CHAPTER 4

STEEL-CONCRETE COMPOSITE TRUSS BRIDGE

4.1 GENERAL

Composite truss bridges are one of the most efficient and aesthetically attractive design solutions in bridge engineering (A. J. Reis, 2001). The most adopted and efficient solution consists in a under slung truss, where the deck slab acts in composite action with the compression chord. But, in the deck type composite bridges the concept only holds good at positive bending moment sections. In case of continuous composite double deck bridges, lateral support to the compression members is provided by the composite deck at the supports by the bottom deck, and at the mid spans by the top deck (A. Reis, *et. al.,* 2011). RCC deck slabs in deck type composite truss bridges provides lateral support to the compression members of the truss, and increases the cross section of the bridge, and thereby prevents buckling and premature failure of the compression members.

Composite construction of RCC floors with trusses is common in case of building construction, and with steel plate girders in case of composite plate girder bridges. Not much literature is available for design and construction of composite steel truss bridges. Despite unavailability of proper design guidelines and design standards for composite truss bridges, various types of composite truss bridges have been constructed in the World (F. Millanes, 2008). In India construction of through type non-composite steel truss bridges is most popular, and construction of deck type composite steel truss bridges is not in practice. Hence further research in this area is required to avail benefits of the composite construction.

In this chapter behaviour of composite truss bridges is discussed from basic principles of mechanics. The stresses developed in the deck slab and steel truss members are highlighted in view of composite action in steel truss bridges. Due to shrinkage strain of 0.0003 in deck slab concrete after its casting, complete composite action between RCC deck and steel truss is not possible. Effect of shrinkage strain on composite action is discussed in detail. Unlike simply supported composite truss bridges, in continuous composite truss bridges, deck slab is in tension at supports due to hogging moment at intermediate support locations. In addition to the shrinkage strain, tensile strain due to hogging moment causes more tensile cracks and hence composite action between RCC deck and steel truss is not possible in hogging moment zone. Therefore, simply supported composite truss bridges are more advantageous than continuous composite truss bridges. An innovative design procedure considering all above mentioned criterions has been proposed in this chapter. Design guidelines for design of composite truss bridge in service condition, at limit state of strength for overload condition and at ultimate collapse are proposed. Also, based on these proposed design guidelines, a comparative study of 90.0m span composite and non composite simply supported deck type truss bridge is carried out.

4.2 CONCEPT OF COMPOSITE TRUSS BRIDGE

A reinforced concrete or composite deck slab is required in bridges to provide a flat surface. The members in compression chord (top chord) of a simply supported deck type steel truss bridge may prematurely buckle before the stresses reach the full material strength. In this context composite action of the RCC slab with the truss compression chord becomes useful and prevents its buckling. Using RCC deck slab as a composite part of the bridge prevents buckling of the truss top chord members and carries compression with the truss system, ensuring higher ultimate strength of the bridge and warning before its failure in terms of large deflection. Concrete has lower strength compared to structural steel, and hence requires larger cross sectional area to support the loads. Consequently, the composite RCC deck slab effectively prevents buckling of the top chord members. Thus, in a composite truss bridge the relative merits of steel and concrete as construction materials are fully exploited individually as well as in combination. Composite truss systems are structurally efficient and economical. Considering functional and structural efficiency and economy, it is only natural that composite steel-concrete truss bridges are a good choice for medium span bridges. Longitudinal shear transfer in a composite deck type steel truss bridge, between the steel truss and concrete deck slab, is mobilised using shear studs (Figure 4.1).

