Influence of Compressive Strength on Ultimate Strength of Composite Concrete-Steel Girder Subjected to External Prestress

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Abstract: ***Experimental programmed was carried out to investigate the influence of compressive strength on composite steel I-girder decks subjected to external prestress.This program included fabricating and testing six scale down 1/4, were designed according to AASHTO LRFD 2012 standard specification. Each girder was test as simply supported with span of 3.90m and classified in three groups. The first group consists of two girders for which the concrete strength is 50 MPa. The second group consists of two girders for which the concrete strength is 70 MPa. The third group consists of two girders for which the concrete strength is 90 MPa. In all groups, the first girder has straight eccentricity without deviator and the second girder have inclined variable eccentricity with deviator at the mid span. The applied loaded incrementally up to failure under the action of two point loads for each increment of load. Vertical deflections were measurement at mid and under point load. The prestressing force in the all girders was (9) Ton applied after the applied of superimposed dead load on RC deck slab. The variables included the compressive strength of concrete, the values of the eccentricity which mean that (location of prestress below bottom face) and with or without deviator .The girders of group (1) showed ultimate load of 210 and 230.The girders of group 2 showed ultimate load of 260 and 280. The girders of group 3 showed ultimate load of 300 and 320. When the compressive strength of concrete is increased from 50 MPa to 90 MPa it causes increased the ultimate load about 42.85%. In general the ultimate load increase by 10% when the compressive strength increases 10 MPa.***

Keywords: External Prestress, Composite section, Compressive Strength

1. Introduction

The research work carried out in strengthening or rehabilitation of existing structures is enormous and covers various types of elements that are commonly used in engineering construction [1] and [2]. External prestressing, initially developed for bridges, is now becoming popular and applicable for a variety of structural systems [3] and [4]. Comparing with an internal prestressing technique, an external prestressing system has the merits of being simpler to construct and easier to inspect and maintain [5]. **Choi et al.(2008)** [6] tested steel – concrete composite bridge specimens after strengthening them by external prestressing. It was concluded from the test specimens that the increment of a tendon force, which is due to the live load, is approximately 5% compared to the total amount of a tendon force. Since the total amount of strands can be reduced by considering this increment of a tendon force, a more economical design for strengthening of a bridge may be feasible. **Ahmed Q. W. (2011)**[7] analyzed four composite beams numerically and the results were compared with those obtained from existing experimental tests. Parametric study was presented to show the effects of the compressive strength of concrete, ratio of effective pre-stressing stress f_{pe} to ultimate stress f_{pu}, effective height to the center of prestressing cables and the type of external prestressing technique. It has been found that, as the compressive strength of concrete increases from 20 MPa to 60 MPa the ultimate load increases by about 18.9 % and for higher value of ratio fpe/fpu (effective prestress to ultimate stress) the ultimate load increases. Increased effective height of tendon has a significant effect on the ultimate strength capacity.

2. Experimental Program

2.1 Fabrication of the models

The scaled-down factor of (1/4) was adopted for scaling down the model dimensions from a full scale (Prototype).To meet the requirements needed in the experimental, the properties of the model girder section are given in Table 1 considering three different cases. Each of the first, second and third case represents the case of the composite beam action where the steel beam is interconnected to a top reinforced concrete deck slab of 28 days cylinder compressive strength fc' of either 50 MPa (Case1) ,70 MPa (case 2) and 90 MPa (Case 3). Each model composite test girder includedSteel beam HEA 160 consist of plate (160mm x9mm) for top and bottom flange and plate (134mm x6mm) for web and concrete deck slab of cross section 300mm x 55mm reinforced with welded wire fabric WWF (gauge 150mm x 150mm and ϕ 6mm) and two rows of ϕ 8mm diameter studs (height 40mm) spaced 80mm in transverse direction and the spaced in the longitudinal direction from the end of the girder are at 50mm at 1200mm, 100mm at 750mm c/c as shown Figure 1.

