CHAPTER-3

EXPERIMENTAL DETAILS AND RESULTS

For the investigation of the effects of a composite deck on an open web steel girder bridge, an experimental investigation was performed. From the experimental study, following points are to be found:

1. It is also important to research how well composite construction can prevent compression members from buckling.
2. It is necessary to confirm the impact of shrinkage strain in initially enabling composite deck participation.
3. It is necessary to quantify the load distribution between the upper chord elements and the deck slab. To determine the behavior, the deck slab's stress variation is also important.
4. The impact of the composite deck on the structure's stiffness, deflection, and load bearing capacity is also investigated.

For the experimental study, two models were tested. Both models had similar frame configurations with top chord and bottom chord of 8mm size square solid members and web members of 25.4 mm size solid square members. The composite model had an RCC deck of thickness 20mm made composite with the frame using shear studs of diameter 3mm and height 18mm. Both the models were loaded gradually on a UTM until failure. Experiment details and observations for laboratory tests for buckling of steel open web steel girder bridges and effects of composite slab over steel open web steel girder bridge model are discussed in this chapter.

3.1 BUCKLING IN STEEL TRUSS BRIDGES

Buckling is the instability of structure that leads to its sudden failure. When a structure is subjected to compressive axial load, buckling may occur. Buckling is characterized by a sudden sideways deflection of a structural member. This occurs even when the stresses in the structure are well below the yield stresses of the material. In a simple buckling case, the failure modes can be found by mathematical solutions. For complex structures, the failure modes are found by numerical tools.

The buckling behaviour of columns was first investigated in 1757 by mathematician Leonhard Euler. He derived the Euler formula for the maximum axial load carried by a long slender ideal column. An ideal column is perfectly straight, made of a homogeneous material, and free from initial stress. When the applied load reaches the Euler load or critical buckling load, the column comes to be in a state of unstable equilibrium. At that load, the slightest lateral force will cause the column to fail by suddenly jumping to a new configuration, and the column is said to have buckled. Critical buckling load (P_{cr}) derived by Euler for long slender columns is given by:

$$P_{cr} = \frac{\pi^2 E I}{(KL)^2} \tag{3.1}$$

Where, E- modulus of elasticity,

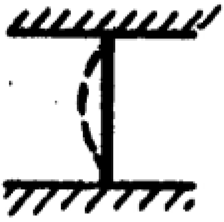
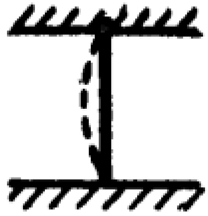
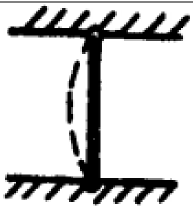
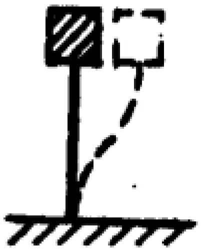
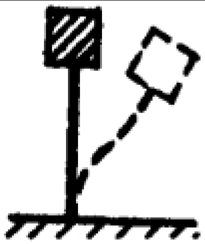
I - the smallest moment of inertia,

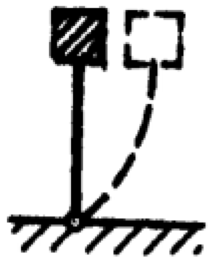
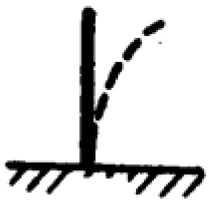
L - unsupported length of column,

K - column effective length factor, whose value depends on end conditions (Table 28, IS 456-2000 and Table 11, IS 800-2007)

The factor for effective length (K) is described in table 28, IS 456-2000 and table 11, IS 800-2007 as shown in Table 3.1.

Table 3.1 Effective length of compression members

Degree of end restraint of compression members	**Symbol**	**The theoretical value of effective length**	**Recommended value of effective length**
Effectively held in position and restrained against rotation in both ends		0.50 L	0.65 L
Effectively held in position at both ends, restrained against rotation at one end		0.70 L	0.80 L
Effectively held in position at both ends, but not restrained against rotation		1.0 L	1.0L
Effectively held in position and restrained against rotation at one end, and at the other restrained against rotation but not held in position		1.0 L	1.2 L
Effectively held in position and restrained against rotation at one end, and the other partially restrained against rotation but not held in position		-	1.5 L

Effectively held in position at one end but not restrained against rotation, and at the other end restrained against rotation but not held in position		2.0 L	2.0 L
Effectively held in position and restrained against rotation at one end but not held in a position not restrained against rotation at the other end.		2.0 L	2.0 L

Where L is the unsupported length of the compression member

For experimental investigation of buckling in bridges, a bridge model's dimensions were proposed based on an actual bridge. The dimensions of the model were similar to a 30.0m span steel truss bridge designed for two lanes of Class-A or 70R loading across a stream on the left bank of river Bhagirathi at Kotibhel, Uttrakhand (Figure 3.1). The bridge had a span of 30m and a depth of 7.5m.

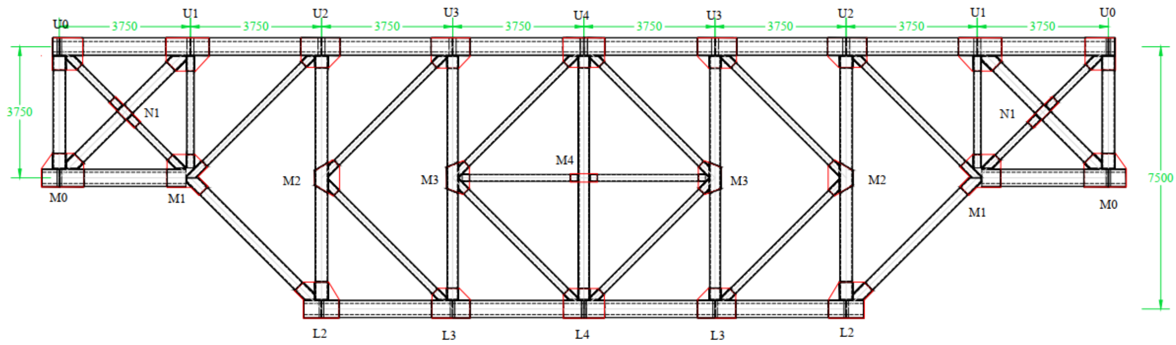


Figure 3.1: Kotibhel bridge [GAD, 30m steel truss, kotibhel bridge].

3.2 MATERIAL TESTS

The following materials used in the model fabrication were tested in the laboratory.

i. Tensile test of a square steel bar of size 8.0mm x 8.0 mm.

The tensile test of the steel bar in the UTM also included calibration of electrical strain gauges used for measuring strains in the bridge model.

ii. Compression test of steel shaped like a standard specimen as per IS 13780-1993

3.2.1 Tensile test of a square steel bar of size 8.0mm x 8.0 mm and calibration of electrical strain gauges.

Tensile loading is common in various structural components, especially in open web steel girder bridges. The ultimate tensile strength and ductility of the material used shall be tested in the laboratory for the safe design of the structure. The tensile test can be conducted on a UTM. As the sample is subjected to tensile load, internal resistance due to atomic bonding between atoms of the material is developed. The force per unit infinitesimal area is called stress.

The value of stress in the material increases to a material increases with increasing applied load to a maximum value. The limit up to which the material behaves elastically is called the yield point of the material. The stress at which the material fractures is called ultimate tensile stress. Percentage elongation in the bar is calculated as the ratio of increased length to the original length of the bar.

A tensile test of 8.0 mm x 8.0 mm steel bar in 20 t UTM was carried out to characterize the mechanical properties and calibration of 120 Ω and 350 Ω electrical strain gauges. Requirements for the test are as follows:

a) Tensile specimen correct dimensions
b) Strain gauges and data logger
c) Vernier calliper
d) Extensometer

The test data was gathered using two strain gauges of 120 Ω and 350 Ω. Extensometer was also used to measure the extension observed in the sample. The experimental setup including extensometer and strain gauges and the sample bar mounted on a 20 t universal testing machine is shown in Figure 3.2.

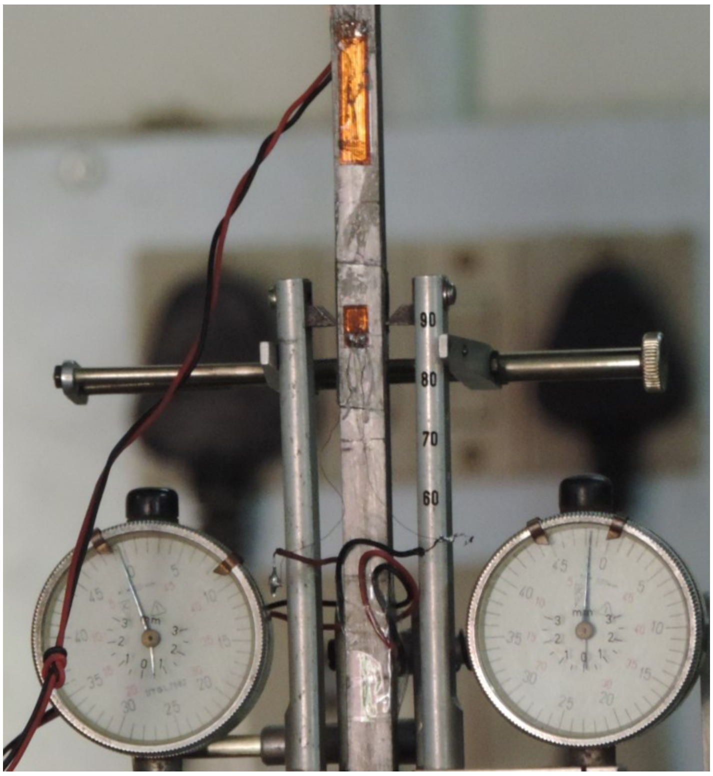


Figure 3.2: Mounting of extensometer and 120 Ω and 350 Ω strain gauges.

The tensile test for the specimen was carried out as per IS 1608-2005. The tensile test specimen was 70 cm in length. Gauge lengths of $5.65\sqrt{S_o}$, where S_o is the area of cross-section of the bar, were marked on it. Steps for the experimental procedure are as follows:

a) Measure the dimensions of the sample. With a pen mark then mark the gauge length reference points. The gauge length should be marked within the parallel section portion of the sample.

b) Measure original width and thickness of the sample at least four times along the reduced section (gauge length) of the specimen. Find the cross-sectional area and average area.

c) Let the testing machine be switched on and stabilized for at least 30 mins.

d) Fix the specimen into the testing machine with appropriate grips.

e) Check the data logger for readings.

f) Start applying the load.

g) As the sample gets fractured, note down the total extension from the chart. Immediately after fracture, there will be a large elastic recovery.

h) Measure final gauge length after fracture carefully.

The sample was loaded at a manually controlled loading rate of 1 ton per minute and the following applied loads and measured strains using an extensometer and the electrical strain gauges were recorded (Table 3.2).