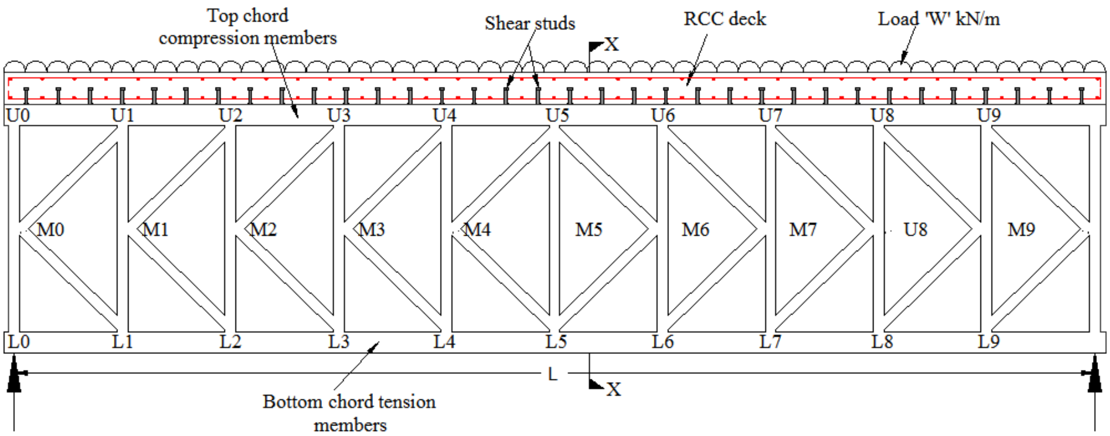


Figure 4.1 Composite deck type steel truss bridge

Due to shrinkage strain in the deck slab of a deck type composite truss bridge, composite action between the steel truss and the deck slab starts only when shrinkage strain is overcome by the flexural stresses under the live load condition. Thus, advantage of the composite section in terms of increased cross sectional area may be derived only near or after full live load condition. Therefore, the steel truss shall be designed for service condition with fatigue for full dead load and live load, and advantage of the composite section may be available in the overload condition. As a result, under the overload condition, sections of the laterally supported top chord compression members, tension members and gusset plates need not be increased from the service condition requirement, and cross sectional areas of only web compression members need be suitably increased.

4.3 EFFECT OF SHRINKAGE STRAIN ON COMPOSITE ACTION

In the deck type composite steel truss bridge, steel truss is first launched and then casting of deck slab takes place and after that super imposed dead loads (SIDL) are applied. After casting and hardening of the deck slab, the bridge is open to traffic. During hardening of concrete in deck slab, shrinkage cracks initiates. Shrinkage strain of the deck slab concrete does not permit composite action of RCC deck with the top chord members of the steel truss, until it is overcome by the flexural strain under the live load or overload condition. Reinforcement present in the RCC deck resists the initiation of shrinkage cracks to some extent. But it has been assumed on the safe side, that composite action between RCC deck and steel truss will not take place until strain in the deck slab under live load or overload condition exceeds the shrinkage strain.

Strain variation under different loads with shrinkage effect for a composite truss bridge is shown in Figure 4.2. After hardening of deck slab, due to shrinkage strain (ε_{sh}) total dead load is carried by steel truss only. During live load there is further increase in strain in top chord as well bottom chord members. In this stage some part of shrinkage strain in deck slab is overcome due to flexural strain. As live load on bridge goes on increasing in overload condition, remaining shrinkage strain (ε_{sh}- ε_{LL1}) shall overcome by flexural strain in deck slab and composite action between RCC deck and steel truss starts taking place. During overload condition or at plastic condition, neutral axis shifts in deck slab and compression shall be carried by RCC deck only.

