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Comparative Experimental Study of Behaviour of Straight and Horizontally Curved Composite Bridges

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Abstract: GEOMETRIC, aesthetic, and economic considerations have led to the increased use of horizontally curved girders for highway bridges and interchange facilities which involve curved alignment. The current paper investigates the behavior of steel-concrete curved composite bridges. An experimental program was conducted to investigate the behavior of steel-concrete composite curved and straight bridges, through the comparison of the deflection and longitudinal slippage at failure state of the specimens under static load. Three different specimens were built, two were horizontally curved; one of the curved specimens with full interaction, while the other with partial interaction between the concrete and the steel girders, and the third one was straight. Each specimen has three I-girders acted parallel connected with X shape cross frames. All specimens dimension scaled as 1/10 in both length and radius of curvature, of Federal Highway Administration (FHWA) full scale model. The Models were examined in the structure laboratory of American University in Cairo (A.U.C.). The static load was applied incrementally and distributed at six points till failure. Linear Variable Differential Transducer, (LVDT) were used to measure the deflection at girders centers and the longitudinal slip. The results show that the configuration of curved geometric specimens clearly affects the pattern of yield and resistance capacity of the specimens. The maximum deflection of the straight model was in the middle girder while it was at the external girder in the curved specimens. The strength of the partial interaction specimen was 15.83% less than the full interaction model, while the straight model showed the maximum capacity of resistance.

Keywords: *composite curved bridges, steel-concrete, slip, deflection, partial-interaction.*

1. INTRODUCTION

Presently, many cities have to face the problems of space limitation for the transportation systems. One probable solution is to construct the bridges with special configurations. Horizontally curved composite bridges are among the most economical options for satisfying these demands. To develop an improved rational set of design guidelines, the Federal Highway Administration (FHWA) initiated the curved steel bridge research project in 1992. As part of this project, (FHWA) constructed a full-scale model of curved steel girder bridge at its Turner- Fairbank Structures Laboratory. This full- scale model made it possible to conduct numerous tests and collect a significant amount of data relating to the static behavior of curved girder bridge. However, relatively little information has been available on the dynamic response of curved girder bridges, and this type of information is needed before a complete design specification can be developed.

Horizontally curved steel girders offer some distinct advantages. Curved girders allow for the use of longer spans [9], which in turn reduces the number of piers, expansion joints and bearings that are required. Horizontally curved Girders more easily satisfy the demand placed on highway structures by predetermined roadway alignments and tighter geometric restrictions, particularly in urban environments. Curved bridges using curved girders are also characterized by simpler and more uniform construction details, since girder spacing and deck overhangs are generally constant along the length of the structure. Horizontally curved girders also permit the use of narrower bridges, which are more aesthetically pleasing than a series of straight girders along the chords of a roadway curve. However, the fundamental behavior of a curved structure is more complex than that of a straight one.

Curvature introduces significant torsional stresses that must be accounted for in the design. The effects of torsion due to curvature also require careful consideration during both fabrication and erection of curved members. Thus, curved girder bridges typically require more time for designing and detailing. Numerous studies have been performed on the behavior of straight and curved composite steel girder bridges. The tentative Load Factor Design criteria for curved I-girder and bridges was adopted by AASHTO and incorporated in the Guide Specifications issued in 1979.

Weiwei Lin and Teruhiko Yoda [2], survey the literature to provide and summarize important researches related to the analysis, design and construction of curved composite girder bridges. Subjects discussed in their review include different curved girder bridge configurations and their applied range, current specifications, construction issues, design methods, analytical methods, load distribution, torsional behavior, warping stresses, stability, ultimate load-carrying capacity, dynamic and seismic response, loading test, long-term behavior and design details. The literature survey presented herein mainly focuses on papers written in English, Japanese and Chinese in relation to curved composite girders. The researchers made the following comments and recommendations that deserve high priority:

The practical requirements in the design process necessitate a need for design codes of such bridges in respective countries. Further research work is required using field tests and finite-element analyses to investigate the behavior of curved composite girders at this phase and to avoid possible failures. Actual torsional behavior of such bridges through laboratory or field tests should be a research focal point. The behavior of curved bridges near ultimate load is unknown since only a limited number of publications are available on this important design aspect. Thus, the failure mechanism of a curved bridge needs to be defined. Then other countries should calculate the distribution factors subjected to different live loads for their own design codes compared with existing specifications.