Case I - Composite with $f_c' = 50$, n=6, $E_C = 4700\sqrt{f_c'}$

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 HEA160							
Top flange	160 × 9	1440	147.5	212400	9720	27.26236	1070259.937
Web	6 × 134	804	76	61104	1203052	-44.2376	1573403.177
Bottom flange	160 × 9	1440	4.5	6480	9720	-115.738	19289091.15
Slab concrete equivalent to steel 300/n=50	50 × 55	2750	179.5	493625	693229.2	59.26236	9658073.882
Summation		6434		773609	1915721.2		31590828.15

$\bar{Y}_b$ = 120.2376 mm , I_{NA} = 33506549 mm^4 , slab width = 300 , $n = \frac{E_s}{E_c}$

Case II - Composite with $f_c' = 70$, n=5, $E_C = 4700\sqrt{f_c'}$

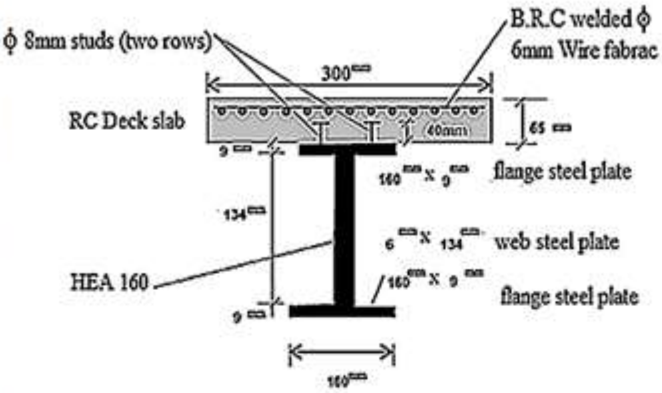
member	Size (mm)	A (mm^2)	Y_b(mm) centroid to bottom	$A \times Y_b$ (mm^3)	I_o (mm^4)	d (mm) C.G to NA	Ad^2 (mm^4)
section HEA160							
Top flange	160 × 9	1440	147.5	212400	9720	22.59536	735192.4764
Web	6 × 134	804	76	61104	1203052	-48.9046	1922897.641
Bottom flange	160 × 9	1440	4.5	6480	9720	-120.405	20876079.07
Slab concrete equivalent to steel 300/n=60	60 × 55	3300	179.5	592350	831875	54.59536	9836156.298
Summation		6984		872334	2054367		3337032549

$\bar{Y}_b$=124.9046 mm ,I_{NA} = 35424692 mm^4 , slab width = 300 , $n = \frac{E_s}{E_c}$

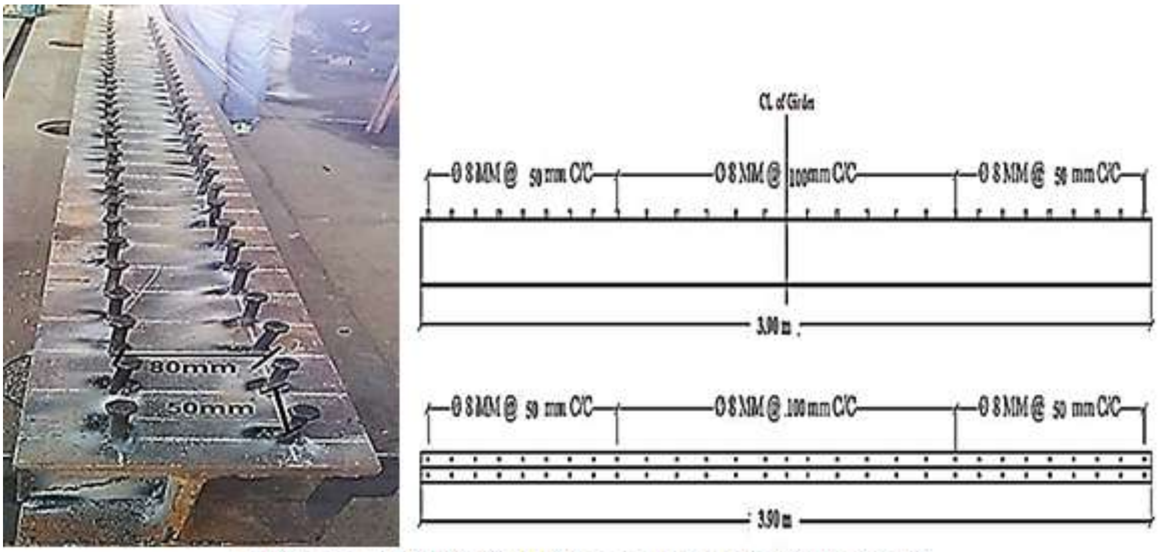
Case III - Composite with $f_c' = 90$, n=4.5, $E_C = 4700\sqrt{f_c'}$

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 HEA160							
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

$\bar{Y}_b$=127.6254mm I_{NA} = 36554434 mm^4 , slab width = 300 , $n = \frac{E_s}{E_c}$