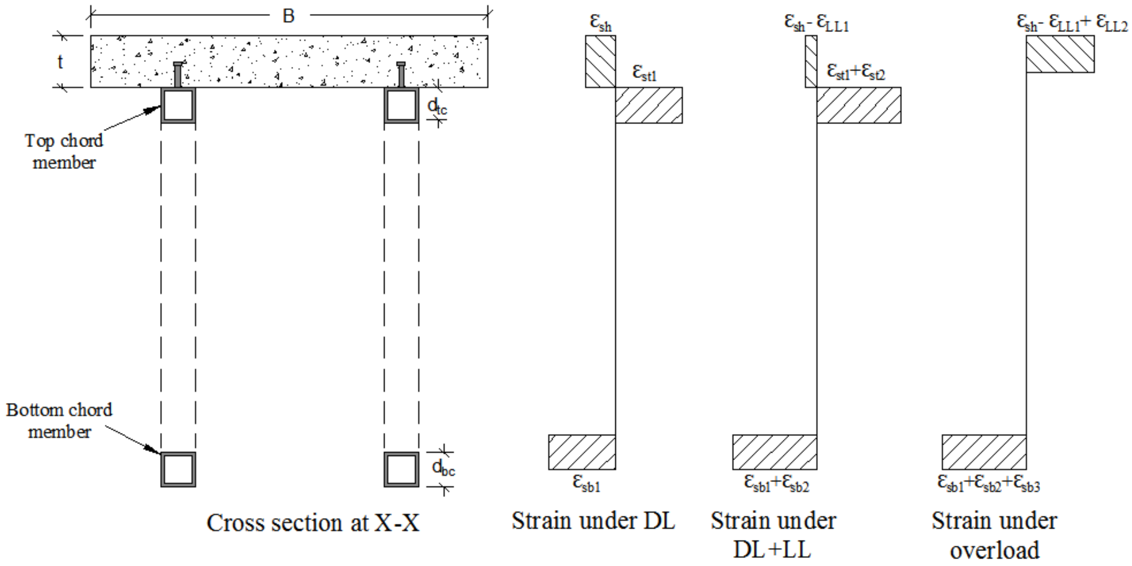


Figure 4.2 Strain diagram

4.4 CODE PROVISIONS FOR COMPOSITE TRUSSES IN BUILDINGS

Since the mid of 1960's many investigations have been made in testing composite trusses for buildings mainly in USA and Canada. The experimental results led to design recommendations and specification for composite open web steel joists (OWSJ) given in Standard Specification for Composite Steel Joists, C-J series (SJI-CJ-2010) of American National Standard and Handbook of Steel Construction on Open-Web Steel Joists (CAN/CSA S16.1, 1997) of Canadian Institute of Steel Construction. The design guidelines given in Canadian and American codes are confined to composite truss system used in building structures only.

There is no particular recommendation for the design of composite truss bridges in Eurocode 4 (ENV 1994-2, Eurocode 4), except the formulas for the local effect of a concentrated longitudinal force and the distribution of the longitudinal shear force into local shear flow between steel section and concrete slab (A. Bouchair, *et. al.,* 2012). In fact, the longitudinal forces are introduced into the concrete slab only locally, at points of increase of the axial force in the chord, i. e. where the web members are connected to the compression chord (panel points).

In Indian code IRC:22-2008, for steel concrete composite construction, there are no provisions for the design of composite steel truss bridges or buildings.

Design specifications stipulated in different international codes on composite trusses for building structures are discussed in the following section to establish the design guidelines for steel concrete composite truss bridge.

4.4.1 Code provisions in the American code SJI-CJ-2010

The design of Composite Steel Joists shall be based on achieving the nominal flexural strength of the composite member and is limited to the design of one-way, composite joist systems that meet the following criteria:

a) Members are simply-supported and are not considered part of the lateral load-resisting system.

b) Horizontal shear connection is achieved by direct bearing of embedment within the concrete slab.

Where any applicable design feature is not specifically covered in SJI-CJ-2010, the design shall be in accordance with the following specifications:

a) For steel that consists of hot-rolled shapes, bars, or plates, use of the American Institute of Steel Construction, Specification for Structural Steel Buildings is recommended.

b) For members, which are cold-formed from sheet or strip steel, use of the American Iron and Steel Institute, North American Specification for the Design of Cold-Formed Steel Structural Members is recommended.

Load combinations

At a minimum, the required stress shall be computed for the factored loads based on the factors and load combinations as follows:

(a) Non-composite

Design load $=1.4D_c$ or $1.2D_c + 1.6Lc$, where,

D_c = construction dead load due to weight of the joist, the decking, and the fresh concrete

L_c = construction live load due to the work crews and the construction equipment

(b) Composite

Design load = 1.4D or 1.2D + 1.6 (L, or L_r, or S, or R), where,

D = dead load due to the weight of the structural elements and the permanent features of the structure

L = live load due to occupancy and movable equipment, lb/ft.2 (kPa)

L_r = roof live load, when composite joists are utilized in roofs, lb/ft.2 (kPa)

S = snow load, when composite joists are utilized in roofs, lb/ft.2 (kPa)

R = load due to initial rainwater or ice exclusive of the ponding contribution, when composite joists are utilized in roofs

Design of members

(a) Top and bottom Chords

(i) Non-composite Design

The bottom chord shall be designed as an axially loaded tension member. The top chord shall resist the construction loads, at which time the joist is behaving non-compositely. An analysis shall be made using an effective depth of the joist to determine the member forces due to construction loads. The effective depth for a non-composite joist shall be considered the vertical distance between the centroids of the top and bottom chord members.