Y. L. Zhang, et al [3], investigated Characteristics of Steel-Concrete Composite Box Beams under the bending-torsion couple loads. The ultimate bearing capacity, section strain, and interfacial slip of the steel-concrete composite box beams are measured. The test results show that, the fully connected composite beams mainly express bending or bending-torsion failure modes, but the partially connected composite beams are mainly sliding failure modes. The existence of the torque doesn't have great influence on the ultimate bearing capacity.

Mahvas, mohammadi, et al [4], conducted Tests of Horizontally Curved Tubular Flange Girder System. Tubular flange girders (TFG) are an innovative I-shaped steel bridge girder proposed for horizontally curved bridge systems. The increased torsional stiffness of the TFG significantly reduces the warping stresses, total normal stresses, vertical displacements, and cross section rotations for an individual curved TFG relative to a conventional curved I-girder.

Cagri Ozgu [5], proved that, in the curved I-girder bridge system, non-uniform torsion results in warping normal stresses in the flanges. Also due to torsion -in curved bridge systems- the cross-frames and/or diaphragms have the added responsibility of restraining the twisting of the girder, thereby reducing the warping stresses in the flanges and reducing the vertical deflection of the system.

1.2 AASHTO Flexural Resistance Equations:

In the curved I-girder bridge system, non-uniform torsion results in warping normal stresses in the flanges. Also due to torsion, the diaphragms and/or cross-frames become primary load carrying members in straight girder bridges. However, in curved Bridge systems, the cross-frames and/or diaphragms have the added responsibility of restraining the twisting of the girder, thereby reducing the warping stresses in the flanges and reducing the vertical deflection of the system. Warping-to-bending stress ratio (f_w / f_b).

The equation was determined to be of the following form based on a preliminary design target:

f_w / f_b of 0.25

$$S_{max} = L\left[-ln\left(\frac{Rb_f}{2{,}000L^2}\right)\right]^{-1.5} \quad (1)$$

Where S_{max} (m) is the design spacing between cross frames, L (m) is the span length of the exterior girder, R (m) is the radius of curvature of the exterior girder, and b_f (mm) is the compression flange width. Davidson and Yoo, demonstrated that there is a reduction in the elastic buckling strength of curved compression flanges due to the presence of warping stress gradient across the flange. The primary factors contributing to the amount of reduction are the warping-to-bending stress ratio in the compression flange (fw / f_b) and the relative rotational restraint on the flange provided by the web. The curvature effect was shown to be conservatively approximated by:

$$(\sigma_{cr})_{cv} = (\sigma_{cr})_{st}\left[1.0643 - \frac{0.15}{0.35}\frac{f_w}{f_b}\right] \quad (2)$$

The curvature reduction was then simplified for design use and defined in terms of the radius of curvature, R, and the cross-frame spacing,l:

$$\boldsymbol{\psi_f} = \left[1.05 - \frac{l^2}{4Rb_f}\right] \quad (3)$$

So that,

$$\left(\frac{b_f}{t_f}\right)_{cv} = \left(\frac{b_f}{t_f}\right)_{st}\sqrt{\psi_f} \quad (4)$$

Where σ_{cr} is the critical stress, t_f is the flange plate thickness, "cv" and "st" refer to curved and straight (flat) panels, respectively.

Although the reduction equation was developed for potential design use, it was noted that the use the equation would result in negligible increase in the required flange thickness for typical bridge curvatures. Results from the flange buckling analyses using the dimensions of the FHWA-CSBRP test frame agreed. Geometric nonlinear analyses of curved I-girder web panels demonstrated that the presence of curvature effectively reduces the contribution by the web to the vertical moment carrying capacity of the curved section over that of the straight girder with comparable cross-section dimensions. Predictor equations were developed to estimate the amount of "bulging" displacement in the web,

$$\delta_{max} \approx \frac{\alpha h_c^4 \sigma_m 12(1 - V^2)[DAF]_d}{Et_w^2 R} \quad (5)$$

where α is a constant that depends on loading and support conditions, h_c is the height of the panel in compression, σ_m is the maximum stress at the top of the panel, and consistent units must be used. It was demonstrated that this equation can be reduced to a simpler form for a steel I-section using conservative values of $\alpha = 0.00651$ and [DAF] d = 3.0, and using the AASHTO symbols f_b (ksi) for bending stress and D_c for depth of the web in compression:

$$\delta_{max} \approx \frac{7.35(10^{-6})D_c^4 f_b}{t_w^2\, R} \tag{6}$$

The units for D_c, tw, and R should be consistent.