(a)Cross section



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

Figure 1: Model Steel – Concrete Composite Girder Used in Experimental Tests

2.2 Concrete Mixes

Three concrete mixes were designed and prepared for constructing the reinforced concrete deck slab of the composite model beams. Mix (1) was normal strength concrete while mix (2) was a high strength concrete and mix (3) was ultra-high compressive strength concrete [8], [9], [10] and [11] . The mix properties of the constituents by weight for these three mixes are listed in Table 2. The concrete was mixed by using a rotary mixer of (0.10m3) capacity. The desired amount of silica was mixed with cement amount in dry for five minutes. Then, fine sand is additional to this mix and mixed ten minutes. Then, the required amount of water which is premixed with super-plasticizer and added slowly and mixed for 15 minutes. Fiber were distributed into the homogenous mix gradually in five minutes during mixing, and mixing process continued for additional five minutes. In total, the mixing of one batch needs approximately (40) minutes.

Table 2: Properties of the Three Types of Concrete Mixes used in Deck Slab

Mix	Type of concrete	Mix Properties (kg/m3)				W/C	Viscocrete 5930* %	Steel fiber** %	f'c (MPa) cylinder strength
		Cement	Sand	silica	Water	ratio			
1	Normal strength	470	835	23.5	164.5	0.35	1.5	0.5	50
2	High strength	770	1140	115.5	208	0.27	3	0.5	70
3	Ultra-high strength	925	1000	232	222	0.24	4.5	0.5	90

*percent of cement weight
**percent of mix volume

2.3 Load Simulation

Before the application of the external pre-sterssing on the girders, an important criterion was satisfied to get an exact simulation of the model composite girders with the prototype composite girder. The equivalent loads were applied on the model to induce the same longitudinal bottom steel flange stress as that of the full-scale bridge due to real self-weight and superimposed dead loads. Concrete block450mm×300mm×100 will use as an additional mass to satisfy the simulation requirement of specific gravity loads. The equivalents (self-weight and superimposed equal to3.2 kN/m) are used as addition superimposed dead load which gives the same stress with prototype.

2.4 Estimation Prestress Force

Figure 2shows a model of composite beam subjected to externally longitudinal prestressing force applied at a level 80 mm below its bottom flange face represent the case of straight tendon with constant eccentricity Figure-(2-a) represent the second case when use the deviator at mid span at a level 160mm below its bottom flange. Each of these two sections is of 0.45 m distance away from the nearer support. The maximum value of the applied prestress force (P_r) in this case from the allowable stress in concrete at top fiber at mid span of composite beam which is calculated as follows; $(\boldsymbol{n} \times \boldsymbol{f_t} = -\frac{P_r}{A} + \frac{P_r \times e \times C}{I} - \frac{M \times C}{I})$. Tensile stress in concrete $\boldsymbol{f_t}$which equal to $0.4\sqrt{fc'}$ was allowed to induce in the concrete and eccentricity (e = 80mm + Y_b') which lead that the prestress force $\boldsymbol{P_r}$ equal to (9 Ton). All the girder contains two brackets for strand anchorage at distance 0.45 m from supported. Girder (G2, G4 and G6) contains deviator was used at mid span of all strengthened beams.

