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Anticipated and actual performance of composite girder with pre-stressed concrete beam and RCC top flange

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Abstract. Load testing is one of the important tests to determine if the structural elements can be used at the intended locations for which they have been designed. It is nothing but gradually applying the loads and measuring the deflections and other parameters. It is usually carried out to determine the behaviour of the system under service/ultimate loads. It helps to identify the maximum load that the structural element can withstand without much deflection/deformation. It will also help find out which part of the element causes failure first. The load-deflection behaviour of the road bridge girder has been studied by carrying out the load test after simulating the field conditions to the extent possible. The actual vertical displacement of the beam at mid span due to the imposed load was compared with the theoretical deflection of the beam. Further, the recovery of deflection at mid span was also observed on removal of the test load. Finally, the beam was checked for any cracks to assert if the beam was capable of carrying the intended live loads and that it could be used with confidence.

Keywords: pre-stressed concrete; road bridge girder; load test; composite girder; mid span; deflection; recovery

1. Introduction

Pre-Stressed Concrete (PSC) girders are being used for long span bridges, flyovers, etc., as they posses certain properties which cannot be expected in ordinary reinforced concrete. The advantages of using PSC girders always outweigh those of steel structures. The construction of a road bridge consists of pre-stressed concrete (PSC) girders and a reinforced cement concrete deck slab. Hence, it is always advisable to check the load carrying capacity of the girder cast, before erection at the required locations. The deflection at mid span is noted down and compared with the theoretical value to find out if the beam could be utilized effectively. The load testing should be carried out at least on one percent of the girders cast after selecting them randomly. This paper deals with the details of the load test conducted on the composite girder that has a post tensioned pre-stressed concrete girder and a reinforced cement concrete slab cast over the top flange of the PSC girder, simulating a deck slab.

2. Review of literature

Akshatha, Katta *et al*. (2015) have carried out an extensive research to study the effect of corrosion on the strength property of reinforced concrete beams. The load deflection behavior of the corroded beams made of reinforced concrete has been analyzed using ‘ANSYS’ software package and compared with the values obtained in the laboratory under two point loading method.

They have concluded that the results obtained from ‘ANSYS’ are almost same as the experimental values and that the Finite Element Modeling could address corrosion effects on load-deflection behavior. The measured corrosion current density values show a variation within ±5% as compared to the actual values.

Kottari and Benson Shing (2014) have estimated the long term pre-stress losses in post tensioned girders using two methods, namely refined method & simplified method. It has been found out that the strength of concrete, the amount of non pre-stressed steel, the relative humidity and the age of concrete are very much important in the assessment of pre-stress losses. The incremental shrinkage strain right from the time of post tensioning is as important as ultimate shrinkage strain.

A field load test has been carried out by Naser and Wang (2012), to the anchorage beam, the steel deviation block devices, and the steel deviation cross beam which clearly depict that the values of the strains, the stresses, and the deflection are less than the respective allowable limit values in the requirements and that the anchorage devices have enough strength, and the working state is good after the tension forces are applied to the external pre-stressing tendons.

Dang, Murray *et al*. (2014) have carried out tests and correlated strand surface quality to the transfer length. In order to assess the bonding capacity of the pre-stressing strands used in pre-stressed concrete, a standard test for strand bond (STSB) has also been developed. A prediction

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equation for transfer length has also been proposed that incorporates strand bond quality as determined by STSB.

The test results of six existing pre-stressed concrete bridges obtained by Cai and Shahawy (2004) were used to evaluate analytical methodologies. These bridges cover different span lengths, number of lanes, skew angles, strains, load distribution factors, and ratings predicted by finite-element analyses and AASHTO code specifications and are compared with those from measurements. The comparison reveals a significant difference between the analytical and test results which is due to the effects of many field factors. Due to these field factors, existing bridges are different from idealized calculated models. Some field factors have a larger effect on the maximum strain than on the load distribution factor. Parametric studies of the effects of diaphragms, stiffness, and skew angles on the load distribution and maximum strain have also been conducted.

Elabadry, Ghali *et al*. (2014) found that the model for prediction of creep in codes and design guides are mainly based on tests in which the load is sustained for a number of years, rarely exceeding 10. However, most models predict that creep reaches its extreme after a span of 30 years.