2. Design, Fabrication, and Testing of specimens:

An experimental research program was conducted at structure Laboratory, American University in Cairo (AUC). The tests included three varied specimens: two ware curved composite bridges; one of them with full interaction and the other with partial interaction between concrete deck and steel girders, the third specimen was straight composite bridge with full interaction between concrete deck and steel girders. The tests examined the behaviour of the composite curved and straight bridges under static load. This research discusses detailed information of the experimental program, including, design and detail of the specimens and displacement results in vertical and longitudinal direction under static load.

2.1 I-girder Dimensions and Details:

The bridge used in this project has been scaled as 1/10 length and radius of curvature of FHWA full scale model which was used for long-term tests, Fig. (1). the composite section then designed according to AASHTTO 2003 guide specification. The design considerate that the natural axis acted at the top flange. The bridge was one span simply supported, with the longest span of 2.87 m, this span was chosen because it can be easily accommodated in the lab, and the limitation of fund for material and fabrication cost.

Three I-girder section of the bridge was used for each specimen; the girders used in the design were IPE 160 with cover plate welded under the bottom flange the section properties provided in Table (1). No bearing stiffeners were added, because it is assumed that the girder web alone was thick enough to satisfy the Strength limit state provisions for web yielding and web buckling. Three intermediate cross frames were installed spaced at 650 mm intervals along the center girder. The cross-frame members used in the design were steel grade 37 with 40*40*3 mm angles and the X-type configurations were chosen. Shear transfer between the girders and the concrete was provided by one and two rows - for partially interaction and full interaction respectively - of grade 37 with 13 mm diameter x 70 mm long mild steel stud-type shear connectors spaced 30 mm apart on center, Studs were spaced longitudinally at (281-318-324 mm) on the center along each length of spans.

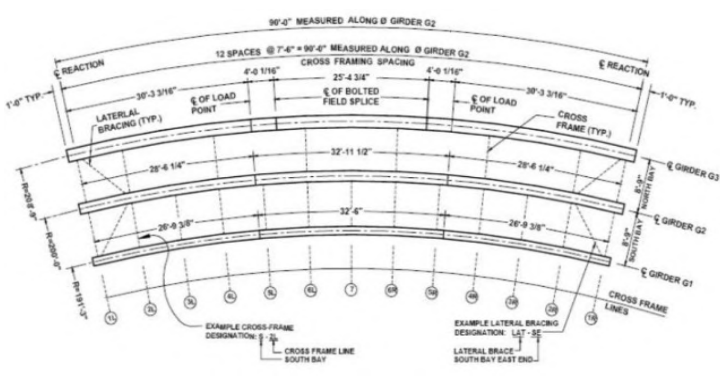

Fig.1. FHWA Curved Bridge Testing Model

2.2 Girders Fabrication and Materials:

All of the steel sections for the test specimens were fabricated as flows: The IPE160 I-girder members have been cut to the specified (2.78 m, 2.6 m and 2.33 m) length, with the 150 mm drops included for material property testing. Then the girders were curved by roller-machine by radii of curvature 6360 mm, 5673 and 4560mm respectively, then girders were set at the required 0.5945m spacing. Once the girders were in place, end diaphragms were installed and then any skew in the frame was removed by checking that both diagonals of the steel frame were equal and adjusted if necessary. The shear connector were welded on the top flanges after the angle cross frames are connected to the adjacent curved I-beams. Coupons were cut from the girder drops and tensile tests were conducted to determine girder steel properties Fig (2- 4).

Table 1. Properties of Steel I-girder

Section type	IPE 160
Depth mm	160
Width mm	82
T_f mm	7.4
t_{web} mm	5
Area cm^2	20.136
Weight kg/m	15.3
Section type	IPE 160

