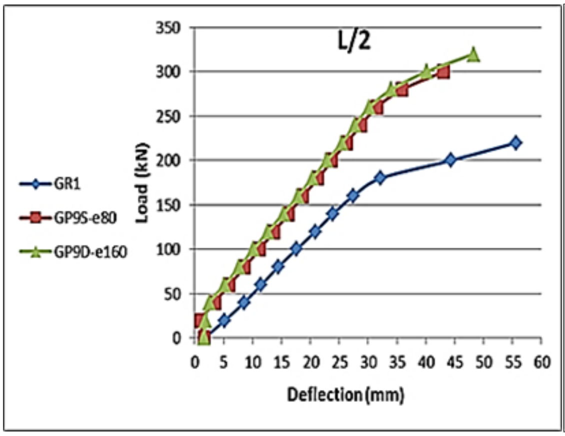


Fig. 12 Load–Deflection Behavior of Girder G2 (designated as GP9S-e80) and G3 (designated as GP9D-e160)

Fig. 13 The Mode of Failure of the Steel Girder (GP9S-e80) G2

Fig. 14 The Mode of Failure of the Steel Girder (GP9D-e160) G3

A comparison of these two load–deflection curves of group 1 with the reference girder GR1 are shown in **Fig. 15**. **Table. 3** shows the percentage increase in the value of the ultimate load of girders GP9S-e80 and GP9D-e160 compared to that of the reference girder GR1. This enhancement in the load carrying capacity is caused by applying external post– tensioned pre-stressing force on the composite steel–concrete girder. The enhancement in ultimate load ranged between 36.36% for G2 to 45.45% for G3



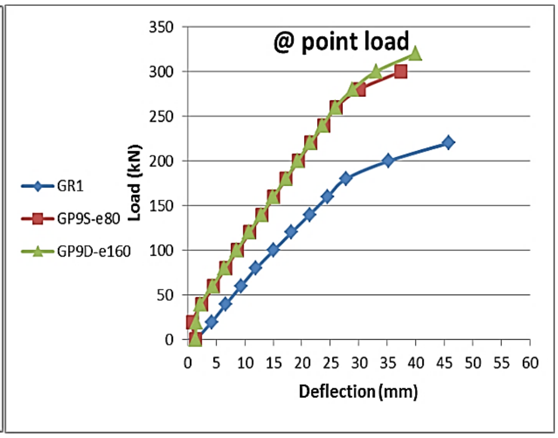


Fig. 15 Behavior of Reference Girder GR1 with the Girders of Group 1

Table. 3 Percentage Increase in Ultimate Load of the Girders of Group (1) Compared to Unprestressed Composite

Group	Designation	Prestress force(Ton)	Pre-stress-moment kN.m	fc' (MPa)	P kN	% increase In (P)
Reference	GR1	–	–	90	220	-
1	GP9S-e80	9	18.69	90	300	36.36%
	GP9D-e160	9	25.89		320	45.45%

3.3 Model Girders of Group (2)

This group consists of the two steel – concrete composite model girders G4 and G5 which were designated as GP7S-e120 and GP7D-e200 respectively. Each girder in this group consisted of a steel beam subjected to an external prestressed force of (7 Ton) for the girders. The values of the ultimate two–point loading for girder GP7S-e120 and GP7D-e200 were 305kN and 325kN respectively. The ultimate deflections at mid–span were 43.97mm and 49.55mm and deflections at under point load of 35.98mm and 42.69mm respectively. The load – deflection curves for these girders are shown in **Fig. 16**. All girders of group 2 failed by flexural mode characterized by full yielding of the steel beam and crushing of the deck slab concrete at mid–span for G4 and under point load for G5 as shown in **Fig. 17 and 18**, respectively.