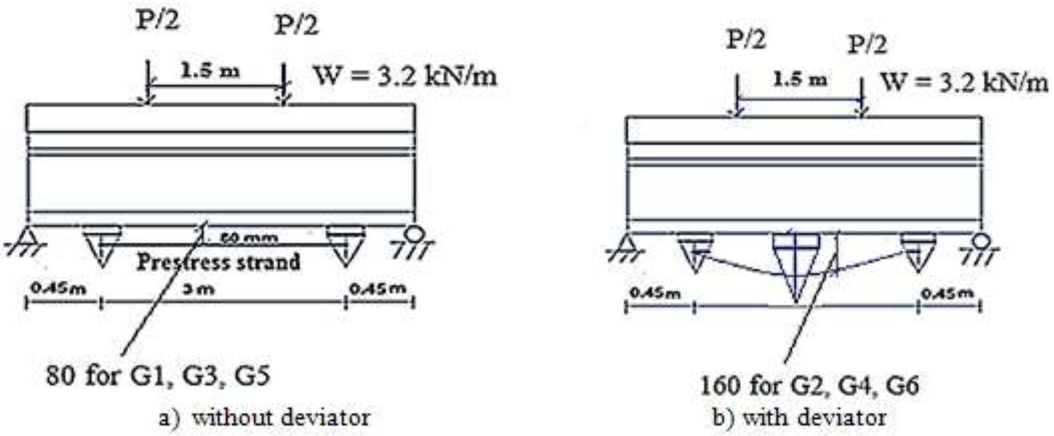


Figure 2: Location of the External Prestress Tendon

2.5 Instrumentation and Testing Procedure

Before testing, the model girder specimen was placed on the supports of the testing machine. Two 30 mm heavy duty steel rollers were used to support the girders over the clear span and then concrete blocks with dimensions (450 x 300 x100 mm) were used to idealize the superimposed dead load of (3.2kN/m) with total number of 38 blocks was applied as uniform load on the girder . Thereafter the prestress tendon was post–tensioned to the specified force required for the test as shown in Figure 3.All model girders were tested under static two concentrated point loads. The (IPN- 400) steel beam used as a rigid be to idealize the two point load case. The jack load was applied at mid-span of rigid beam and then transfer to the two point load. Steel plate 10 mm thick was used under roller supporting of the rigid beam to avoid the local concrete block failure. The concentrated load was subjected on the test model girder specimen through a jack of the testing machine as shown in Figure 4. After the preparations were finished and the initial readings of the dial gauges were taken, the load was applied with a loading increment rate of about 5 kN.

Figure 3: Prestress Post-Tension

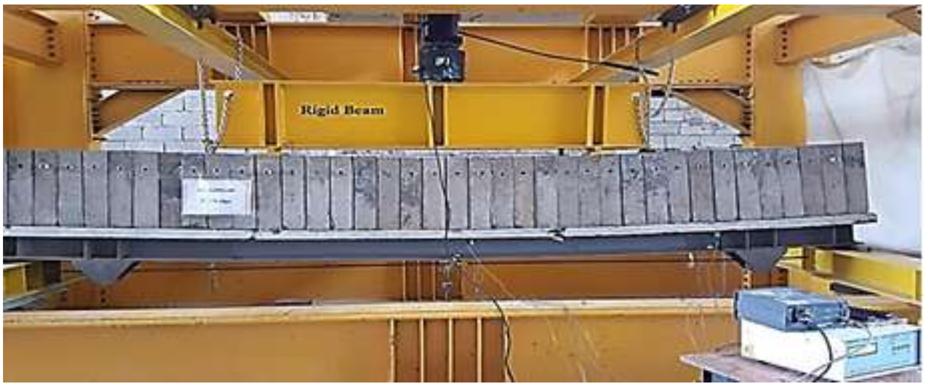


Figure 4: Loading System in the Testing Machine

3. Results and Discussion

Table 3 gives a full detailed description of the experimental results for all the tested composite girders models of the present investigation.

Table 3: Experimental Test Results of the Tested Models

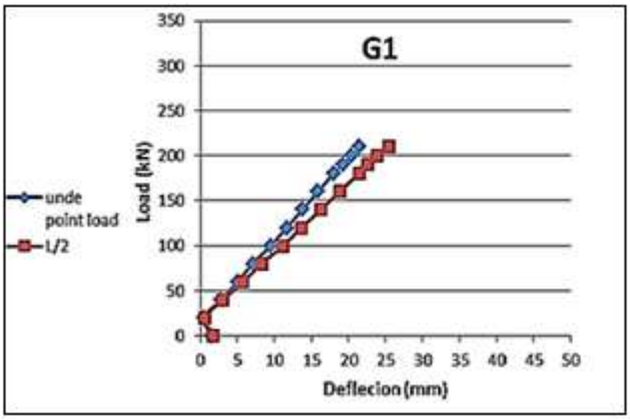
Group	Symbol	Designation	e @ distance 45cm from end span	e @mid span by deviator	fc' (MPa)	P(kN)	Δu mid span (mm)	Δu Under point load (mm)
1	G1	GP9S-e80	80	80	50	210	25.43	21.44
	G2	GP9D-e160	80	160		230	26.53	22.68
2	G3	GP9S-e80	80	80	70	260	29.87	25.81
	G4	GP9D-e160	80	160		280	33.54	29.74
3	G5	GP9S-e80	80	80	90	300	43.04	37.42
	G6	GP9D-e160	80	160		320	48.31	40.03

* P = Total applied on two point load (P = 2V)
e80 = the distance below bottom face of flange

G = Girder, **P9** = the value of the external prestressing force was 9 Tons, **S =** Strand of prestressrepresents as straight line, **D =**Strand of prestressrepresents by deviator