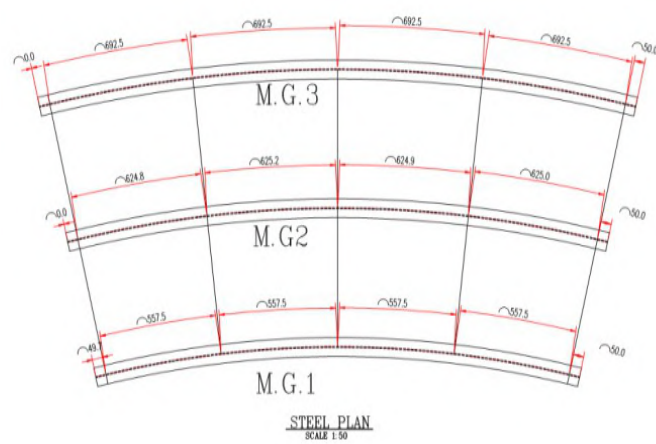

Fig.2. Steel Frame Plan

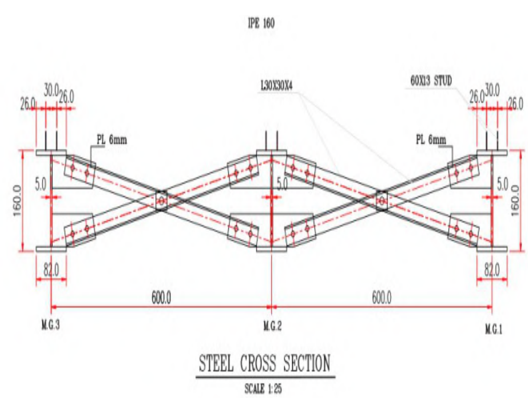

Fig.3. Steel Frame Cross Section

Fig.4. Steel Frame of Curved Specimen

2.3 Deck Design and Details:

The deck is 100 mm thick cast-in-place concrete slab, composed of concrete having a design compressive strength (f_{cu}) of 38 N/mm^2. The concrete elastic modulus for the design analysis is taken as 25 MPa. The slab reinforcing is set approximately at the base requirements of the AASHTO empirical method, two layers of steel grade 37 uncoated (black) reinforcing steel in a top mat and a bottom mat.

The final rebar layout utilized both ϕ6 and ϕ8 bars placed as shown in the cross section given in Fig. (5 - 8). The target mix concrete design for each specimen ratio was 1: 2.5: 3.3 and water cement ratio 0.65. Description of the composite bridge and girder cross section dimensions are presented in Table (2).

Table 2. Dimensions of the Specimens

Number of girders	Three I- girders for each specimen
Length of spans	2.87-2.6-2.33 m
Spacing between girders	0.5945 m
Thickness of concrete deck	10 cm
Spacing between shear connecter	281-318-324 mm
Radius of curvatures	(6360, 5673 and 4560mm)
Section of I girders	IPE160
Cover plate	150*14 mm

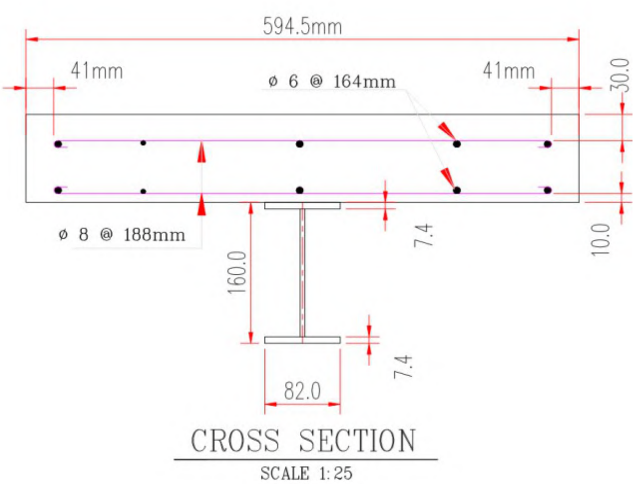

Fig.5. Composite Cross Section

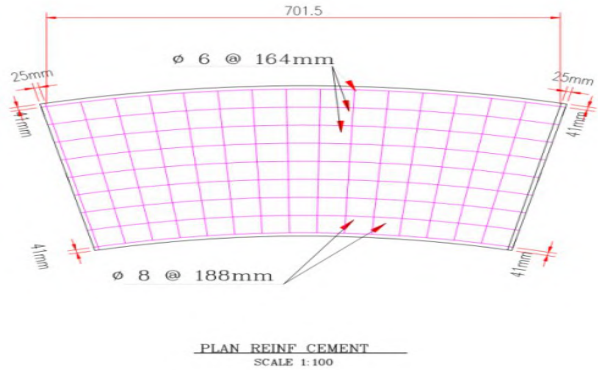

Fig.6. Deck Reinforcement Plan

Fig.7. Ditributions of Reinforcement and Form Work

Fig.8. Placing Fresh Concrete

3.1 Test Setup:

The test setup for all parts of this study is the simply-supported bridge test specimens, which are full interaction curved bridge with 2.87 m span length, partially interaction curved bridge with 2.87 m span length and full interaction straight bridge with 2.6 m span length. All construction, instrumentation, and testing was completed at the structure laboratory in AUC. The three test specimens were tested using frame with a 500 Ton capacity with Static actuator (200 Ton) for final static test and two hydraulic power systems Fig. (12) Each test specimen was simply supported on two braced rigid-beams by a system of steel plates and rollers. The applied vertical load was distributed on six points on the tested models by system of steel beams and plates Fig .(9 - 11).