Table 3.2: Applied loads and recorded strains

S.No.	Load (t)	Stress (N/mm^2)	Observed strain		
			120 Ω	350 Ω	Extensometer
1	0.0	0.0	0.0	0.0	0.0
2	0.5	78.1	0.0003	0.0003	0.0004
3	1.0	156.2	0.0008	0.0007	0.0007
4	1.5	234.4	0.0013	0.0012	0.0011
5	2.0	312.5	0.0018	0.0016	0.0015
6	2.5	390.6	0.0023	0.0021	0.0019
7	3.0	468.7	0.0029	0.0027	0.0023
8	3.5	546.9	0.0037	0.0034	0.0030
9	4.0	625.0	0.0050	0.0046	0.0040
10	4.25	664.0	0.0055	0.0047	-
11	4.3	671.9	0.0058	0.0050	-
12	4.4	687.5	0.0061	0.0051	-
13	4.5	703.1	0.0068	0.0055	-
14	4.6	718.7	0.0086	0.0067	-
15	4.7	734.4	0.0162	0.0070	-

Stress-strain curves from the two strain gauges and extensometer are given in Figure 3.3.

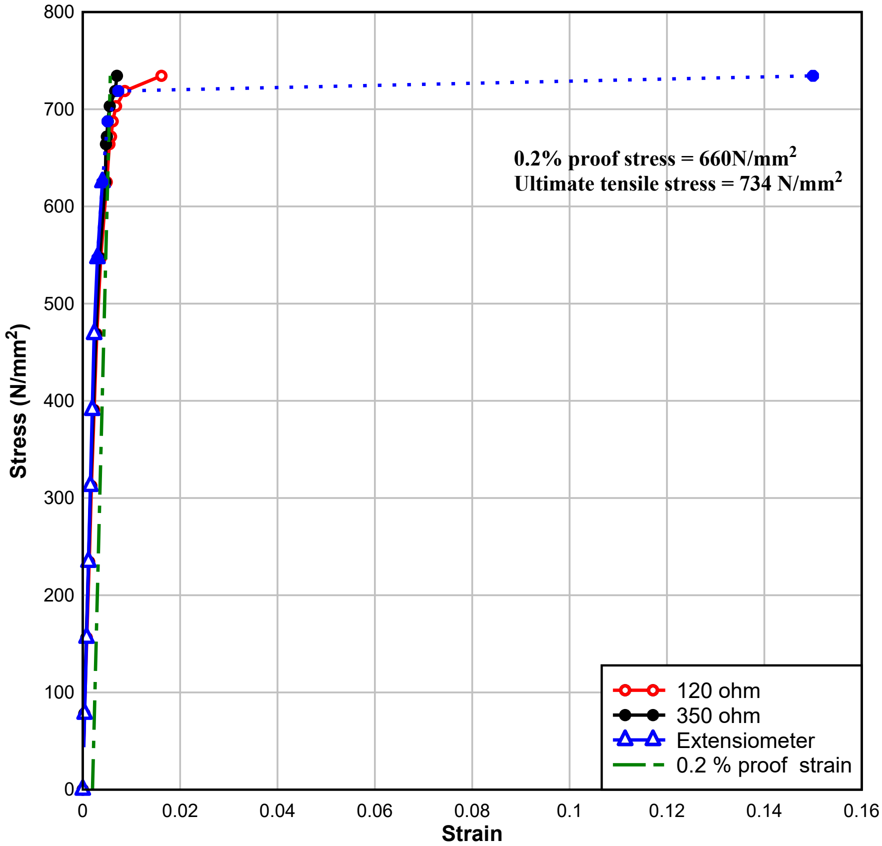


Figure 3.3: Stress-strain curves for 8 mm x 8 mm square bar

The maximum extension for a marked gauge length of 4.50 cm was recorded to be 0.68 cm.

Hence % elongation = $\frac{5.18-4.50}{4.50} \times 100 = 15.1\ \%$

Material properties obtained from the above test are given in table 3.3.

Table 3.3: Material Properties of 8 mm square bar

S. No.	Properties	Values
1	Ultimate tensile stress	734.0 N/mm^2
2	0.2% proof stress	660 N/mm^2
3	Maximum elongation	15.1%
4	Modulus of elasticity (extensometer)	200492 N/mm^2

Correction factor for 120 Ω and 350 Ω strain gauges

Strain recorded by the extensometer is the standard strain, and the strain gauges are calibrated against the extensometer strain. The correction factors are found as the corresponding ratios of the slopes of the curves for strain gauge readings and the extensometer readings.

Correction factor for 120 Ω strain gauge = 1.15

Correction factor for 350 Ω strain gauge = 1.06

3.2.2 Compression Test

The sample for the compression test was prepared in a workshop using a lathe machine. The sample was scaled 1.75 times the size specified by IS 13780- 1993 (Figure 3.4).

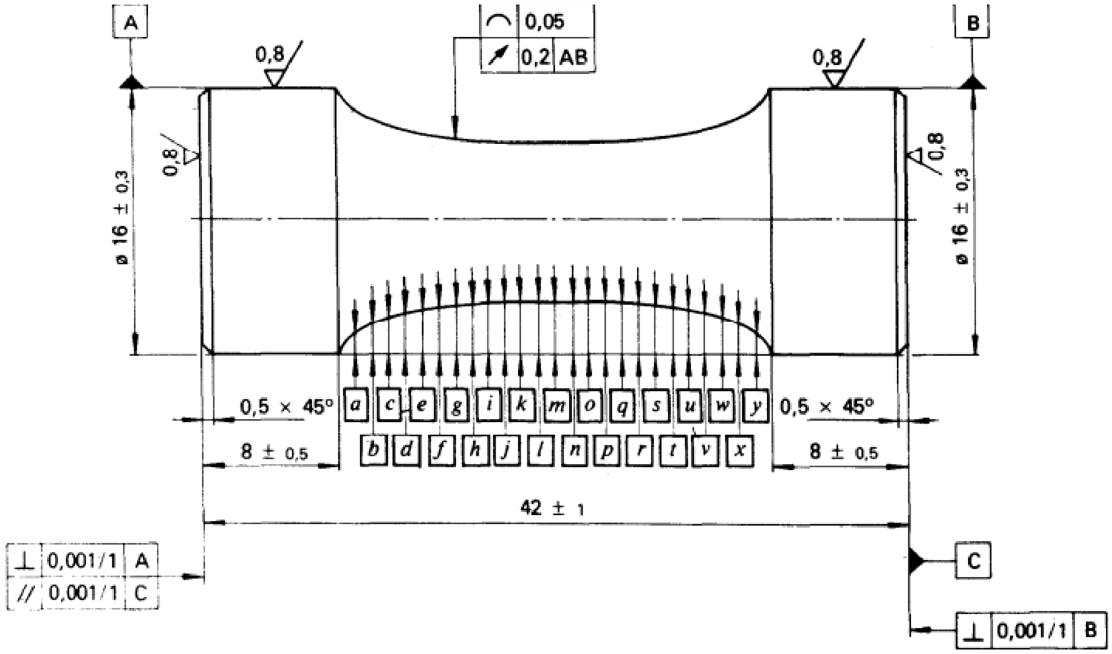
The compression test was carried out on 100 t UTM. A 350 Ω electrical strain gauge was mounted on the sample (Figure 3.5) to record the strain.

During the test, the sample kept on taking compression and the test was stopped at 25t of loading. The strain was recorded using a strain gauge up to 9t, after that the strain gauge dislodged from its original position due to excessive strain.

Stress-strain values recorded from the strain gauge during the experiment are given in table 3.4. Appropriate correction factor was also used on the recorded strains.

Table 3.4: Strain from the compression test

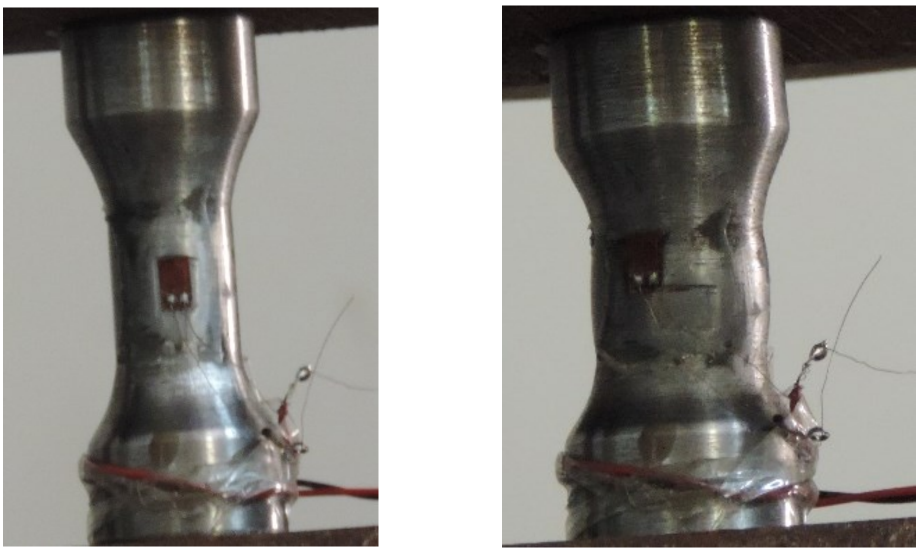
S.No.	Load (t)	Compressive Stress (N/mm^2)	Compressive Strain
1	0.0	0	0
2	1.0	41.6	0.0002
3	2.0	83.2	0.00042
4	3.0	124.8	0.00061
5	4.0	166.4	0.00085
6	5.0	208.0	0.00105
7	6.0	249.6	0.00129
8	7.0	291.2	0.0015
9	8.0	332.8	0.00172
10	9.0	374.4	0.00192
11	25.0	1039.9	0.03012



a	*b*	*c*	*d*	*e*	*f*	*g*	*h*	*i*	*j*	*k*	*l*
1,21	1,90	2,29	2,54	2,69	2,79	2,86	2,91	2,94	2,96	2,98	2,99

m	*n*	*o*	*p*	*q*	*r*	*s*	*t*	*u*	*v*	*w*	*x*	*y*
3,00	2,99	2,98	2,96	2,94	2,91	2,86	2,79	2,69	2,54	2,29	1,90	1,21

Figure 3.4: Dimension of sample for compression test. [IS 13780- 1993]



(a) **(b)**

Figure 3.5 Test sample, (a) Initial (b) Failed

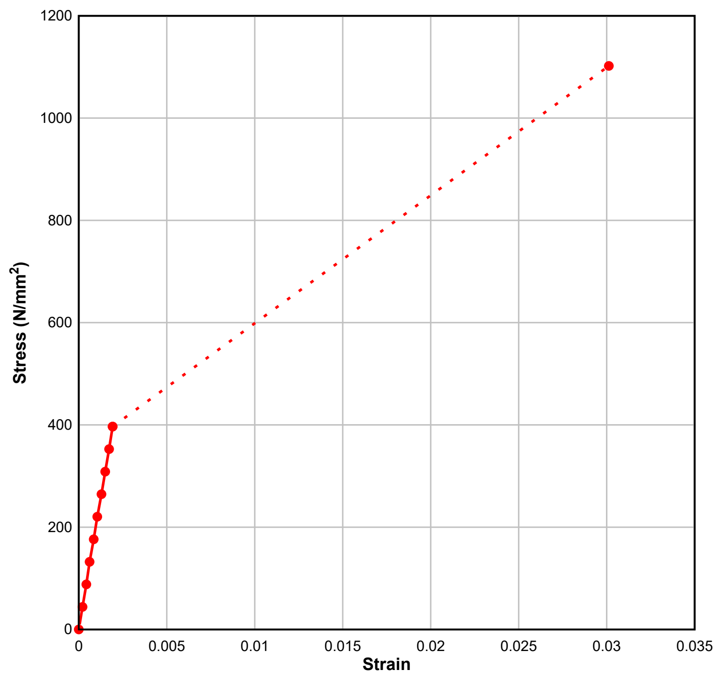


Figure 3.6: Stress-strain curve for the compression test.