Barrios and Ziehl (2012) in their study focused on the performance and validation of the 24 hour load test (24h LT) method and the cyclic load test (CLT) method as applied to full scale lightweight and normal weight self consolidating pre-stressed concrete girders. They examined data obtained from the four point loading tests of six full scale T-girders and applied the current criteria from these methodologies to evaluate the presence of damage and structural integrity. The experimental results indicated that the recovery criteria of the 24h LT method were insensitive to damage and hence did not provide a satisfactory integrity assessment of the members. Also, the permanency and repeatability criteria of the CLT were insensitive to damage for the girders studied. The global integrity parameter (GIP) based on the deviation from linearity criterion from the CLT has been proposed for the quantitative assessment of the level of damage in pre-stressed concrete girders, and the results indicated good correlation with experimental data.

Kumar, Dubey *et al.* (2014) have done a study on the ultimate strength of epoxy repaired braced partial infilled reinforced concrete frames and found out that the strength is restored up to a significant level for deficient frames. They also concluded that the frames repaired with epoxy have maximum deflection as compared to bare reinforced concrete frames. It is also found out that after the application of epoxy on reinforced concrete frames, the stiffness has increased.

Pape and Melchers (2011) have presented the effects of corrosion on 45 year old pre-stressed concrete bridge beams. For a detailed evaluation, three of the beams were selected and load test was conducted. The most severely corroded beam failed at a load which was just fifty percent of the load that the sound beam withstood. It was also found out that there was severe loss of ductility. There was a clear correlation between the degree of corrosion and the load carrying capacity of the beam; it was pointed out in the study.

Xiao, Li *et al*. (2014) constructed a bridge model of 9.4 m long full scale glued laminated bamboo (glubam) to duplicate a prototype truckload bridge. The soffit of the 600 mm deep and 120 mm wide glubam girders was further enhanced with Carbon Fiber Reinforced Polymer (CFRP) sheets. The mid span deflection of the bamboo girders were obtained over the course of 3.7 years from creep tests under the gravity load of the bridge. The results indicated that the average creep deflection of the girders after 3.7 years was within acceptable range. It was found that the creep deflection was affected by seasonal humidity and temperature. After 3.7 years of creep loading, the bridge model was loaded with sand bags and cast iron weigh blocks until failure. The final failure mode was identified as the flexural failure of a section where a finger joint of one layer of the glubam girder was located.

Experimental investigations by Abeles (1966) have shown that micro cracks develop when the tensile stress reaches 3 N/mm^2. On further loadings, cracks become visible when flexural tensile stresses are between 3.5 and 7 N/mm^2. The higher values correspond to beams with well bonded steel distributed close to the tensile face, in the pre-tensioned beams. The load deflection curve is almost a straight line up to the stage of visible cracking and beyond that, the deflections increase at a faster rate which is attributed to the stiffness reduction of the beam. In the post cracking stage, the behavior of the beam is almost same as that of reinforced concrete members. The instantaneous deflection in the post-cracking stage is obtained as the sum of the deflection up to the cracking load based on gross section and beyond the cracking load considering only the cracked section.

Indian Road Congress (2010), as per clause 6.6.2., for girder bridges, the load for rating should be taken as the least of (i) the load causing a deflection of 1/1500 of the span in any of the main girders for simply supported spans (or) for cantilever span, the load causing a deflection of 1/800 of the cantilever span in any of the main girders, (ii) the load causing tension cracks of more than 0.3 mm in any of the girders for normal cases and 0.2 mm for structures exposed to very severe and adverse conditions, (iii) the load causing appearance of visible diagonal cracks of width more than 0.3 mm for normal cases and 0.2 mm for structures exposed to very severe and adverse conditions or opening/widening of existing cracks close to the supports in concrete girders, (iv) the load at which recovery of deflection on removal of load is not less than 80% for reinforced cement concrete elements and 90% for pre-stressed concrete girders.

As per clause 5.2 of Indian Road Congress (1999), for bridges designed for IRC standard loadings, criterion for load testing of PSC beams, after retention of test load for 24 hours, the minimum recovery of deflection after removal of test load should be not less than 85%. A general acceptance criterion for the behavior of a structure under test load is that it shall not show “visible evidence of failure” which include appearance of cracks of width more than 0.3 mm, spalling or deflections which are excessive and incompatible with safety requirements.