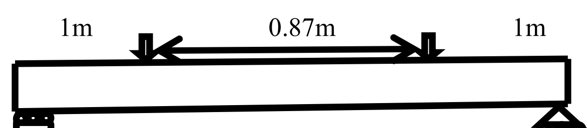

Fig.9. Load Position Elevation Longitudinally

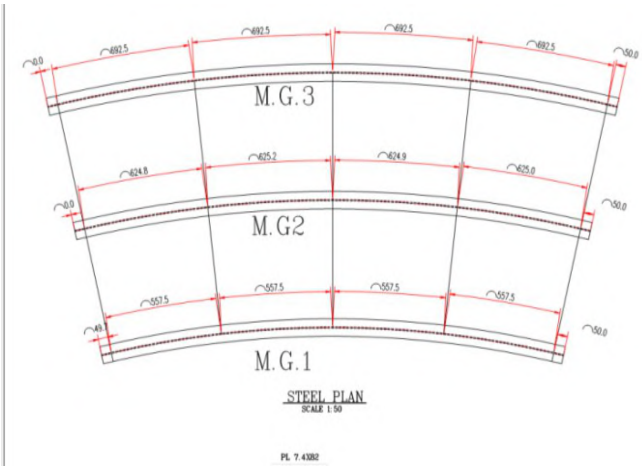

Fig.10. Plan of Model Load Points

Fig.11. 500 Ton Steel frame and two Braced Rigid Beam

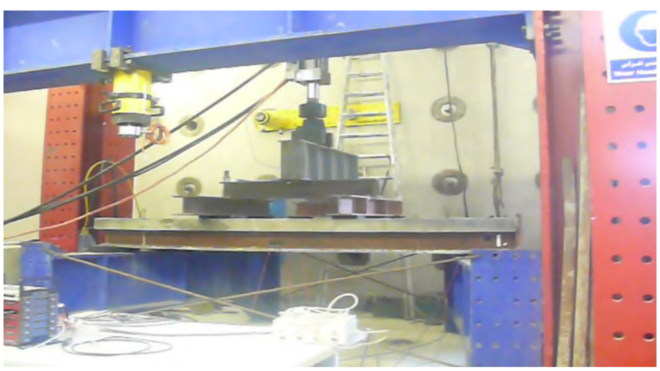

Fig.12. 500 Ton Steel Frame and two Braced Rigid Beam

3.2 Testing Instrumentation

To acquire data relating to displacements over the course of the testing, five linear variable differential transducer (LVDT) sensors were supported vertically, three of them against the central point of the longitudinal axis of each girder, and two against the two ends of the central girders to measure the deflections during the tests, and three (LVDT) were used to measure the longitudinal slippage between concrete deck and steel I-girders Fig. (13 and 14). Data was collected from a load cell, which provided measurements of actuator deflection and load to be used in analysis. All data from LVDT's, the hydraulic actuator LVDT and load cell during static testing were collected to the data acquisition system in conjunction with the computer program Strain Smart installed on a lab computer, Fig. (15) Once collected, raw data for the static tests reduced into a Microsoft Excel spreadsheet for analysis.