Modulus of Elasticity = 2.04 x 10^5 N/mm^2

The Combined (tension-compression) stress-strain curve for the steel used is shown in Figure 3.7.

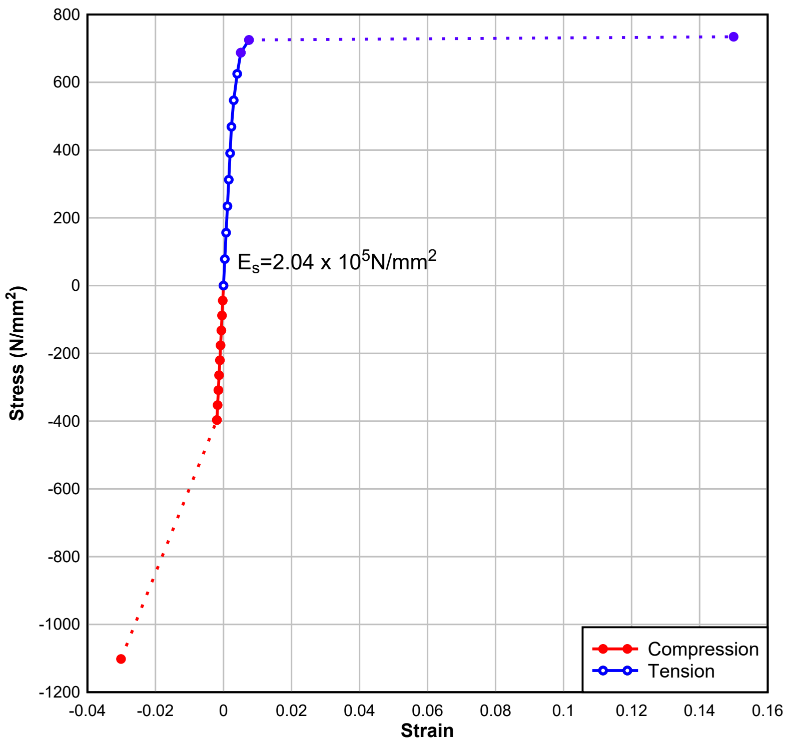


Figure 3.7: Stress-strain curves for tension and compression.

3.3 MODEL DESIGN AND FABRICATION

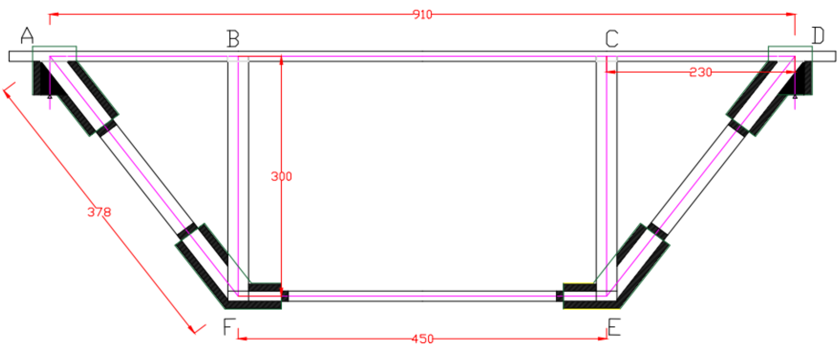
The test on the composite and non-composite model was done on 100 t UTM. The following experimental constraints have been considered.

(i) The model was to be tested on UTM with the length of the mounting arm = 1.2m.

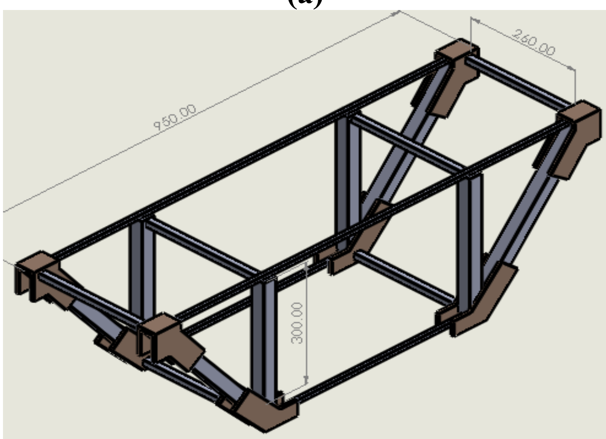
(ii) The failure mode was to be by buckling of the middle top chord member.

(iii) All other members except the buckling member have to be sufficiently strong to eliminate any other mode of failure.

The total length of the model is taken as 910 mm and the depth of the truss is 300 mm. The model has two two-dimensional frames connected at a spacing of 260 mm from centre to centre. The model dimensions are shown in Figure 3.8.



(a)



(b)

Figure 3.8: Model details (a) Dimensions, (b) 3-D View

The model with an 8.0 mm x 8.0 mm cross-section of top chord and bottom chord was prepared. The 2-two dimensional frames were connected using 12.7mm size solid square members. The model was loaded at four points. For a two dimensional truss, the loading was applied at points B and C (Figure 3.8 a).

The bridge model was prepared using welded joints. For analysis, point loads of magnitude P are assumed to act at points B and C (Figure 3.9).

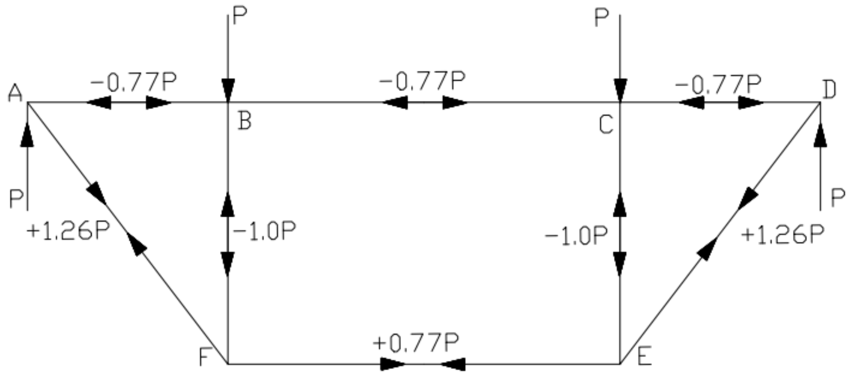


Figure 3.9 Member forces under joint loads P.

Member forces in the truss under loads P at each of the 2 joints are calculated and shown in Figure 3.9.

3.3.1 Design of joints

For joint E/F

Welded joints are designed for yield and fracture conditions. All the members will be joined using a double V butt weld. Apart from it, a plate of 6 mm thickness will be welded with the joints with high stresses.

Leg length (S) =3 mm

Throat thickness (T) = K x S {for 90 degree weld K=.707}

T=2.12 mm

As per IS 800-2007, allowable stress in weld =108N/mm^2

Hence, Load= 108 x 6 x effective length

Load = 648 x effective length

Yield stress of 8 mm member = 660N/mm^2.

Failure load = 660 x 8 x 8 = 38.4 kN

For yield of member FE, P=49.87 kN

Force in member FB =1.0 x 49.87 =49.87kN.

Force in member DE=1.26 x 49.87 =62.84kN.

For member FE

The effective length for member FE with a load of 38.4 kN =$\frac{38400}{108*1.12}$ =167.7 mm

Total length of the weld provided = effective length + 2 x S = 173.7 mm

Hence, provide 180 mm of the total length of the weld.

Weld on the bar across = 25.4mm

Plate length= 180-25.4= 154.6mm

For the member FE, filler material on both sides of the member will be used for weld length. The shape and dimensions of the welded plate are shown in Figure 3.10.

For member DE

The effective length for member DE with a load of 62.84 kN =$\frac{62840}{108*1.12}$ =274.46 mm

Total length of the weld provided = effective length + 2 x S = 280.46 mm

Provide 284mm weld length. The shape and dimensions of the welded plate are shown in Figure 3.10.

For joint D/A:

For member DE

A plate of 6 mm thickness is welded on joint D as shown in Figure 3.10.

Length of the weld required for member DE = 284mm

Provide a plate of a given dimension as in Figure 3.10.

CD is under compression, having a force as same as that of member FE. Hence provide a weld length of a minimum of 4 cm. Dimensions of plates provided and the joints are shown in Figure 3.10.

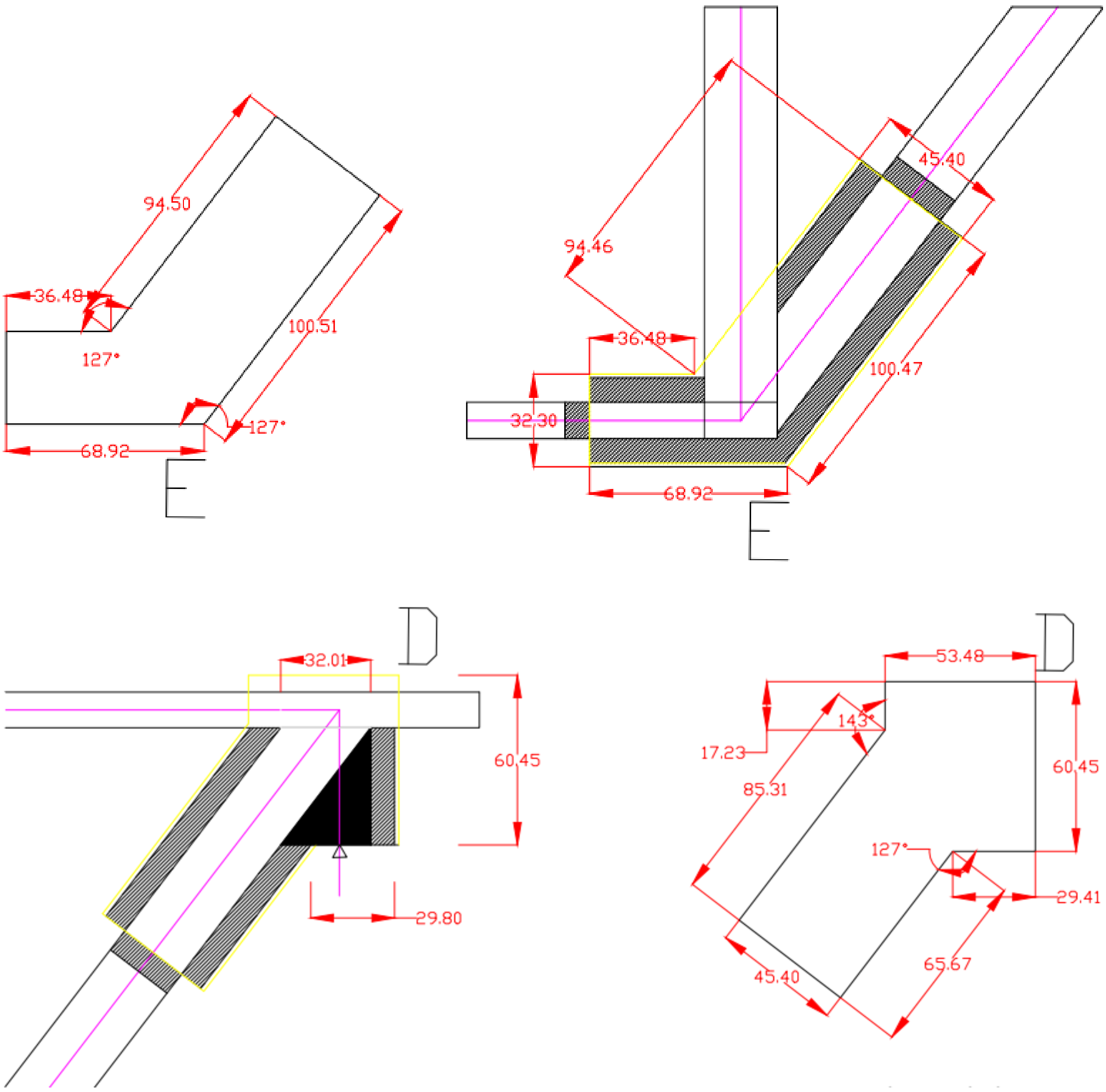


Figure 3.10: Plates and weld dimensions for joints E and D

3.3.2 Design of supports

After making the RCC deck slab composite with the top chord members, the load-carrying capacity of the structure is expected to increase. The strength of the support was kept high so that it withstands the experimental load and show less compressive strain under load. Hence, the supports are designed for a total load of 60 tons of load.