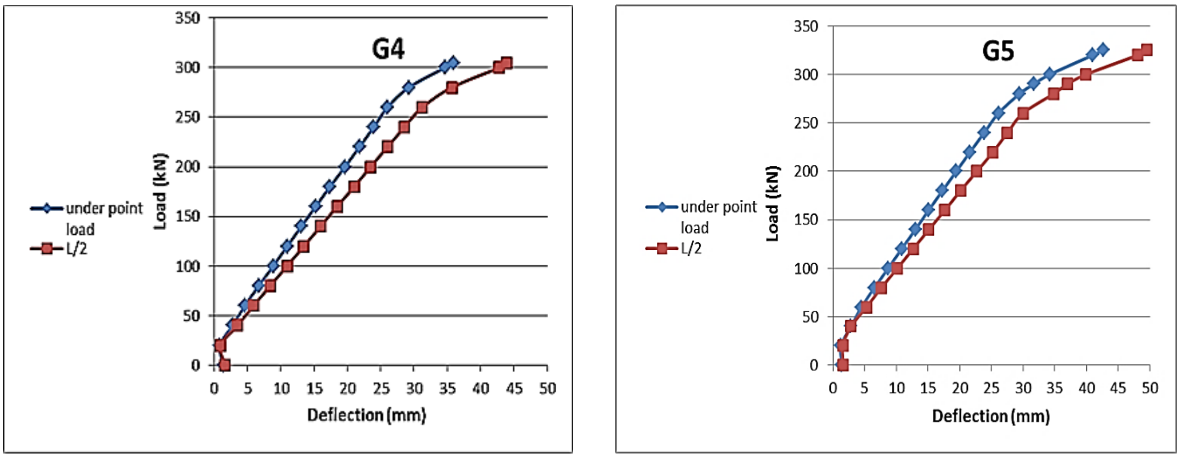


Fig. 16 Load–Deflection Behavior of Girder G4 and girder G5

Fig. 17 The Mode of Failure of the Steel Girder (GP7S-e120) G4

Fig. 18 The Mode of Failure of the Steel Girder (GP7D-e200) G5

A comparison of these two load–deflection curves of group 2 with that of the reference girder GR1are shown in **Fig. 19**.

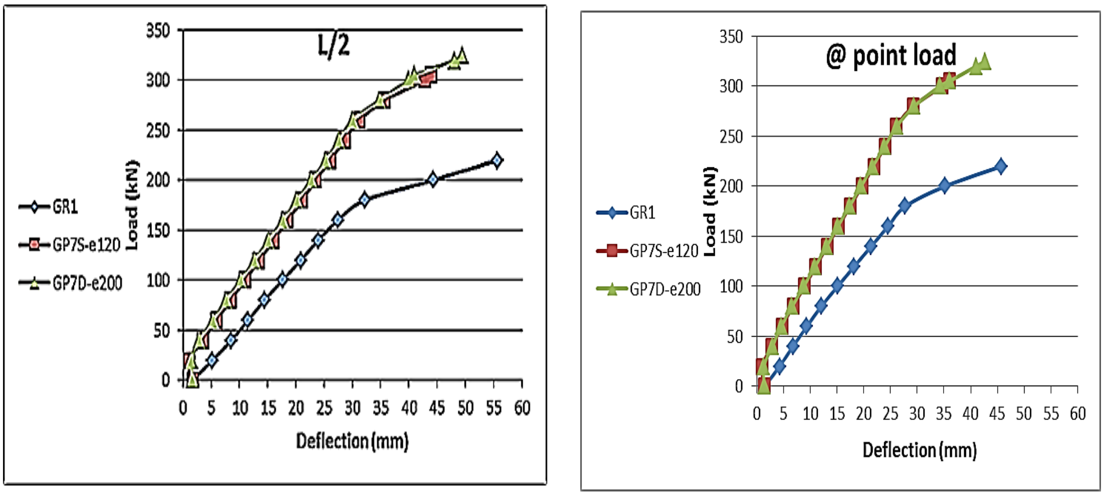


Fig. 19 Behavior of Reference Girder GR1 with the Girders of Group 2

Table. 4 shows the percentage increase in the value of the ultimate load of girders GP7S-e120 and GP7D-e200 compared to that of the reference girder GR1. The enhancement in ultimate load ranged between 38.63% for G4 to 47.72% for G5.

Table. 4 Percentage Increase in Ultimate Load of the Girders of Group (2) Compared to Unprestressed Steel Girder

Group	Designation	Prestress force(Ton)	Pre-stress-moment kN.m	fc' (MPa)	P kN	% increase In (P)
Reference	GR1	–	–	90	220	-
2	GP7S-e120	7	17.33	90	305	38.63%
	GP7D-e200	7	22.93		325	47.72%

3.4 Compressions between Groups

A compression of load–deflection curves for the girders (G2 with G4, G3 with G5) are shown in **Fig. 20**.The two system have different value of prestress force but have the approximately the same prestress moment due to different value of eccentricity. It can be seen that the flexural stiffness's of the girders with external prestress 7 Ton are slight higher than the flexural stiffness's of the girders with external prestress 9 Ton because the value of e @ end is 120mm in group 2 which more than value of e at end is 80mm in group 1 as shown in **Fig. 20**.