3.1 Model Girders of Group (1) fc' =50 MPa

This group consists of the two steel–concrete composite model girders G1and G2 which were designated as GP9S-e80 and GP9D-e160, respectively. Each of which was subjected to an external prestressing force prior to the gradual application of two–point loading up to failure. The external prestressing force was (9 Ton) for these two composite girders; the cylinder compressive strength of the deck slab concrete in all these girders was fc'= 50 MPa. Due to the presence of the external post–tensioned prestressing force, these two composite girders of group 1 showed an ability to carry higher ultimate loads with only little increase in vertical deflection. The values of the ultimate load for girders GP9S-e80 and GP9D-e160 were 210kN and 230kN, respectively corresponding to deflection at mid–span of 25.43mm and 26.53 mm and deflection at under point load of 21.44mm and 22.68mm. The load–deflection curves for these girders are show in Figure 5 respectively. The two girders of group 1 failed by flexural mode of failure as shown in Figure 6 and 7 respectively.



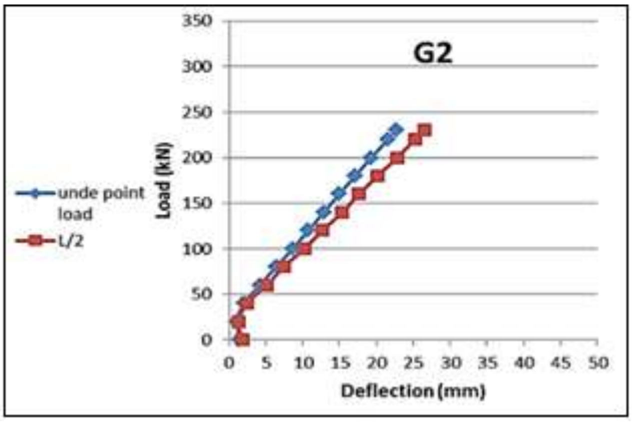


Figure 5: Load–Deflection Behavior of Girder G1 (designated as GP9S-e80) and G2 (designated as GP9D-e160)