The load carried by one column = 30 tons

Height of the column = 480 mm

For one end fixed and one end restrained condition, effective height = 1.2 x 480 mm =576 mm.

Assuming design compressive stress f_{cd} = 120 N/mm^2

Hence, Area required, 'A' = 300000/120 = 2500

The support is constructed using four sections of ISA 65x65x6 mm combined. The angle sections were welded together throughout the length and a batten plate of 6mm thickness along with one 12mm diameter circular member was welded as lacing.

Area of each angle section = 744 mm^2

Total cross-sectional area of the support= 4 x 744 mm^2 = 2976 mm^2 > 2500 mm^2

Area = 2976 mm^2.

Radius of gyration = 50.68 mm

Kl/r = 11.36

For the above Kl/r ratio, as per IS 800-2007 table 9(c), buckling stress ≈ 226 N/mm^2.

The actual load-carrying capacity of one support = 67.3 tons.

As the loads were to be transferred through the centerline of the supports, and ISCM 100 member was welded on the top of the support as shown in Figure 3.11. An extra plate member of 10mm thickness was also welded along with the web of the ISMC 100. The arrangement of the model and support is shown in Figure 3.11. A groove on the base of the support was also prepared so that the arrangement can be mounted on the UTM.

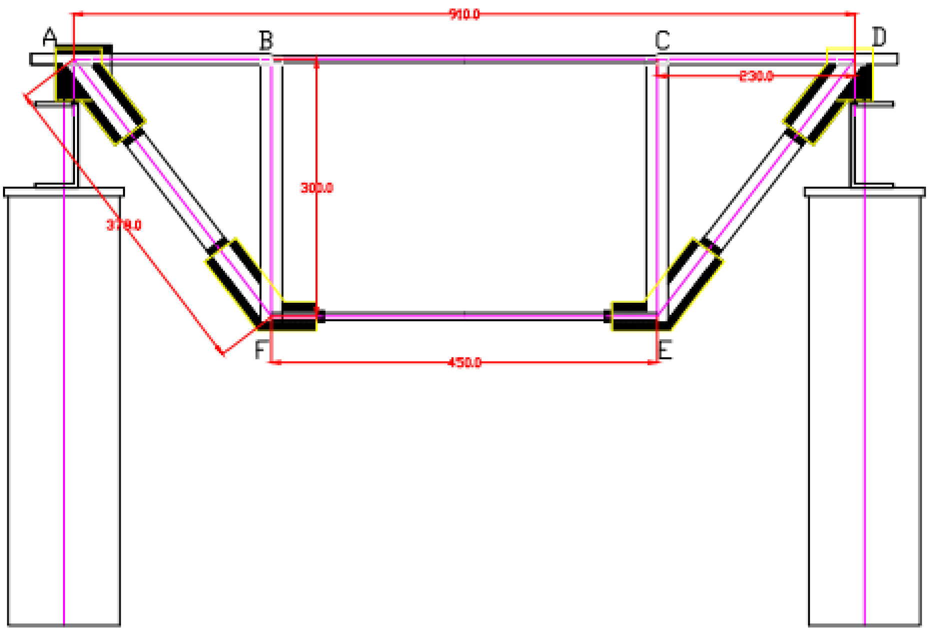
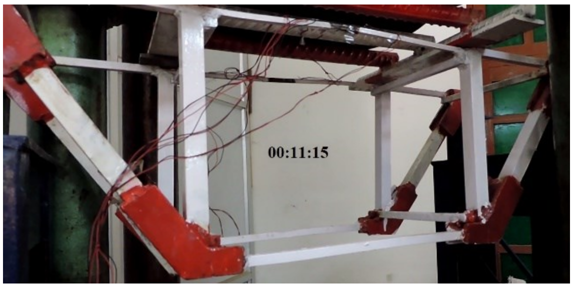


Figure 3.11 Arrangement of the model above the supports

3.4 LOAD TEST FOR NON-COMPOSITE MODEL

In the non-composite model, the top chord member and bottom chord members were square members with 8 mm size, the model is shown in Figure 3.12.

The objective of the experiment was to ensure a buckling failure of the model. Member BC which is 450mm long centre to centre is assumed to fail first. The member was welded at points B and C hence member BC was assumed fixed for the analysis. Four strain gauges were used to measure the strain in the top chord members as shown in Figure 3.12. Dial gauges were used on the bottom chord members to measure the vertical deflection during the experiment. The model was loaded gradually using UTM. A loading frame was used to transmit the loads at the desired locations. Figure 3.12 shows the models before failure and after the failure of the top chord member. The mode of failure was due to the buckling of top chord members.



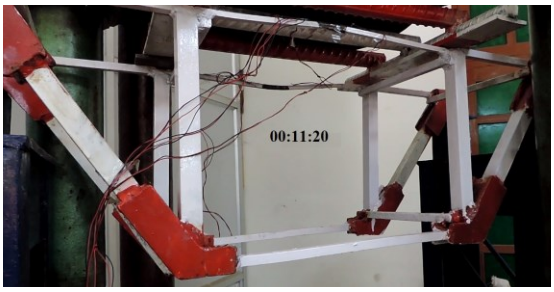


Figure 3.12: (a) Model just before failure at 00:11:15 (b) Model after sudden top chord buckling failure at 00:11:20

Four 350-ohm strain gauges (two on each member) were used to record the strain during the experiment. The strains recorded during the experiment are shown in table 3.5 along with the corresponding loads and stresses in the members. The graph for stress-strain for all the four strain gauges are also plotted in Figure 3.13

Table 3.5: Strain measured using Strain gauges

S.No.	Load UTM	Stress (N/mm2)	Measured Strain			
			Gauge 1	Gauge 2	Gauge 3	Gauge 4
1	0	0	0	0	0	0
2	0.4	12.0	7.12E-05	5.78E-05	6.93E-05	5.20E-05
3	0.8	24.1	0.000142	0.000116	0.000139	0.000104
4	1.2	36.1	0.000214	0.000173	0.000208	0.000156
5	1.6	48.1	0.000285	0.000231	0.000277	0.000208
6	2	60.2	0.000388	0.000303	0.000347	0.000283
7	2.4	72.2	0.00047	0.000351	0.000434	0.000347
8	2.8	84.2	0.000541	0.000383	0.000484	0.000392
9	3.2	96.2	0.00057	0.000462	0.000554	0.000416
10	3.6	108.3	0.000601	0.00049	0.000601	0.000485
11	4	120.3	0.000633	0.000527	0.000666	0.000546
12	4.4	132.3	0.000678	0.000564	0.000711	0.000568
13	4.8	144.4	0.000734	0.000582	0.00073	0.000615

14	5.2	156.4	0.000776	0.000619	0.000804	0.000693
15	5.6	168.4	0.000852	0.000642	0.000833	0.000723
16	6	180.4	0.000879	0.000693	0.000887	0.000767
17	6.4	192.5	0.000914	0.000711	0.000961	0.000878
18	6.8	204.5	0.000992	0.000748	0.001007	0.000975
19	7.2	216.6	0.001069	0.000776	0.001053	0.001099
20	7.6	228.6	0.001156	0.000785	0.001243	0.001247

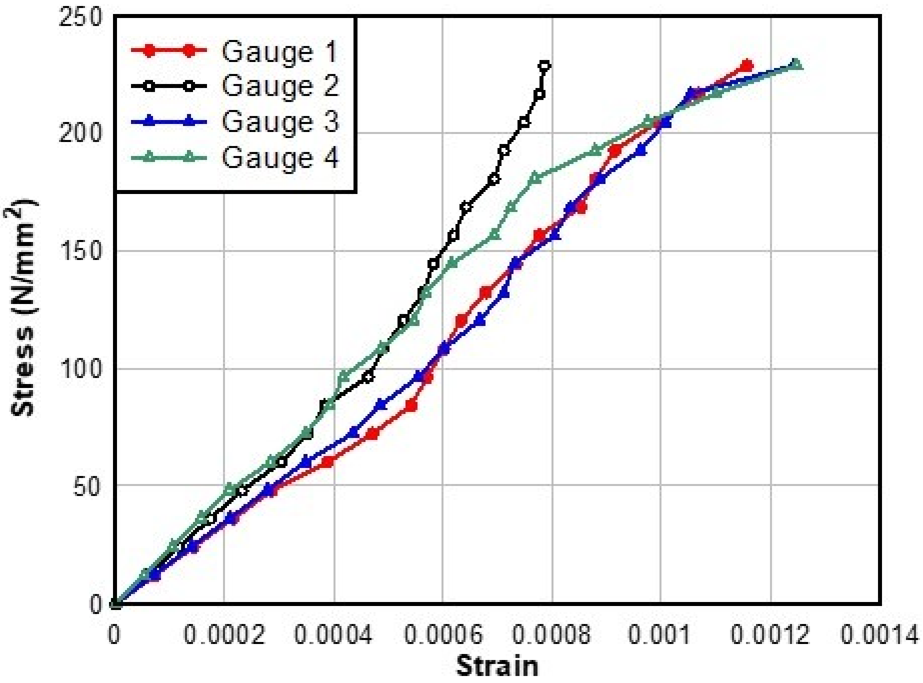


Figure 3.13: Strain recorded from strain gauges

The average of the strains recorded in the compression members was found and was compared to the linear stress-strain curve for the material's young's modulus of elasticity = 200492 N/mm^2. The graph showing the average compressive strain in the member is plotted in Figure 3.14. From the Figure, it can be observed the materials behave linearly in the beginning and slight variation was observed as the stress increased.