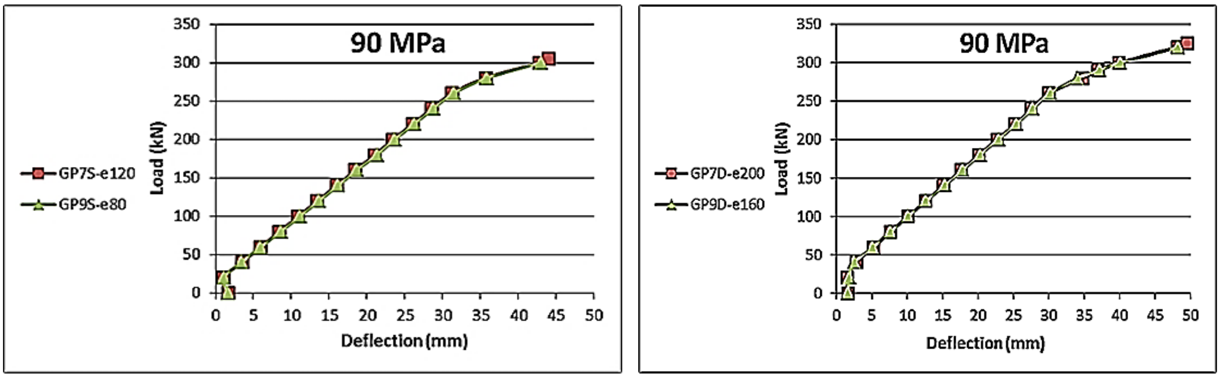


Fig. 20 Comparison Load–Deflection Curves with Different External Perstress (7 Ton and 9Ton)

4-Conclusion

1- The technique of applying external pre-stress to strengthen composite girders, which are usually used as main structural elements in bridges, was found in this research very beneficial and practical.

2-The ultimate loads of such girders were shown experimentally to increase remarkably with the increase in the amount of the applied external prestressing force and the eccentricity. Practically, the application of the prestressing force, being at the underside (bottom flange) of the steel, allows freedom of strengthening work to be performed without interference with the flow traffic on the bridge.

3- The four composite girders of group 1 and 2(GP9S-e80, GP9D-e160,GP7S-e120 and GP7D-e200), which had RC deck slab of cylinder compressive strength f'c =90 MPa and subjected to external prestressing force of 9 Ton and 7 Ton , carried ultimate loads which were greater than the ultimate load of their corresponding Without deviator represent 36.36% and 38.63% respectively. unprestressed reference girder GR1 by 36.36 % , 45.45 % , 38.63 %and 47.72% respectively, these percentages of load enhancement were caused by the combined effect of value of external pre-stressing and the value of eccentricity.

4- To stand on the actual enhancement in ultimate loads of composite girders caused by changing only the sequence of applying the external prestress force (whether with or without the deviator), while keeping the other parameters constant, the girders (GP9S-e80MPa and GP7S-e120) were compared with their corresponding girders (GP9D-e160MPa, and GP7D-e200). Results of such comparison showed that the percentage enhancements in the ultimate load due to applying the external prestress force with deviator represent (45.45% and 47.72%) respectively higher than those whose external prestress force

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التحري العملي للمقاطع المركبة من الخرسانة عالية الأداء والروافد الحديدية المسبقة الجهد خارجيا

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قسم الهندسه المدنية
جامعه بغداد/ العراق

الخلاصة :- في هذا البحث تم أجراء التحري التجريبي لتأثير أستخدام تقنية الأجهاد المسبق الخارجي في تقوية روافد الجسورالمركبة من الروافد الحديدية والخرسانة فائقة الأداء. شمل الجانب العملي فحص خمسة نماذج مركبة من رافدة فولاذية – وسطحة خرسانيه مسلحة , واحدة غير مسبقه الأجهاد وأربعة مسبقة الجهد خارجيا . تم تقسيم هذه الروافد الى مجموعتين . تم انشاء جميع نماذج الروافد أعتمادا على تصغير نموذج جسر اصلي لمعامل 1/4 تم تصميمها وفقا لمواصفات القياسية AASHTO LRFD 2012. كل نموذج تم فحصة على اساس أسناد بسيط وبفضاء 3.90m تم التحميل بصورة تدريجية حتى الوصول الى الفشل تحت تأثير نقطتي تحميل. قوة الأجهاد الخارجي المسبقة الشد لروافد المجموعة الأولى هي 9Ton تم تسليطها بعد صب البلاطة الخرسانية المسلحة للروافد المركبة وتسليط الأحمال الأضافية المكافئه للأحمال الميتة بينما المجموعة الثانية هي 7Tan. المتغيرات المتضمنه هي شكل وموقع تسليط الاجهاد المسبق الخارجي والذي يكون مرة على شكل خط مستقيم يبعد مسافة محددة لامركزية عن الوجة السفلي الخارجي والثاني على شكل خط مائل وبقيمة مركزية في الوسط أكثر من النهايات والمتغير الثاني هو قيمة قوة الأجهاد المسبق الخارجي المسلط. لنتائج التجريبية أظهرت بشكل ملموس التحسين الحاصل في سعة تحميل نماذج الروافد التي فحصت تحت تأثير الأجهاد المسبق الخارجي مقارنة مع رافد المرجع وغير المسلط عليها اجهاد مسبق خارجي. المجموعة الأولى أظهرت زيادة في الحمل الأقصى بنسب 36.36% و 45.45% بتسليط قوة أجهاد مسبق خارجي 9Ton وبأنحراف (80mm و 160mm على التوالي) بينما المجموعة الثانية أظهرت 38.63% و 47.72% بتسليط قوة أجهاد خارجي 7Ton وبأنحراف (120mm و 200mm على التوالي)