Elham Alizadeh and Mehdi Dehestani (2015) have

Fig. 1 Road bridge girder - skeleton reinforcement and sheathing pipes

Fig. 2 Road bridge girder - after casting

carried out research on hat shape composite girder with concrete slab by doing the analysis using software, ABAQUS to find out certain parameters which otherwise would have required a huge expenditure for carrying out the experiments to find out the same parameters. The results thus obtained in the finite element (FE) analysis were validated by those obtained in the experiments and then the parametric studies were performed. The flexural behavior and failure modes of box girders made of Glass Fibre Reinforced Polymer (GFRP) with and without concrete slabs were predicted by the finite element model and they were almost in agreement with the experimental results. The specimen with concrete slab failed at a load of 430 kN while the predicted load was 425 kN. It has been found out that the ultimate load and the stiffness of the composite girder increases with the increase in the modulus of elasticity of hat shape section and GFRP plate. The increase in the load capacity is as high as 56% when the modulus of elasticity increases from 26.2 GPa to 100 GPa. It is also pointed out that increasing the strength of concrete will increase the ultimate load of the girder despite the fact that it will have only a negligible effect on stiffness. When the girder is completely filled with concrete, it increases both the stiffness and the load carrying capacity. Though providing a hole in the section reduces the load capacity, the strength to weight ratio increases. The research has also revealed that changing the material from GFRP to steel has increased the stiffness and also the ultimate load to a great extent even as the strength to weight ratio decreases.

Samaaneh, Sharif *et al*. (2016) have carried out experiments by bonding the carbon fibre reinforced polymer (CFRP) sheet of different thicknesses over the negative moment region of the concrete slab of the continuous composite girder. It is needless to say that the stiffness of girder is directly proportional to the thickness of CFRP and the use of the same will reduce the moment redistribution. They have done the numerical analysis using ABAQUS and a comparative study has been made with the experimental results. They have suggested providing CFRP sheet for the entire length of the negative moment region with a development length of 150 mm. It is also concluded that the ultimate capacity of the girders with CFRP development length extended more than 200 mm beyond the inflection point did not yield satisfactory results on the ultimate strength. The upshot of debonding of the CFRP on the load carrying capacity has also been determined by conducting a parametric study using finite element method.

3. Details of PSC road bridge girders

The PSC girders of M40 grade have been cast & pre-stressed by post tensioning method. The entire pre-stress is applied to the precast girder only. After grouting and sufficient curing, they are launched to the required location and placed atop the pedestals on the piers. Subsequent to the erection of the PSC girders, a reinforced cement concrete deck slab is cast over the erected girders. The

Fig. 3 Load test set up

superimposed loads are carried by the composite girders. For each PSC road bridge girder, 2.5 m wide and 200 mm thick RCC slab forms part of the top flange of the composite girder.

4. Salient points in stressing operation

- Cable No. 3 (near bottom flange) was in the first stage while Cable Nos. 1 (near top flange) & 2 (in the middle) were in that order in second stage.
- First stage stressing was done after 5 days or after concrete attained a cube crushing strength of 23 N/mm^2 whichever was later.
- Second stage stressing was done after 18 days or after concrete reached a cube crushing strength of 37 N/mm^2 whichever was later.
- Cable 1 had 6 strands while Cable 2&3 had 8 and 9 strands respectively.
- All cables are of 12 T 13 pre-stressing system with 66mm ID sheathing and are uncoated stress relieved low relaxation seven – ply strands conforming to IS: 14268 – 1995. The reinforcements were locally adjusted wherever they interfered with sheathing.
- Stressing was done from both ends simultaneously. Jack pressures were derived suitably considering the ram area.
- Cable profile consists of straight inclined portion behind the anchorage and then vertical deviation through a second degree parabola. The start of the parabola is the point on the cable nearer to the support and the end point nearer to mid span. All horizontal deviations are by means of a pair of reverse curves of second degree parabola.
- The jack pressure & elongation should tally with each other to within 5% after correction for actual jack efficiency, area of strand and modulus of elasticity of high tensile steel wires. If the elongations are obtained earlier than at the desired jack pressure, then continue stressing further till the desired jack pressure or a maximum elongation of 5% whichever is earlier, is obtained. If the jack pressure is reached earlier, then the stressing shall be continued further to a maximum of 5% extra jack pressure or attaining the required elongation whichever is earlier. If the designed jack pressure / elongation are not obtained even after 5% increase in elongation / jack pressure, redesigning of the girder should be done.