Figure 6: The Mode of Failure of the Steel Girder (GP9S-e80) G1

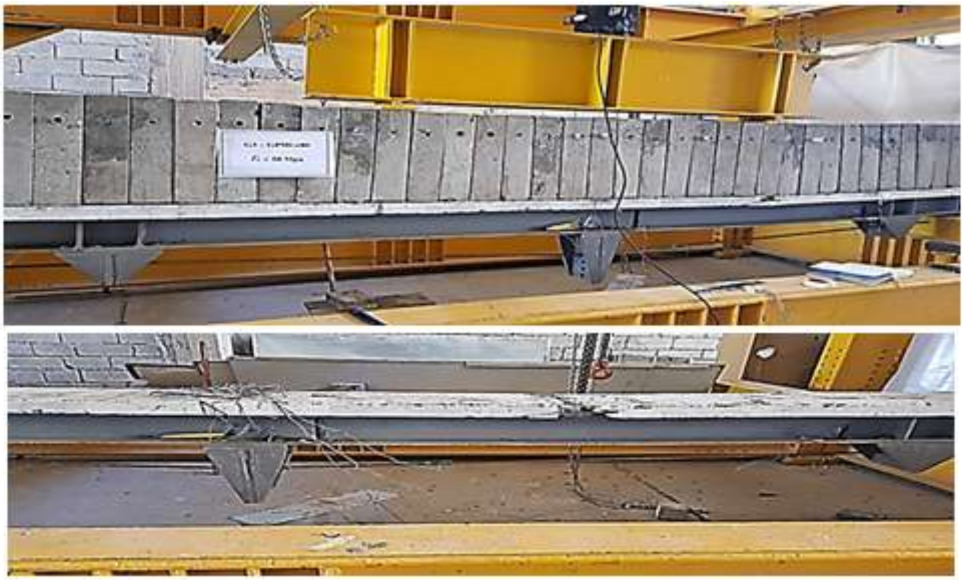


Figure 7: The Mode of Failure of the Steel Girder (GP9D-e160) G2

3.2 Model Girders of Group (2) fc' =70 MPa

This group consists of the two steel – concrete composite model girders G3and G4 which were designated as GP9S-e80and GP9D-e160 respectively. A high strength concrete was cast to represent the deck slab of these two girders which had a 28 days cylinder compressive strength of value fc'= 70 MPa. Prior to the application of two point loading up to failure, each girder was subjected to an external prestressing force. The value of this prestressing force was (9 Ton) for the two girders of this group in their respective order. The presence of such external post - tensioned prestressing force and the high strength concrete used for the deck slab increased the flexural strength of the composite model beams of this group remarkably. The values of the ultimate load for girders GP9S-e80and GP9D-e160 were respectively 260 kN and 280 kN corresponding to deflections at mid–span of 29.87mmand 33.54 mm and deflections at under point load of 25.81mm and 29.74mm respectively. The load–deflection curves for these girders are shown in Figure 8 respectively. As the applied load was increased (2P > 260 kN) in the other specimens for external pre-stress , 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. When the load approached the peak value, the girder showed rapid increases in the vertical deflection for only small increments in load. The two girders of group 2 failed by flexural mode of failure as shown in Figure 9 and 10 respectively.

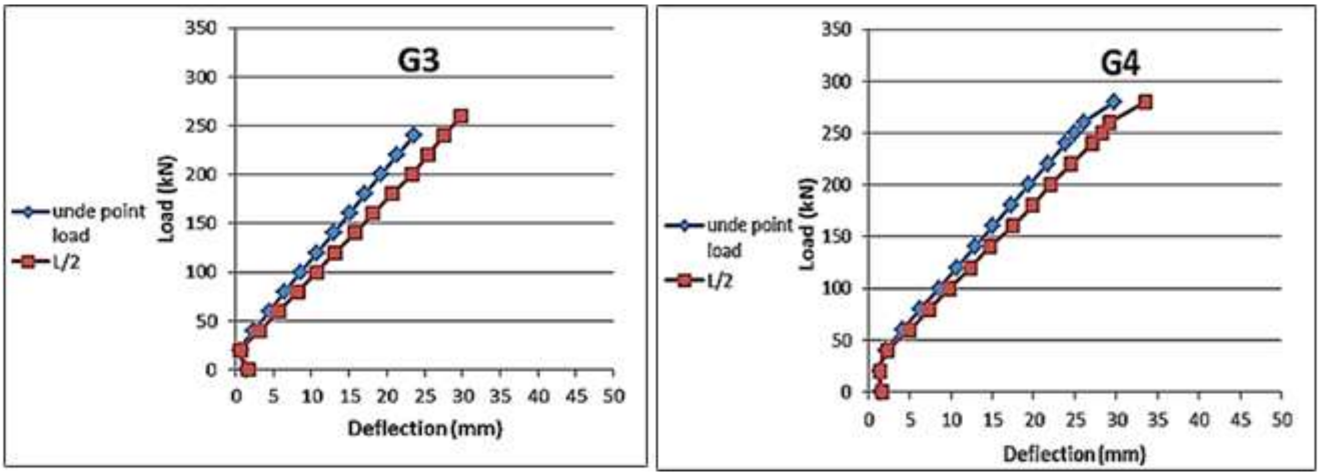


Figure 8: Load–Deflection Behavior of Girder G3 (designated as GP9S-e80) and G4 (designated as GP9D-e160)

Figure 9: The Mode of Failure of the Steel Girder (GP9S-e80) G3

Figure 10: The Mode of Failure of the Steel Girder (GP9D-e160) G4

3.3 Model Girders of Group (3) fc'=90 MPa

This group consists of the two steel – concrete composite model girders G5, G6 and which were designated as GP9S-e80and GP9D-e160 respectively. 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 a reinforced concrete deck slab was cast on top and the superimposed dead load is applied. The values of the ultimate two–point loading 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 at under point load of 37.42mm and 40.03mm respectively. The load – deflection cures for these girders are shown in Figure 11. 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 Figure 12 and 13 respectively.