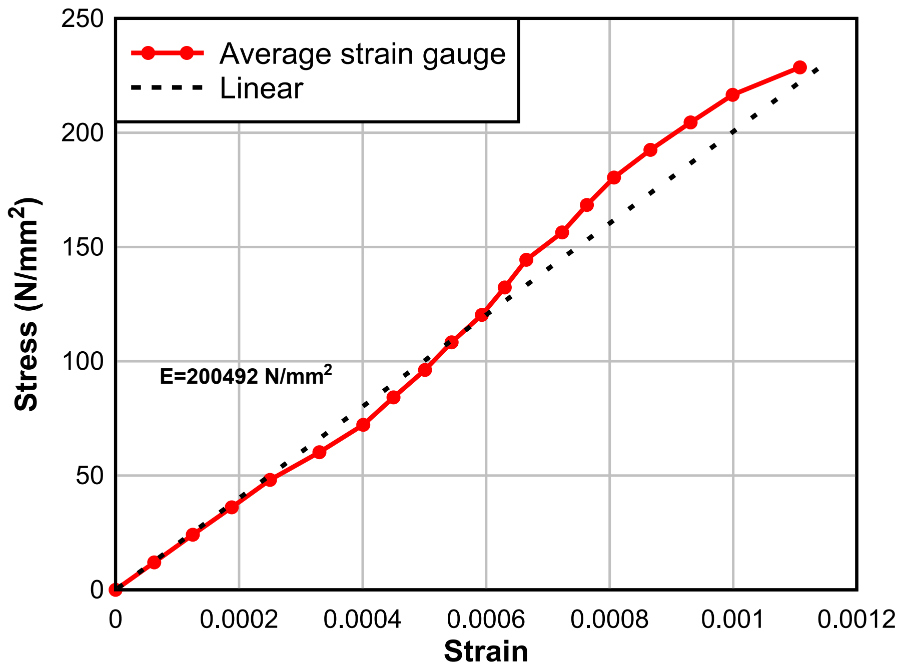


Figure 3.14: Stress- average recorded strain variation (8 mm x 8 mm model)

The non-composite model buckled at a load of 7.8 tons, which corresponds, to 76.5 kN. As the member BC was welded at both ends, it was analyzed assuming fixed end conditions.

As buckling load is given by Euler buckling load $P_{cr} = \frac{\pi^2 E I}{(KL)^2}$

Where E (Young's Modulus) = 200492 N/mm^2.

I (Moment of Inertia) = $(8)^4/12$ = 341.33 mm^4.

KL (effective length) = 0.5 x 450 = 225 mm.

P_{cr}= 13574.98 N = 13.6 kN. Theoretical stress at buckling = 212.12 N/mm^2

The failure load of the model as per Euler buckling load = 70.65 kN.

During the experiment, the bridge model failed at a load of 76.5 kN.

Force in member observed experimentally = 0.77 x (76.5/4) = 14.72 kN

Stress in member = 230.00 N/mm^2.

Stress in members at buckling during the experiment was observed to be slightly higher than theoretical stress.

The same model was also analyzed in STAAD Pro. The STAAD editor file is attached as annexure A.

Vertical deflection of the model during the test was measured using dial gauges attached at the midpoint of bottom chord members. The load-deflection data are shown in table 3.6 below:

Table 3.6: Vertical deflection of the non-composite model

S.No.	Load (kN)	Dial gauge-1	Dial gauge-2
1	0.0	0	0
2	4.0	26	14
3	8.0	52	30
4	12.0	80	46
5	16.0	103	60
6	20.0	116	82
7	24.0	130	98
8	28.0	151	114
9	32.0	171	127
10	36.0	184	150
11	40.0	198	176
12	44.0	211	189

The plots of the vertical deflection as recorded by dial gauges are shown in Figure 3.15. The two dial gauges here show slight variation in results. This may be due to slight error in the manual welding in (i) the members of the model, (ii) slight error in the welding in the loading frame (iii) Slight error in the support height may also contribute in the variation. The average recorded deflection from dial gauges is then compared to the STAAD.Pro model deflection and is shown in Figure 3.16. From Figure 3.16 it can be seen that average vertical deflection from experiment and vertical deflection from STAAD analysis match.

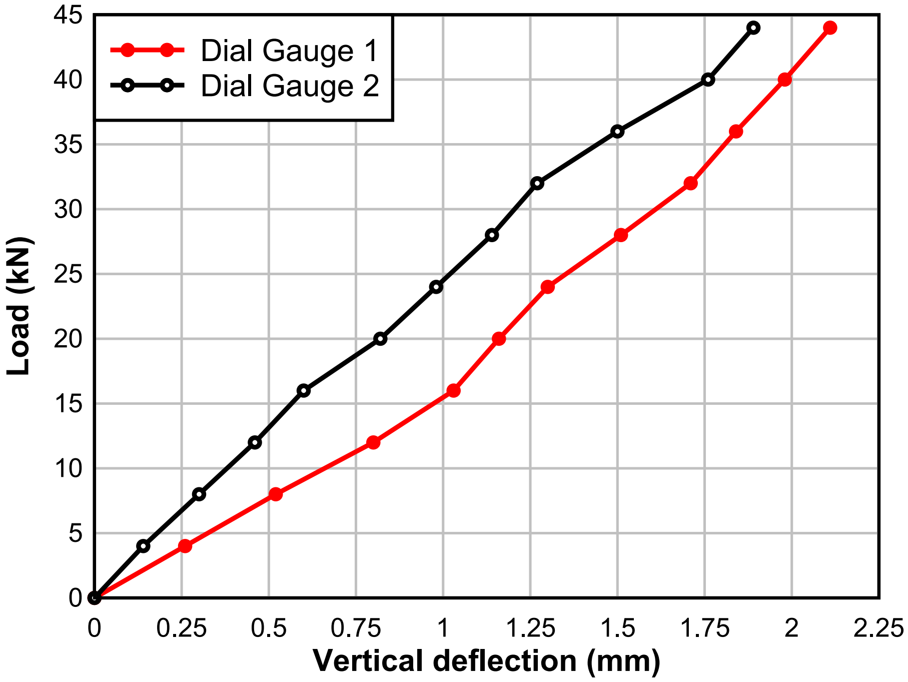


Figure 3.15: Load deflection curves from the two dial gauges attached to bottom chord members of the non-composite model.

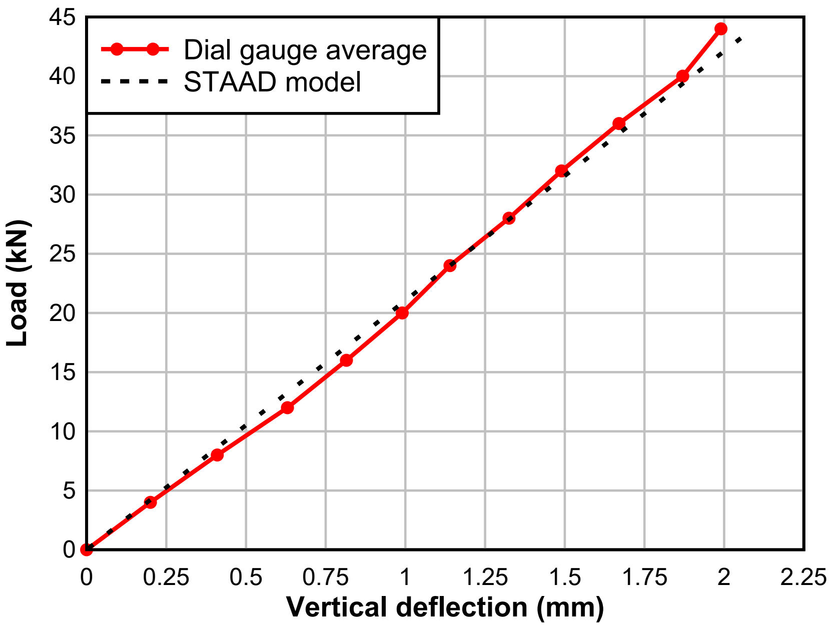


Figure 3.16: Average of experimentally observed and numerically observed vertical deflection for the non-composite model.

3.5 LOAD TEST FOR COMPOSITE MODEL

For the composite model, An RCC deck was cast over the frame used in the previous experiment. The RCC deck and steel members were attached using shear connectors, which were designed as per IRC 22-2015. The composite bridge model comprises 8 mm x 8 mm top chord and bottom chord members. The depth of the deck slab was kept to be 20 mm. The deck slab was reinforced using 3mm diameter bars. The height of shear connectors was kept as 18 mm.

3.5.1 Model preparation

For preventing buckling of the top chord member, a concrete deck was made composite with the top chord member. The composite model was tested to analyze:

(i) Effect of shrinkage of concrete on composite action.

(ii) Change of load taking behaviour due to composite action.

(iii) Performance of shear connectors until failure.

(iv) Stresses developing in the deck slab.

(v) The extent of load sharing by RCC deck during the experiment.

The casted deck slab had the dimensions (l x b x h) of (910 mm x 360 mm x 20 mm). The extension of 50 mm of deck slab width beyond the steel member was considered in accordance with a 42m span steel truss girder bridge on Devidhar-Fold to Bhatwadi-Panchangaon Road at Uttrakhand (Figure 3.17). The bridge was designed in 2014 and is operational now. The bridge has a deck slab of 220mm thickness. The RCC slab has 16mm diameter longitudinal reinforcement at a spacing of 150mm c/c and 12mm diameter lateral reinforcement at a spacing of 200mm c/c.

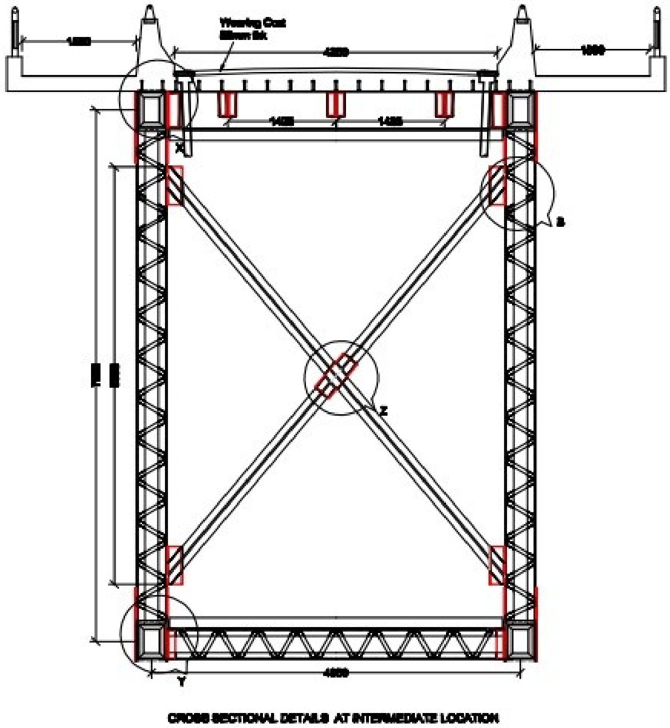


Figure 3.17: Deck slab on 42.0 m span open web steel girder bridge on Devidhar-Fold to Bhatwadi-Panchangaon Road at Uttrakhand

In the composite bridge model, a 20 mm thick RCC deck slab was laid on the model frame and was connected with the top chord members using T shaped shear connectors. For reinforcement, two layers of 3.0 mm mild steel wire were used. The spacing of reinforcement wire across the deck is 6.0 cm and along the deck is 10.0 cm as shown in Figure 3.18.

3.5.2 Design of shear stud

The calculation for the spacing of the shear studs was done as per clause no 606 of IRC 22-2015. The calculation was done for 2 mm and 3 mm diameter of shear studs. The shear studs were of T shape with 18mm as the height of studs and 15mm as the width of the T shaped studs. The calculation for shear studs was done for fatigue and strength considerations.

Height of truss = 300 mm

Depth of N.A.:

F_{ck} x a x b_f = A_s x f_u

For deck slab of 20mm thickness and made of M-30 grade concrete

30 x a x 300/2 = 8 x 8 x 650

a = 9.24mm.

Table No. 3.7: Shear stud analysis

Shear stud diameter	2mm	3mm
Height of shear stud	1.8 cm	1.8 cm
Height to diameter ratio	9	6
The cross-section area of each stud	3.14 mm^2	7.07 mm^2

Number of rows of shear connectors provided = 1

Design as per fatigue characteristics (IRC 22-2015)

Fatigue constant for 2 x 10^6 cycles = 55 N/mm^2.