Fig.13. LVDT to Meager Horizontal Slip

Fig.14. Curved model Instrumentations-LVDT to Meager Girders Deflections

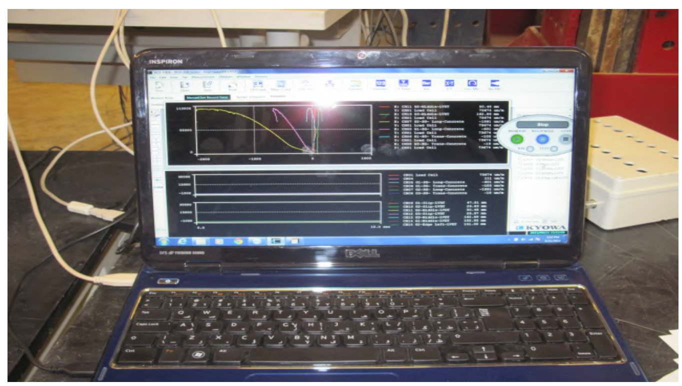

Fig.15. Data Acquisition System

3.3 Static Testing Procedure:

Static test was conducted on the test specimen to determine the residual capacity (or plastic moment capacity) of each type of bridges. To generate the amount of load necessary for this test, a 200 Ton capacity static hydraulic actuator was utilized. The static actuator was mounted to a steel load frame at locations directly above the test specimen steel girders, and the load applied through the distributing system setup. All instruments were zeroed following the seating of the specimen. The test was conducted, and load was then applied in increments till collapse.

4. Results and discussion:

In this section the results of vertical displacement at center of girders are presented for curved and straight specimens. The displacements were measured by LVDT positioned vertically at the expected maximum deformation points. The results and its discussions are in the figures (16 - 20), and tables (3 and 4).

4.1 Strength and Deflections response:

4.1.1. Deflections Interior girders for all specimens

Although the 50% of the total load is applied directly on the middle girder and each of the two other girders carry 25% of the load in both curved and straight specimens, the maximum vertical displacement in the curved specimen happened in exterior girder G3, but the maximum deflection was happened in the middle girder G2 in straight specimen. In the case of the straight specimen and the larger deflection is observed in the center of the middle girder. The deformation and the behaviour in the other two exterior girders are similar. Linear behaviour is observed until 1300 KN load limit at middle girder. In the case of curved specimens, the behaviour is quite different. In the case of full interaction between concrete and steel girders: With regard to the internal girder the behaviour of deformation with load remains linear until 1000 KN load limit while the submission stage began after that with the increasing of the load. In the case of partial interaction between concrete and steel girders, the interior girder was in linear manner till 800 KN. All interiors girders are in yield stage when the test stopped because of concrete crushing and failure in exterior girder in curved specimens and middle girder in the straight specimen.

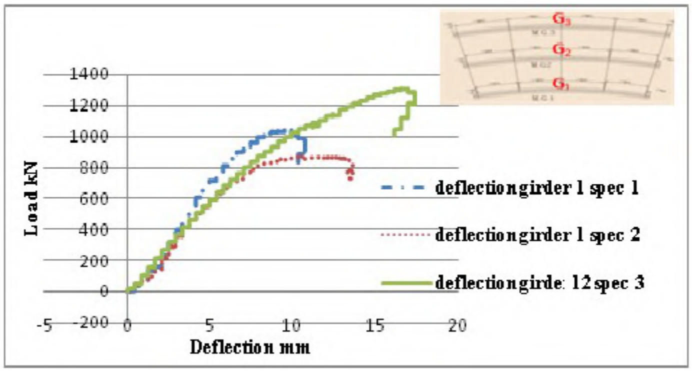

Fig.16. Deflection at Center of the Interior Girders for all Specimens

4.1.2 Deflections at middle girders for all specimens

Fig. (17). Shows the deflection at middle girders for all specimens. In the full interaction curved specimen at middle girder deformation behaviour with respect to the load remains linear until 900 KN then turned to submission stage. In the case of partial interaction curved specimen at the middle girder the deformation is linear until the load reached 790 KN. In the straight specimen the yield stage started in the middle girder before other girders, the deformation was linear until load reached 1284 kN.

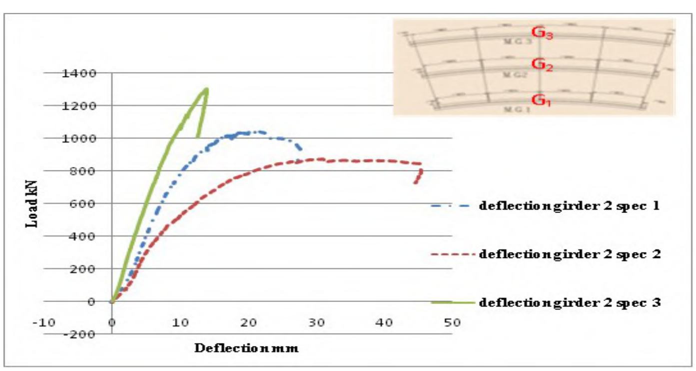

Fig.17. Deflection at Center of the Middle Girders for all Specimens

4.1.3 Deflections at Exterior Girders for all Specimens

Fig. (18). shows the deflection at exterior girders G_3 for all specimens. In the full interaction curved specimen at exterior girder deformation behaviour with respect to the load remains linear until 800 KN then turned to submission stage and take longer before failure.