5. Load testing arrangement

Prior to commencement of load test, the following arrangements were made at site:

- Pedestals were constructed and cribs were erected on hard base at girder's support region, and at mid span for placing of dial gauge.
- The position of the bearing pads was marked on the pedestals and bearings were placed at appropriate locations.
- The composite girder was placed over the bearing pads on the pedestals. The steel sections, mild steel sheets, etc., were pre-weighed and recorded.

6. Equipments used for the load test

1. Hydraulic jacks 1000 KN capacity (double acting)-2 Nos.

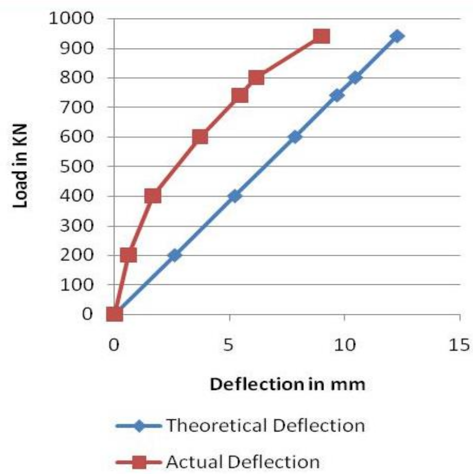

Fig. 4 Load- deflection curve during loading process

Table 1 Observations during load test

Sl.	Pressure				During Loading			During Unloading		
No.	Load in Kilo Newton				Time of loading	Deflection in mm		Time of Un loading	Deflection in mm	
		Required in N/mm^2		Actually applied Pressure (N/mm^2)	On Day 1 (Hrs.)	Theoretical	Actual	On Day 2 (Hrs.)	Theoretical	Actual
1.	0	0.00	0.00	0.00	17.00	0.00	0.00	18.00	0.00	0.20
2.	200	$\frac{200000}{39741\times2}$	2.51	2.5	17.00	2.61	0.60	17.56	2.61	1.58
3.	400	$\frac{400000}{39741\times2}$	5.02	5.0	17.03	5.22	1.66	17.52	5.22	3.27
4.	600	$\frac{600000}{39741\times2}$	7.53	7.5	17.06	7.83	3.72	17.47	7.83	5.32
5.	740	$\frac{740000}{39741\times2}$	9.31	9.5	17.09	9.65	5.44	17.41	9.65	6.25
6.	800	$\frac{800000}{39741\times2}$	10.04	10.0	17.12	10.44	6.16	17.35	10.44	7.67
7.	940	$\frac{940000}{39741\times2}$	11.83	12.0	17.19	12.26	9.01	17.30	12.26	10.45

Fig. 5 Deflectometer placed on a firm support and below the girder at mid span

2. Pressure gauges
3. Hydraulic power pack (electrically operated)-1 No.
4. Dial gauge: Plunger type, Range: 0-25 mm, Least count: 0.01 mm
5. Jack ram diameter: 225 mm, sectional area=39741 mm^2

7. Test methodology adopted

The following is the stepwise procedure adopted for conducting load test which is as per standard practice:

◦ The girders were examined thoroughly for any cracks or imperfections. Dial gauge was fixed at mid span to ensure that the plunger of dial gauge was having a firm contact with the bottom of girder.

◦ The superimposed load component (inclusive of self weight of ISMB, Rails, ISMC and MS sheets) on the girder was calculated and the same was imposed on the girder through sand bags corresponding to Kent Ledge load.

The initial reading of the dial gauge, and time were recorded. Kent ledge load was applied uniformly over the entire area of platform.

The readings in the dial gauge were noted for every one hour for 24 hours. Observations were made for any imperfections in the composite girder.

After application of full test load on the girder, the instantaneous deflection was recorded. The deflection was monitored for 24 hours under sustained load by noting down the readings in dial gauge at every 1 hour interval.

At the end of 24 hours after loading, the dial gauge reading was recorded and the maximum deflection was worked out after considering the support settlement.