Shear strength of each 2 mm diameter stud = 55 x 3.14 = 172.7 N

Shear strength of each 3 mm diameter stud 55 x 7.07 = 388.85 N

F_{AB} = 15.4 kN (for design load of 7.8 t)

Number of shear studs per row = 15.4 x 1000 /170 = 91 studs (for 2 mm stud dia.) @ 1 cm c/c

Number of shear studs per row = 15.4 x 1000 /380 = 41 studs (for 3 mm stud dia.) @ 2.2 cm c/c

Design as per strength characteristics (IRC 22-2015)

$$\boldsymbol{V_L} = \sum[\frac{\boldsymbol{V.A_{ec}.Y}}{\boldsymbol{I}}]_{\boldsymbol{dl,ll}} \quad (3.2)$$

Where,

V_L = Longitudinal shear per unit length.

V = The vertical shear forces due to dead load and live load (including impact) separately at each state of load history.

A_{ec}= The transformed compressive area of concrete above the neutral axis of the composite section with the appropriate modular ratio depending on the nature of load (whether short term i.e. live load or long term i.e. dead load)

Y= C.G. distance of transformed concrete area from the neutral axis.

I = Moment of inertial of the whole composite section using appropriate modular ratio.

dl, ll = different load history, i.e. sustained load or composite action dead load, transit load or composite action live load. Those loads are to be considered with an appropriate load factor at this stage.

Modular ratio (m) = 211805.7/ (5000 x √30) = 7.73

Transformed area of concrete = 150 x 20 / 7.73 = 388.09 mm^2.

Depth of neutral axis= 9.24 mm

Moment of inertia of transformed section =

$=19.4 \times 9.24^3 /3 + 8^4/12 +64 \times 14.76^2 + 8^4/12 + 64\ 314.76^2$

= 6360453.922 mm^2.

Tensile strength of member = 41.6 kN

Using all the values in the above formula we get:

V= 41.6 x 1000 x 388.09 x 4.62/ 6360453.92 = 11.72 N/mm.

Shear strength of one stud assuming strength to be 150 N/mm^2.

Strength of 2 mm stud = 417 N

Spacing of stud = 417 / 11.72 = 40.19 mm

Number of stud = 910/ 40.19 = 23 studs

Strength of 3 mm stud = 1060.5 N

Spacing of stud = 1060.5 / 11.72 = 90.49 mm

Number of stud = 910/ 90.49 = 11 studs

Based on the above design procedures following are the spacing and number of studs required:

Table 3.8: Spacing and number of studs

Method	Number of studs		Spacing (mm)	
	2 mm	3 mm	2 mm	3 mm
IRC 22-2015 (Fatigue considerations)	91	41	10	22.2
IRC 22-2015 (Strength considerations)	23	11	39.56	82.72

Choosing 3 mm shear studs arrangement as per IRC 22-2015 as per fatigue consideration.

Shear stud dimensions:

Height: 18 mm

Dia. of stud: 3 mm

Flange width: 15 mm

Shear studs were welded on the frames and two such frames are welded together at a spacing of 26.0 cm centre to centre using a 12.7mm size square member.

3.5.3 Fabrication of composite model.

The steel frame used has an 8 mm square member as its top chord and bottom chord. Figure 3.18 shows the composite model with reinforcement tied before casting the RCC slab. T-shaped shear connectors were welded on each of the frames in one row at a spacing of 22mm centre to centre. The spacing is taken as minimum recommended by the two above considerations discussed in the code.

Figure 3.18: Shear studs and deck reinforcement.

Nominal concrete used in casting the deck slab was in ratio concrete: sand: coarse aggregate: 1: 1.5: 3, ratio mix. The water-cement ratio used was 0.4. As per IS 456-2000, the maximum size of aggregate used shall not be greater than one-fourth of the minimum dimension of the member. Hence, 5 mm aggregates were used for the construction of the deck slab. SIKA viscocrete superplasticizer: 1% of the weight of cement was also used while casting the deck. The model was inverted to mould the deck slab as shown in Figure 3.19. After hardening the deck slab, curing of the deck slab was done for 28 days.

While casting the deck, eight samples cubes of 100 mm side were also made for testing. Three cubes were tested after 7 days of curing. Rest five were tested on the day of the test. The strength of the cubes is shown in table 3.9.

After casting the deck slab, concrete: sand: 1:1 ratio slurry was used for plastering the top surface of the deck slab (Figure 3.20) to obtain a smooth for pasting the strain gauges. The thickness of the plaster was kept as minimum as possible.

Figure 3.19 Inverted model in the mold for deck slab casting

Figure 3.20 Deck slab before and after plastering.

Average failure stress after 7 days of casting was observed to be 27.03 N/mm^2 and on the day of the test (60 days after casting), it was 58.16 N/mm^2

Table 3.9: Deck concrete cube test results

S.No.	Cube number	Failure load (kN)	Stress (N/mm2)
Cube testing results after 7 days of casting			
1	1	254	25.4
2	2	284	28.4
3	3	273	27.3
Cube testing results on the day of testing after 60 days of casting			
4	1	607	60.7
5	2	598	59.8
6	3	533	53.3
7	4	565	56.5
8	5	605	60.5

Figure 3.21 shows the finished model before pasting the strain gauges.

Figure 3.21 Finished model

3.5.4 Experimental setup

The load test was conducted on 100 ton UTM as shown in Figure 3.22. The strain was recorded using 13 strain gauges

- 8 no. 120-ohm strain gauges on the members. Two strain gauges on two compression and two tension members
- 5 no. 350-ohm strain gauges on deck slab as shown in Figure 3.23.

Two strain gauges were used in each member of the top chord and bottom chord. Before pasting the strain gauges, the paint on the member was removed and the surface was thoroughly cleaned. The classification of strain gauges is shown in table 3.10.

Strain gauges in the deck slab were pasted after scrapping the plaster and clearing the surface thoroughly. On the deck slab, two strain gauges were pasted above the top chord member locations and in between them, 3 strain gauges were used to measure the variation of strain. The strain gauges on the deck slab were at a spacing of 65 mm centre to centre. The data from the strain gauges were recorded using a well-calibrated data logger as shown in Figure 3.22.

Figure 3.22 Setup for the composite model test, showing UTM, data logger and mounted strain gauges.

The arrangement of strain gauges mounted on the steel members and the deck slab concrete is shown in Figure 3.23. While applying strain gauges on the concrete deck of the model, the thin plaster layer was scrapped off using sandpaper.

Table 3.10: Classification of strain gauges mounted on members

Member	Strain gauge
Frame I top member	CH-1 and CH-5
Frame II top member	CH-3 and CH-7
Frame I bottom member	CH-2 and CH-6
Frame II bottom member	CH-4 and CH-8

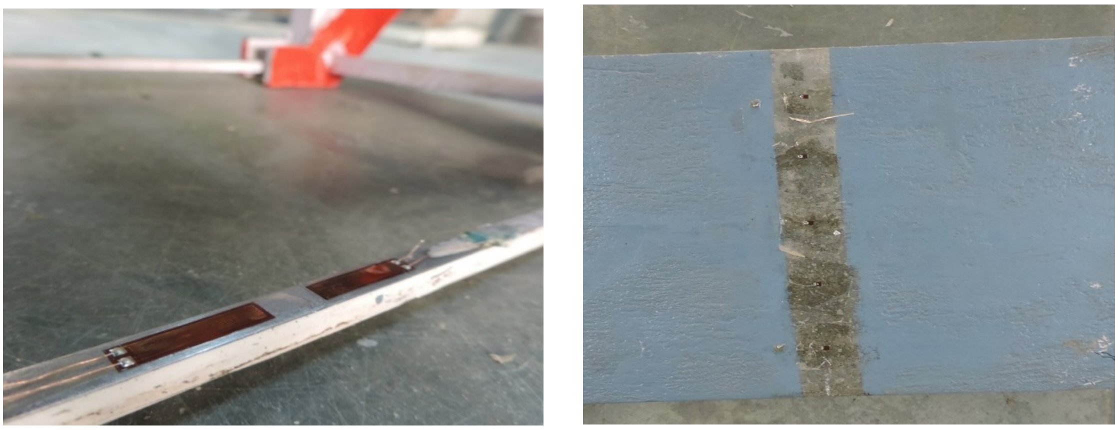
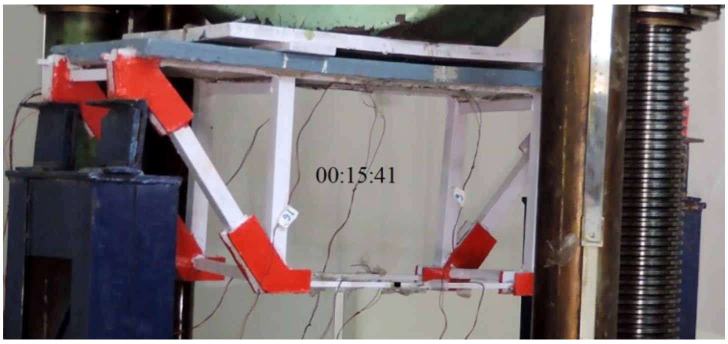


Figure 3.23: (a) 2 number 120-ohm strain gauges mounted on each steel member at top and bottom chords. (b) 5 number 350-ohm strain gauges at 6.5 cm c/c mounted of the deck slab

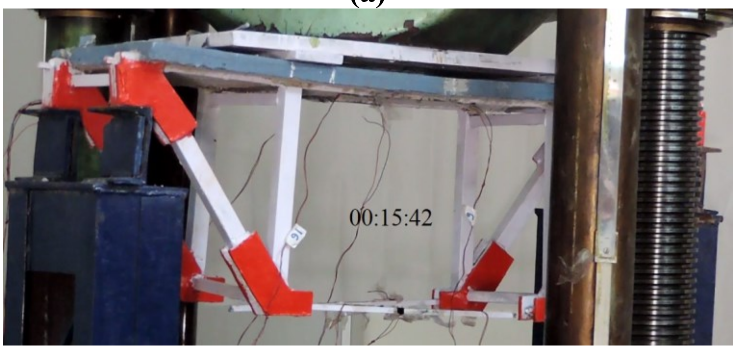
3.5.5 Composite model test results

The composite model was tested as shown in Figure 3.22. The load from UTM was gradually increased up to Failure. Figure 3.24 shows the model before and after the failure. The composite model failed at a load of 196.2 kN. Failure mode was a tensile failure of bottom members (Figure 3.25). RCC deck also experienced minor cracks near the support and along with the top members (Figure 3.26). On the deck slab, minor cracks were observed near supports and along with the members. With increasing load, the deck slab kept on taking the load and reported strains accordingly, it can be said that the shear connectors performed satisfactorily up to failure and the composite action was observed from the beginning till the end.

The ultimate load at failure for the non-composite bridge model is 76.5 kN and for the composite bridge model, it is 196.2 kN. Therefore, the load-carrying capacity for the composite bridge is 2.56 times that of the non-composite bridge model. The mode of failure for the non-composite bridge model is sudden due to buckling of the top chord members at a stress of 234.6N/mm^2. The failure mode for the composite bridge model is entirely changed to rupture of its bottom chord members at higher stress of 614.8N/mm^2. Thus, composite bridge design may permit higher reserve strength in comparison to the non-composite bridge.