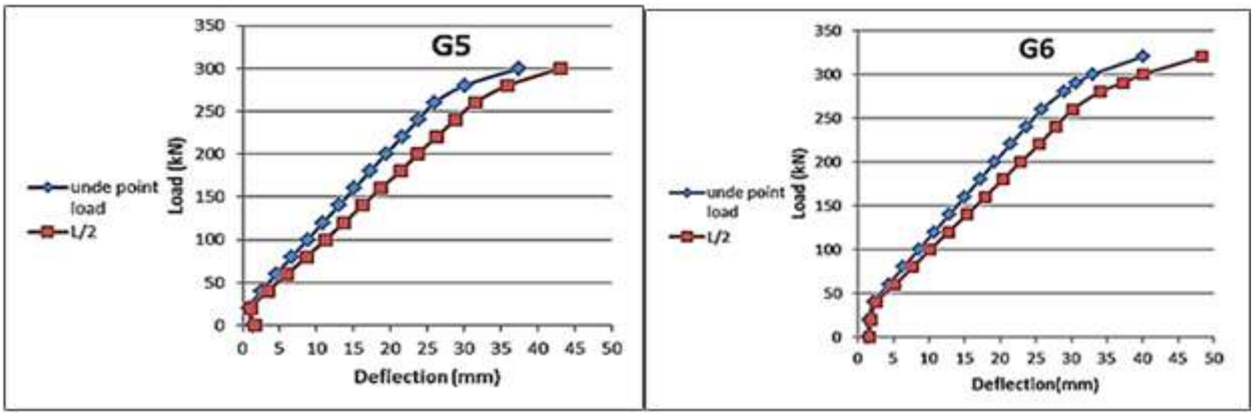


Figure 11: Load–Deflection Behavior of Girder G5 (designated as GP9S-e80) and G6 (designated as GP9D-e160)

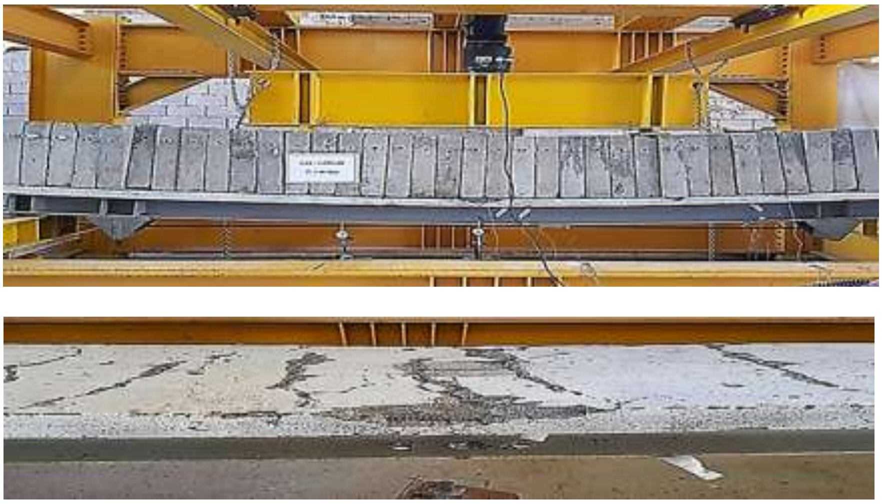


Figure 12: The Mode of Failure of the Steel Girder (GP9S-e80) G5

Figure 13: The Mode of Failure of the Steel Girder (GP9D-e160) G6

3.4 Compressions between Groups

To stand on the actual enhancement in the load carrying capacity of composite steel-concrete girders caused by the increase in the compressive strength of the deck slab concrete alone, the ultimate loads of the girder G1 are to be compared with their corresponding values of the girder G3 and G5 straight shape of strand and G2 are compared with G4 and G6 strand with deviator at mid span as shown Figure 14.

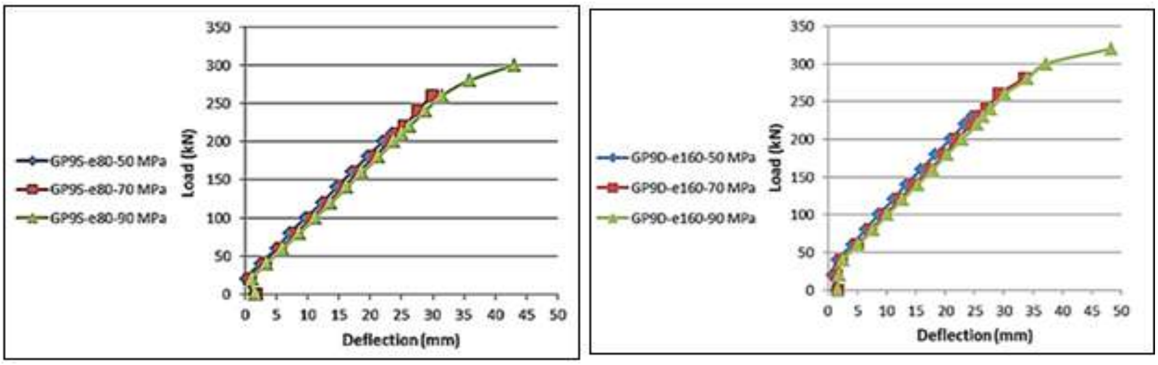


Figure 14: Comparison of these 6 Load–Deflection Curves with Different Compres-sive Strength (50, 70, and 90) MPa

It can be seen from Table 4 that the enhancement in ultimate load caused by the increase in the value of fc' from 50 MPa to 90 MPa. From the value of percentage increase of ultimate load, one can conclude that the percentage increases are equal to different between the values of reference compressive strength and other compressive strength i.e. when the compressive strength increase by 10 MPa the percentage increase in percentage of ultimate load equal to 10%.Figure 15 is plotted to show clearly that the effect of fc' on ultimate load.