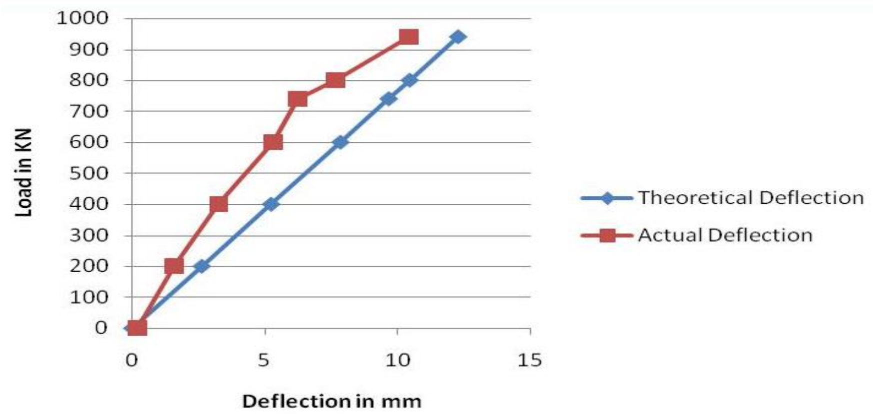

Fig. 6 Load- deflection curve during unloading process

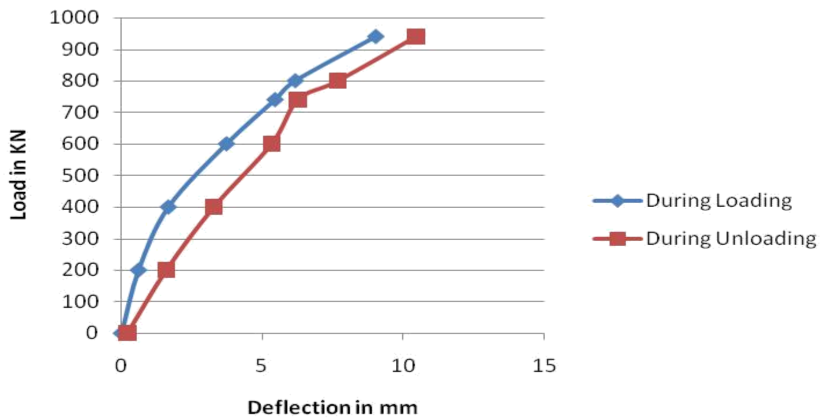

Fig. 7 Load- deflection curves during loading & unloading process

Recovery of deflection at the end of 2
4 hours of removal of test load was also found out.

8. Acceptance criteria

The minimum percentage recovery of deflection as per clause 5.2 of Indian Road Congress SP 51– 1999 at 24 hours after removal of test load is 85%. If within 24 hours after the removal of the load, the residual deflection measured at any point of the member is more than 15% of the corresponding maximum deflection observed during the test, the test shall be repeated a second time after 72 hours. The member is considered to have failed the test if the residual deflection is greater than 10% of maximum deflection observed during the second test. The maximum deflection is 1/1500 of span from the true horizontal level of the girder as per clause 6.6.2 of Indian Road Congress SP 37-1991. Also, the member shall be deemed to have failed the test if the maximum deflection exceeds the theoretical deflection.

9. Load test

The load test has been carried out on the composite girder consisting of one PSC girder with 2.5 m wide RCC slab cast atop the girder simulating the field conditions. For the load test, two point loads of equal magnitude are applied with one each at the middle third points (centre of the intermediate diaphragms) of the span. The test loads applied are of such a magnitude that the resulting bending moment at the mid span does not exceed the maximum bending moment caused by 1.25 times design superimposed loads.

The load test is carried out in stages of up to 1.25 times the design super imposed load. The total Kent ledge load arranged was 1530 kN. The total maximum test load applied in this test was 940 kN and the corresponding theoretical deflection was 12.26 mm. The load was applied on to the test girder as two loads of same magnitude at two points, one each at the location of the intermediate diaphragms by means of 2 numbers of hydraulic jacks of 1000 kN capacity each, operated in tandem. The test load was increased in stages from zero to the maximum test load of 940 kN (total) and this maximum load was maintained for 24 hours and thereafter the load was reduced to zero in stages. At each of the loading and unloading stages, the deflection of the girder at mid span was recorded using dial gauge.

The actual and theoretical deflection values for various loads during the process of loading have been plotted and it is found that the actual deflection is far less than the theoretical values for all loads.