(a)



(b)

Figure 3.24: Model (a) before failure (00:15:41) (b) after failure due to rupture of the bottom chords (00:15:42)

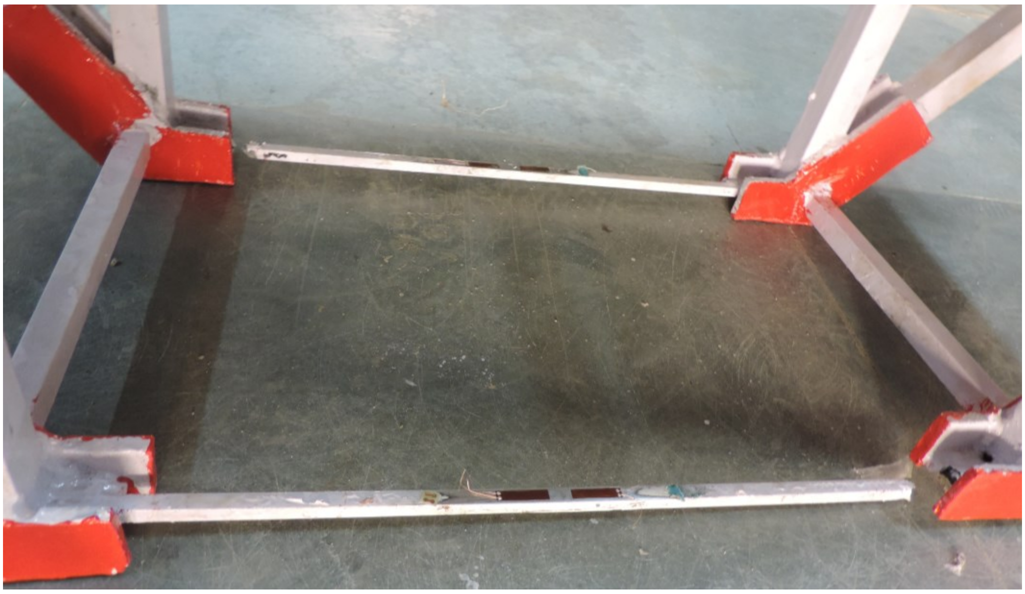


Figure 3.25: Bottom chord rupture at failure

Figure 3.26: Deck slab top view after failure

The strains recorded from eight strain gauges pasted on the members of the model are shown in table 3.11. The strain data is also plotted for top chord and bottom chord members separately. The curves for load vs microstrain for each member are shown below.

The plot for all strain gauges mounted on 2 compression members (top chord members) is shown in Figure 3.27.

Table 3.11: Load versus recorded strain data for members

		Strain gauge channel number							
S.No.	Load (kN)	1	2	3	4	5	6	7	8
1	9.81	2.19E-06	0.000117	0.000153	0.000188	0.00018	6.68E-05	2.76E-06	0.000267
2	19.62	4.96E-05	0.000236	0.000264	0.000343	0.00027	0.000183	5.22E-05	0.000439
3	29.43	6.06E-05	0.000361	0.000334	0.000509	0.000331	0.00032	7.88E-05	0.000604
4	39.24	8.93E-05	0.000487	0.000391	0.000673	0.000393	0.000462	9.55E-05	0.000751
5	49.05	0.000115	0.000603	0.000437	0.000842	0.000464	0.000592	0.000113	0.00091
6	58.86	0.000146	0.000717	0.000476	0.001008	0.000552	0.000718	0.000132	0.001064
7	68.67	0.000177	0.00084	0.000527	0.001164	0.000626	0.000859	0.000181	0.001228
8	78.48	0.000227	0.000964	0.000568	0.001306	0.000693	0.001001	0.000212	0.001405
9	88.29	0.00029	0.001103	0.000622	0.001461	0.000757	0.001163	0.000245	0.001554
10	98.1	0.000366	0.001244	0.000684	0.001612	0.000826	0.001328	0.000275	0.001698
11	107.91	0.000388	0.00138	0.000742	0.001765	0.000868	0.001482	0.000304	0.001827
12	117.72	0.000423	0.001515	0.000784	0.001914	0.000905	0.001639	0.000333	0.001912
13	127.53	0.000466	0.001656	0.000829	0.002072	0.000939	0.001804	0.000366	0.002057
14	137.34	0.000504	0.001796	0.000869	0.002229	0.000974	0.001963	0.000396	0.002199
15	147.15	0.000536	0.001949	0.000902	0.002352	0.00099	0.002136	0.000428	0.002344
16	156.96	0.000547	0.002051	0.000929	0.002454	0.001003	0.002253	0.000443	0.002452
17	166.77	0.000574	0.002156	0.000947	0.002568	0.00103	0.002373	0.000478	0.002561
18	176.58	0.0006	0.002292	0.000969	0.002707	0.001043	0.002522	0.000508	0.002707
19	186.39	0.000609	0.00244	0.001012	0.002868	0.001055	0.002681	0.000552	0.002869
20	196.2	0.000635	0.002667	0.00106	0.003233	0.00106	0.002921	0.000626	0.003235

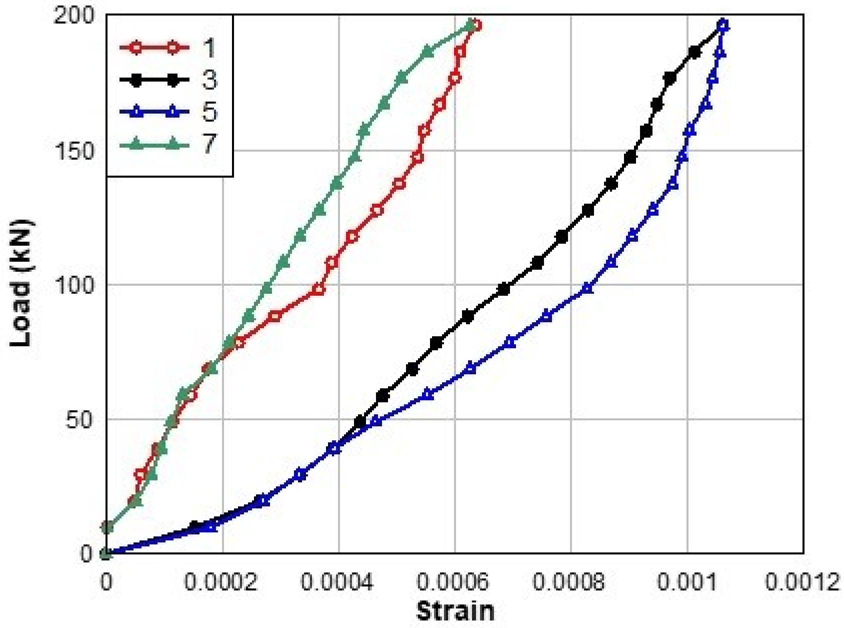


Figure 3.27: Load (kN) - strain curves for the four strain gauges pasted on the top members (Strain gauge number:1,3,5,7)

The plot for all strain gauges mounted on 2 tension members (bottom chord members) is shown in Figure 3.28. From the Figure, it is evident that both the tension members experienced similar strains.

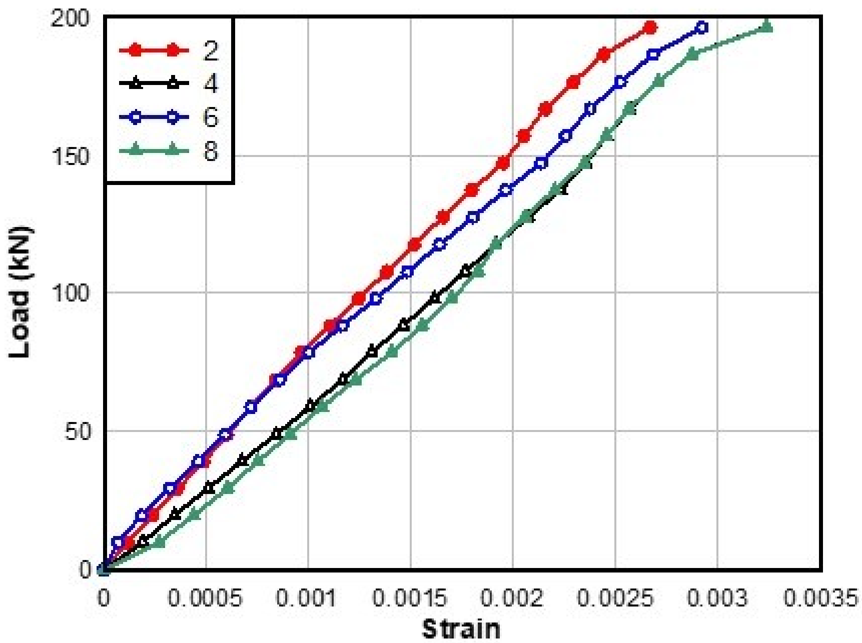


Figure 3.28: Load (kN) - strain curves for the four strain gauges pasted on the bottom chord members (Strain gauge number: 2,4,6,8)

The strain in the deck slab is recorded using five strain gauges mounted at 6.5 cm centre to centre from each other. The strain recorded at five locations in the deck slab is shown in table 3.12 for various load intervals. The strain variation is also plotted in Figure 3.29. The average strain in the deck slab at the failure load of 196.2 kN is 0.000431. This indicates that the deck slab at failure fully participates in the composite action. Strain in the deck slab significantly reduces from maximum strain at the top chord member locations from 0.00058 to minimum strain at the mid-section as 0.00026, which is reduced by 54.0%. Deck slab top view after failure is shown in Figure 3.26. From the Figure, it is clear that there is no adverse cracking or crushing of the deck slab at any location

Table 3.12: Strain recorded in the deck slab

Load (kN)	CH-10	CH-9	CH-11	CH-12	CH-13
0	0	0	0	0	0
19.62	7.96E-05	5.26E-05	5.54E-05	5.34E-05	8.26E-05
39.24	0.000126	8.16E-05	7.44E-05	8.86E-05	0.000136
78.48	0.000201	0.000138	8.57E-05	0.000145	0.000214
117.72	0.000305	0.000208	0.000151	0.000213	0.000322
156.96	0.000428	0.000274	0.000205	0.000285	0.000442
196.2	0.000555	0.000372	0.000265	0.000388	0.000576

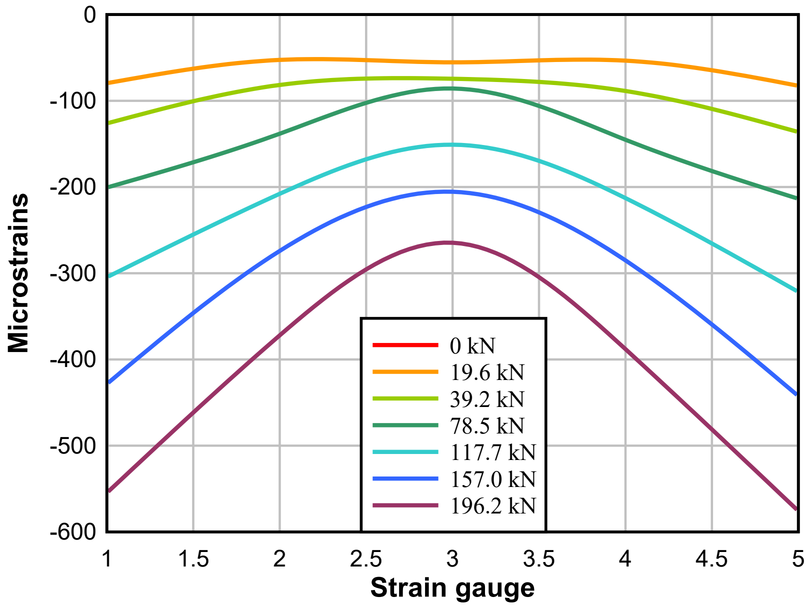


Figure 3.29: Deck Slab Strain Profiles at Different Loadings

Stress in the deck slab is also obtained from the STAAD analysis of the tested model. The STAAD editor file for the same is attached as annexure B. Stress contour can be obtained on all load steps and as an example; stress at 3t load is shown in Figure 3.30