Table 4: Comparison of these Percentage Increase in Ultimate Load

Symbol	Designation	fc' (MPa)	P kN	% increase In (P)	$\frac{Pu\ 70\ or\ 90}{Pu\ 50}$	$\frac{P\ with\ deviator}{P\ without\ deviator}$
G1	GP9S-e80	50	210	Reference	-	Reference
G3	GP9S-e80	70	260	23.80%	1.238	Reference
G5	GP9S-e80	90	300	42.85%	1.428	Reference
G2	GPD-e160	50	230	Reference	-	1.095
G4	GP9D-e160	70	280	21.73%	1.217	1.077
G6	GP9D-e160	90	320	39.13%	1.391	1.067

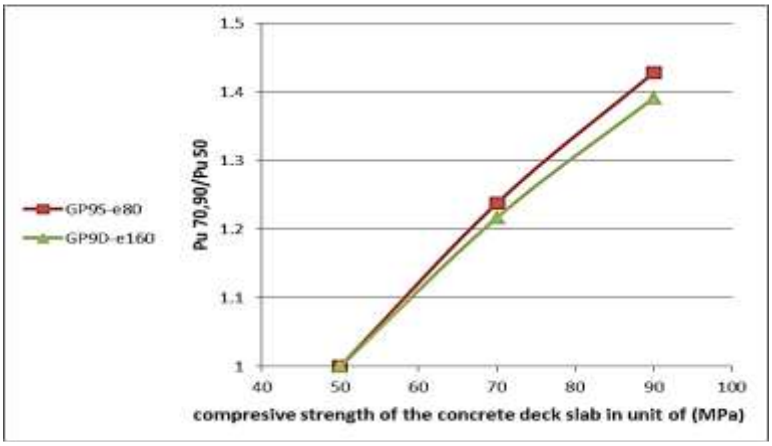


Figure 15: Effect of the Compressive Strength fc' of the Concrete deck Slab in Composite Steel- Concrete girders

4. Conclusion

1-From the experimental carried out to study the effect of compressive strength of concrete on the strength behavior, it was found that as the compressive strength of concrete is increased from 50 MPa to 90 MPa the ultimate load capacity is increased by about 42.85 % from 50MPa to 70MPa the ultimate load increase by 23.80%.

2-To stand on the actual enhancement in ultimate loads of composite girders due to using higher strength concrete for the deck slab only (i.e by keeping the other parameters including the amount of external prestressing force constant and eccentricity), the girders of group 1 were compared with their corresponding girders of group 2 and group 3 that had the ultimate load increase by 10% when the compressive strength increases 10MPa

3-Strengthening only part of the composite girder span was proved to enhance the strength of prestressed. The performance and procedure of strengthening and prestressing of girder was found to be effective and of remarkable effects since partial span prestressing technique in strengthening can provide a space between the supports and anchors system as a free area for fixing and jacking.

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 including the amount of the external prestress force and the compressive strength of the RC deck slab, the girders (GP9S-e80-50MPa, GP9S-e80-70MPa, ,GP9S-e80-90MPa) were compared with their corresponding girders (GP9D-e160-50MPa, GP9D-e160-70MPa, GP9D-e160-90MPa). Results of such comparison showed that the ultimate load due to applying the external prestress force with deviator represent (230, 280 and 320) respectively higher than those whose have strength external prestress force without deviator represent (210, 260 and 300) respectively.

5-The simply supported externally prestressed girders, which were tested up to failure shows high stiffness at early stage of loading represented by initial steep curve of the load-deflection behavior with elastic stresses in both the steel I-beam and RC deck slab. When the applied load was increased, the state of stress in the steel beam changed from elastic to elastic-plastic and the compressive stress block in the RC deck slab became inelastic. This was concluded from the shape of the P-Δ curve which showed a noticeable reduction in slope during this stage of loading. When the load approached the peak value, the beam showed rapid increases in the vertical deflection for only small increments in load. At ultimate load, the concrete of the deck slab crushed at mid-span and the whole steel I-beam reached the fully plastic condition indicating a sign of flexural failure.

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