The actual and theoretical deflection values for various

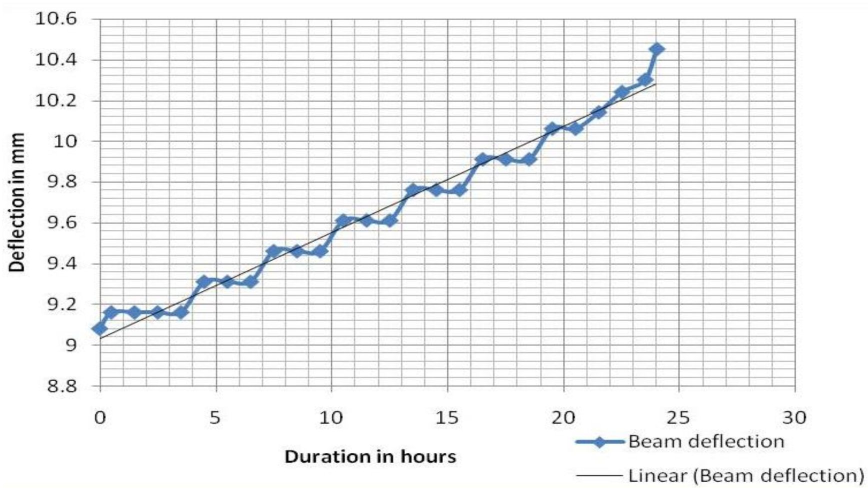

Fig. 8 Deflection during 24 hour period

Table 2 Load test readings

Day	Time of Loading (Hrs.)	Theoretical Deflection (mm)	Actual Deflection (mm)	Day	Time of Loading (Hrs.)	Theoretical Deflection (mm)	Actual Deflection (mm)
Day 1	17.30	12.26	9.08	Day 2	6.00	12.26	9.61
Day 1	18.00	12.26	9.16	Day 2	7.00	12.26	9.76
Day 1	19.00	12.26	9.16	Day 2	8.00	12.26	9.76
Day 1	20.00	12.26	9.16	Day 2	9.00	12.26	9.76
Day 1	21.00	12.26	9.16	Day 2	10.00	12.26	9.91
Day 1	22.00	12.26	9.31	Day 2	11.00	12.26	9.91
Day 1	23.00	12.26	9.31	Day 2	12.00	12.26	9.91
Day 1	24.00	12.26	9.31	Day 2	13.00	12.26	10.06
Day 2	1.00	12.26	9.46	Day 2	14.00	12.26	10.06
Day 2	2.00	12.26	9.46	Day 2	15.00	12.26	10.14
Day 2	3.00	12.26	9.46	Day 2	16.00	12.26	10.24
Day 2	4.00	12.26	9.61	Day 2	17.00	12.26	10.30
Day 2	5.00	12.26	9.61	Day 2	17.30	12.26	10.45

loads during the process of unloading have also been plotted and it is again found from this graph that the actual deflection is less than the theoretical values invariably at all loads.

The actual deflection values for various loads during the process of loading and unloading have been plotted as well.

After the loading reached the maximum of 940 kN, it was noticed that the deflection was 9.01 mm. After sustaining the load for 24 hours, the deflection got increased to 10.45 mm, which was attributable to time dependent deformation. It was found from this graph that the actual deflection even after releasing the entire load on the girder was 0.2 mm, though the theoretical value was zero. Generally, the actual value and the theoretical one at no load condition will not be equal as ideal girder cannot be expected in reality.

The actual maximum deflection of the girder under the maximum test load of 940 kN was observed as 10.45 mm as against the theoretical deflection of 12.26 mm. The residual deflection after complete removal of the load was observed to be 0.20 mm which is less than 1.57 mm, i.e., 15% of the maximum deflection recorded under the test load. Also, during the entire process of the test, the girder did not show any sign of distress. Hence, the road bridge girder can be considered to have successfully passed the load test.

The deflection readings were noted down for 24 hours at the sustained load and it is evident from Table 2 above that there is approximately a gradual increase of 0.15 mm for every two to three hours during the first eighteen hours and then it increases sharply every hour. The deflection increase in 24 hours during sustained loading is 1.29 mm (from 9.16 mm to 10.45 mm) which is 14.08% of the instantaneous deflection for the maximum load of 940 kN.

10. Conclusions

The maximum deflection calculated as per clause 6.6.2 of Indian Road Congress SP37-1991 is 11.55 mm. The actual maximum deflection observed even after sustaining the load for 24 hours is only 10.45 mm only. Since the actual values of deflection and the theoretical values of deflection are almost the same, it is concluded that the design is perfect and the behavior of the girder is as expected and designed. The composite girder cast is said to have passed the test satisfying the standards laid down in the above said Indian Road Congress. By having conducted such a test, it has been found / verified that the actual defections match almost with the theoretical values. Had the theoretical behavior and experimental behavior of the girder differed significantly, the design would have to be improved in such a way that the actual behavior would almost be similar to that of the theoretical behavior.

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