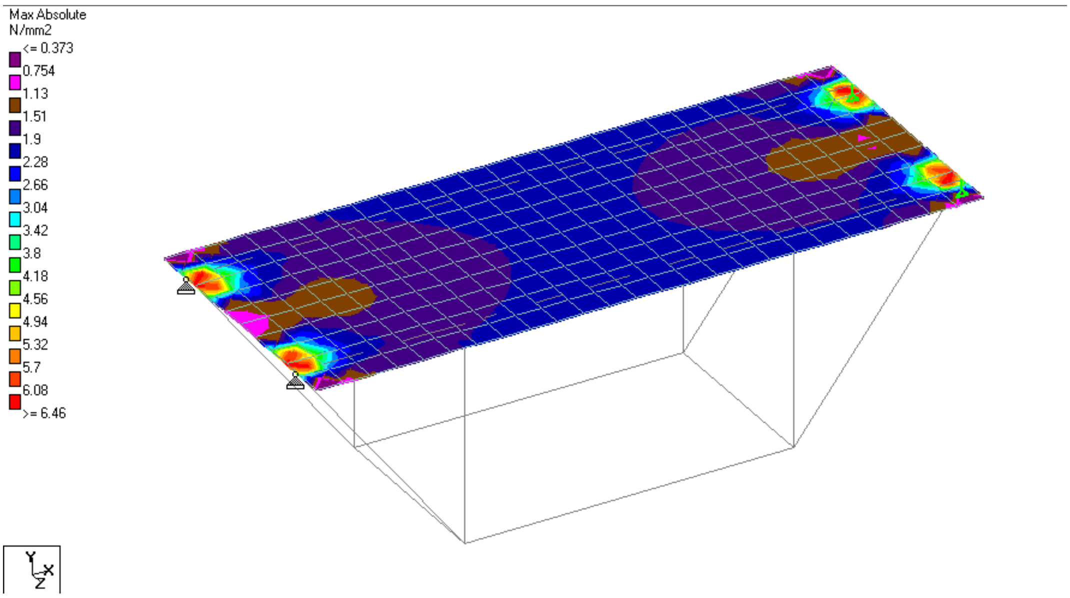


Figure 3.30: Stress in deck slab as per STAAD analysis at 3t load

The comparison of the average strain recorded in the deck slab and strain observed in the STAAD pro is plotted in Figure 3.31. From Figure 3.31 it can be seen that the concrete is showing relatively more strain in the beginning. This could be due to presence of micro shrinkage cracks in the deck.

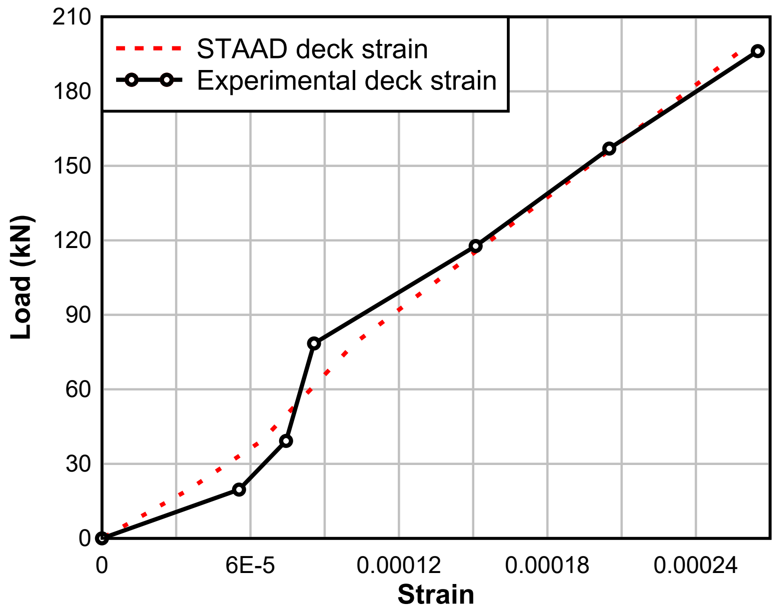


Figure 3.31: Comparison of recorded strain in deck slab and strain as per STAAD analysis

The strains recorded from the top chord, bottom chord and deck slab was averaged at each load step. The average strain recorded is shown in Figure 3.32. From the figure, it can be seen that there is a considerable difference between the strain observed in the top chord and the bottom chord. This is because of the load sharing by the deck slab. The average strain taken by the deck slab is lower than the top chord compression member. The strain in the deck slab is constantly rising until failure, which shows the participation of deck slab in composite action till failure.

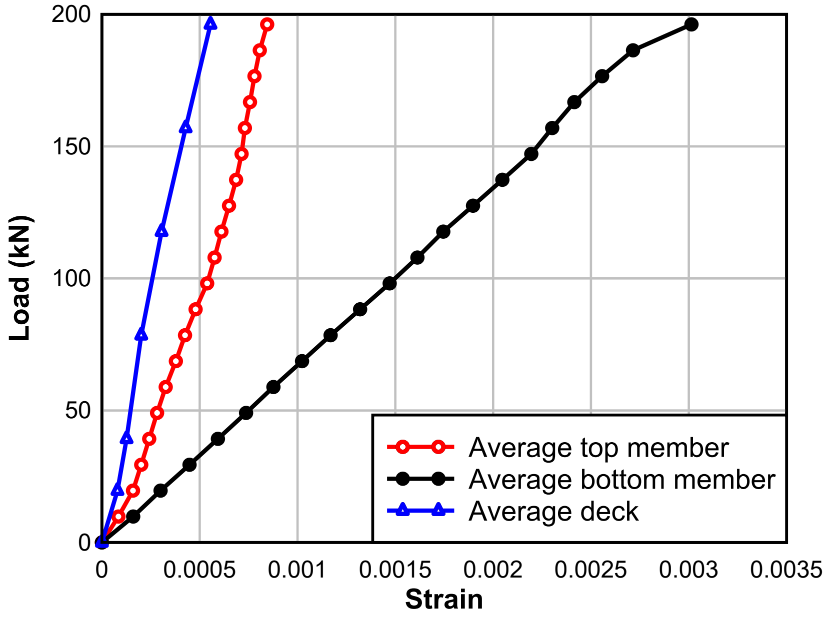


Figure 3.32: Average load - strain curves for top and bottom chords, and the deck slab at truss locations

Two dial gauges were used to measure the vertical deflection of the model. The readings from both the dial gauges are shown in table 3.13 and are plotted in Figure 3.33. The average of the dial gauge readings was taken and is compared with STAAD results are shown in Figure 3.34. The deflection in the composite bridge at a load of 147.15 kN was 3.95 mm. The experimental result closely tallies with the STAAD model result in which the deflection was 3.99 mm.

At the common load of 44 kN, the ratio of deflections for the composite bridge model and the truss model was 0.56. Thus, a composite bridge is preferable from the serviceability consideration. Due to composite action, a considerable change in stiffness was also observed. The stiffness for the composite model was 37.2 kN/mm and stiffness for the non-composite model was 20 kN/mm.

Table 3.13: Vertical deflection of the composite model

S.No.	Total load (t)	Dial gauge reading (x 10^{-2} mm)	
		1	2
1	0	0	0
2	9.81	23.5	28.5
3	19.62	43	55
4	29.43	74	76
5	39.24	100	107
6	49.05	127	148.5
7	58.86	154	174
8	68.67	180	194.5
9	78.48	204	220
10	88.29	229	258.5
11	98.1	259	276
12	107.91	285	303
13	117.72	312	343
14	127.53	340	367
15	137.34	367	380
16	147.15	390	404

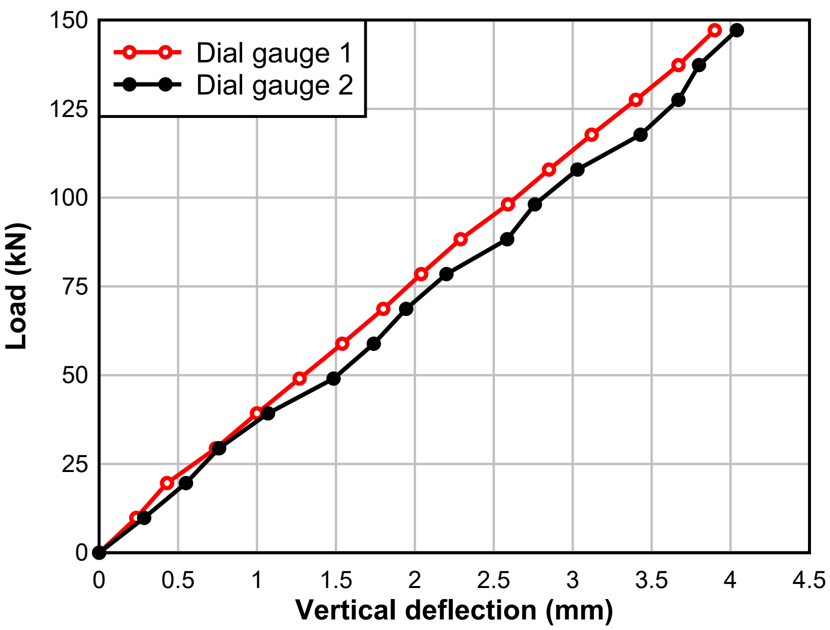


Figure 3.33: Load–deflection curves for dial gauges 1 and 2 for the composite model.

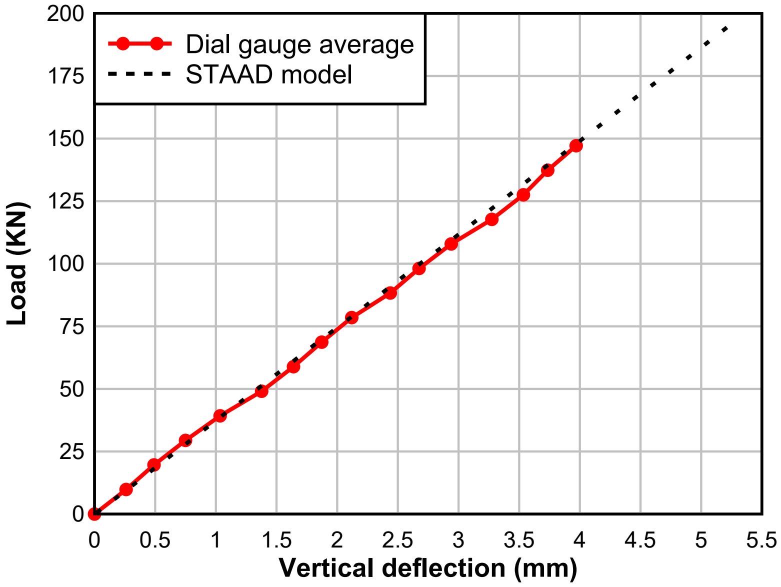


Figure 3.34: Average experimental deflections for the composite model, and corresponding STAAD deflection.