In the case of partial interaction curved specimen at the exterior girder switch to submission stage at 600 KN of the load, and stage of submission takes longer before the point of collapse. In the straight specimen the exterior girder yield stage started at the same time of the interior girder at load 1300 kN. The maximum deflections for full interaction specimen (spec1) and partial interaction specimen (spec2) curved happened at the exterior girder, for straight specimen (spec3) happened at middle girder.

The static test results shows that straight specimen have higher strength, it was failed with greater plastic load (1306 kN) and smaller vertical deflection (13.9752 mm) compared with the full interaction curved specimen plastic load (1042 kN) and maximum deflection (82.919 mm). The partial interaction specimen was less strength than full interaction curved specimen by 15.97%, and vertical displacement increased by 39%. The displacement in the straight model was less than curved model by 83.2%. Fig. (19). Shows the comparison of the maximum vertical displacements for three specimens. (See Table (3 and 4). The mode of failure of the straight specimen compared with the mode of failure of curved bridges and the behaviour of three girders in each curved specimen were variable, although the two interior I-girders were in linear phase the exterior I-girder becomes non-linear.

This results Confirm that the exterior girder carries a greater part of the load. The maximum torsion moment observed at the exterior girder. The curved bridges displacement was detected at three dimensions because the lateral and torsion moments effects on the deformation of the bridge while the displacement in the straight specimen was vertically in origin.

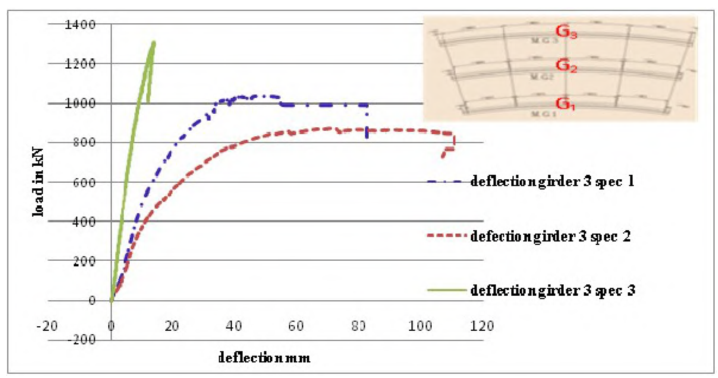

Fig.18. Deflection at Center of Exterior Girders for all Specimens

Table 3. Maximum deflections of full interaction and partial interaction curved specimens

Specimen	Interior girder G_1	Middle girder G_2	Exterior girder G_3	Load kN
Full interaction curved Specimen	10.74814mm	27.767mm	82.91952mm	1042.012
Partial interaction curved specimen	13.64187mm	45.53399mm	110.8699mm	875.54
Variation percentage	21.26%	39%	25.22%	15%

Table 4. Maximum deflections of full interaction curved and straight specimens

Specimen	Interior girder G_1	Middle girder G_2	Exterior girder G_3	Load kN
Full interaction curved Specimen	10.74mm	27.76mm	82.91mm	1042.012
Full interaction straight Specimen	13.98mm	17.36mm	13.97mm	1306
Variation percentage	22.73%	37.46%	83.15%	20%

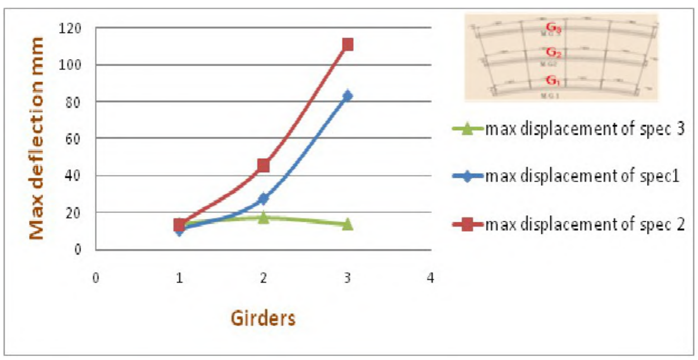

Fig.19. Comparison Maximum Deflections for all Specimens

Fig. (20) Shows the flexural mode of failure at exterior girder G3 of full interaction curved specimen, after full load acted. Concrete deck crushed and the middle cross frame bended strongly. The shear mode failure happened at the end diaphragm of the middle girder.

Fig.20. Torsion of exterior girder and buckling in middle cross frame for full interaction curved specimen

Table 5. Maximum slip of all specimens

specimen	Interior girder	Middle girder	Exterior girder	Maximum load kN
Curved full interaction Specimen	-0.778mm	-5.71mm	-11.36 mm	1042.012
Curved partial interaction Specimen	-4.089mm	-8.381mm	-21.057mm	875.537
Straight full interaction Specimen	-2.070mm	-3.310mm	-1.469mm	136.096

The maximum slip in the partial interaction specimen was 21.05 mm at exterior girder which mean that disintegration occurred between concrete and shear connector clearly, but in the full interaction specimen the maximum slip was 11.36 mm at exterior girder G3 which is less than the value of partial interaction specimen by 46%. In the straight full interaction specimen, the maximum slip was 3.31 mm, at the middle girder, which is less than maximum value in curved specimen by 70%, this mean that the straight specimen was stiff more than curved specimen. In general the amount of slip between the concrete and steel increased in the case of the curved specimens compared with the straight one, also it was increased in case of partial interaction between the concrete and steel. Fig. (21 and 22) and table (5).

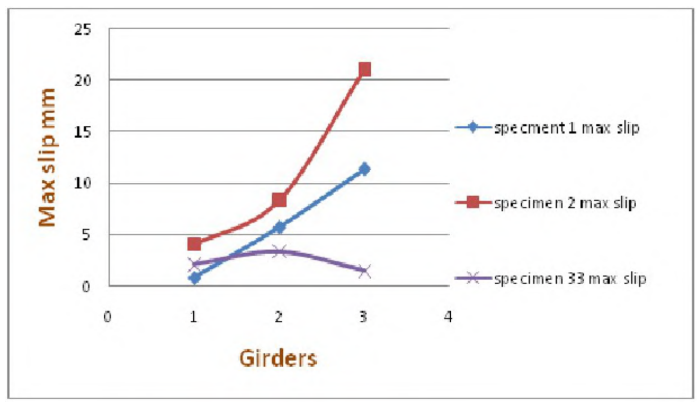

Fig.21. Comparison between maximum slip of all specimens

Fig.22. Slip in mm at exterior girder after static test of curved partial interaction specimen

CONCLUSIONS

The ultimate goal of the experimental work on the straight and curved (full interaction and partial interaction) composite specimens, is to measure the maximum resistance and study the behaviour of the curved bridges. Throughout the results of deflections and slip the following conclusions were made:

- For curved specimens, the majority of the load distributed towards the exterior girder.
- Results of static test showed that the resistance of composite straight specimen was higher than the composite curved specimens. And the partial interaction composite curved specimen resistance was less than full interaction composite curved specimen by 15.97%
- Although the maximum load was carried by in the middle girder in both curved and straight specimens, the maximum vertical displacement in the curved specimen happened in exterior girder G3, but the maximum deflection happened in the middle girder G2 in straight specimen.
- Flexural deformations happened firstly in the exterior girder in curved composite specimen (full interaction and partial interaction).
- The maximum slip in the partial interaction specimen was 21.05 mm at exterior girder, which mean that disintegration occurred between concrete and shear connector clearly, but in the full interaction specimen the maximum slip was 11.36 mm at exterior girder G3 which is less than the value of partial interaction specimen by 46%. In the straight full interaction specimen, the maximum slip was 3.31 mm, at the middle girder, which is less than maximum value in curved specimen by 70%, this mean that the straight specimen was stiff more than curved specimen.
- The curved bridges displacements were detected at three dimensions because the lateral and torsion moments effects on the all deformation of the bridge while the displacement in the straight specimen was vertically in the origin.
- The mode of failure of the straight specimen compared with the mode of failure of curved bridges and the behavior of three girders in each curved specimen were variable, although the two interior I-girders were in linear phase the exterior I-girder becomes non-linear. This results Confirm that the exterior girder carries a greater part of the load.
- Since the curved specimen strength is less than the straight specimen, and its deformations are more due to the torque and the lateral moments, imperatively we must include these affects in the design, that by increasing the dimensions of the section to accommodate the additional distortions resulting from the configuration variation.

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