(ii) Composite Design

The distance between the centroid of the tension bottom chord and the centroid of the concrete compression block, de, shall be computed using a concrete stress of

$0.85f_c'$ and an effective concrete width, be, taken as the sum of the effective widths for each side of the joist centreline, each of which shall be the lowest value of the following:

a. one-eighth of the joist span, center-to-center of supports;

b. one-half the distance to the center-line of the adjacent joist;

c. the distance to the edge of the slab.

$$A = M_n /(0.85\ f_c'\ b_e\ d_e) \leq tc, \text{ in. (mm)}$$

$$d_e = d_j - y_{bc} + t_c - a/2, \qquad \text{where,}$$

a = depth of concrete compressive stress block, in. (mm)

b_e = effective width of concrete slab over the joist

d_j = steel joist depth, in

f_c = specified minimum 28 day concrete compressive strength

M_n = nominal moment capacity of the composite joist

t_c = thickness of concrete slab above the steel deck

y_{bc} = vertical distance to centroidal axis of bottom chord measured from the bottom of the bottom chord

The contribution of the steel joist top chord to the moment capacity of the composite system shall be ignored. The contribution of the steel joist top chord to the moment capacity of the composite system shall be ignored. The first top chord end panel member shall be designed for the full factored load requirements as a non composite member.

$$M_u \leq \phi M_n$$

ϕM_n = minimum design flexural strength of composite section

M_u = required flexural strength determined from applied factored loads

The design flexural strength of the composite section, ϕM_n, shall be computed as the lowest value of the following limit states:

a) Bottom Chord Tensile Yielding: $\phi_t = 0.90$, $\phi M_n = \phi_t A_b F_y d_e$

b) Bottom Chord Tensile Rupture: $\phi_{tr} = 0.75$, $\phi M_n = \phi_{tr} A_n F_u d_e$

b) Concrete crushing: $\phi_{cc} = 0.85$, $\phi M_n = \phi_{cc} f_c' b_e t_c d_e$

d) Shear Connector Strength: $\phi_{stud} = 0.90$, $\phi M_n = \phi_{stud} N Q_n d_e \geq 0.5 \phi_t A_b F_y d_e$

Where,

A_b = cross-sectional area of steel joist bottom chord

A_n = net cross-sectional area of the steel joist bottom chord

b_e = effective width of concrete slab over the joist

d_e = vertical distance from the centroid of steel joist bottom chord to the centroid of resistance of the concrete in compression

F_u = tensile strength of the steel joist bottom chord

F_y = specified minimum yield stress of steel joist bottom chord

N = number of shear studs between the point of maximum moment and zero moment

t_c = minimum thickness of the concrete slab above the top of the metal deck

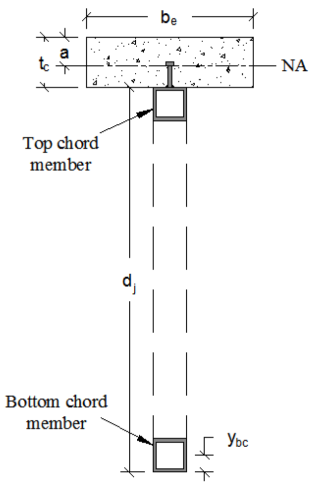


Figure 4.3 Cross section of composite joist

(b) Design of web members

Vertical shears to be used in the design of the web members shall be determined from the controlling load combination but such vertical shears shall not be less than the following:

i) 25% of the factored end reaction.

ii) Tension web members controlled by (i) shall be designed for a compressive force resulting from a factored shear value of:

$$V_{c\min} = \frac{(1.6W_L)L}{8}, \qquad \text{where,}$$

w_L = non-factored live load due to occupancy and moveable equipment

L = design length for the composite joist

$V_{c\,min}$ = minimum factored compressive design shear in tension web members

4.4.2 Canadian code provisions in CAN/CSA S16.1

As per provisions given for composite truss and Open Web Steel Joist (OWSJ) for buildings in the Canadian code CISC 2003 (CAN/CSA S16.1, 1997), design of the bridge at collapse in plastic condition is carried out as given below.

OWSJs are designed for loads acting in the plane of the joist applied to the top chord, which is assumed to be prevented from lateral buckling by the deck. For the purpose of determining axial forces in all members, members are assumed to be pin-connected and the loads are replaced by statically equivalent loads applied at the panel points.

As per cl. 17.9.5 of CISC 2003, for full shear connection, the total horizontal shear, V_h, at the junction of the steel section, truss, or joist and the concrete slab or steel deck, to be resisted by shear connectors distributed between the point of maximum bending moment and each adjacent point of zero moment shall be taken as the maximum force in the bottom chord tension member.

Force equilibrium diagram at mid section is given in Figure 4.4 Total tension in the bottom chord member is taken as $0.85A_sF_y$. Force of compression in the top chord member shall be equal to force of tension at the mid section, and it is given by $0.85\phi_c btf_c'$. It has been assumed here that entire tension is taken by the bottom chord and equal compression is taken by the composite deck slab above the neutral axis.

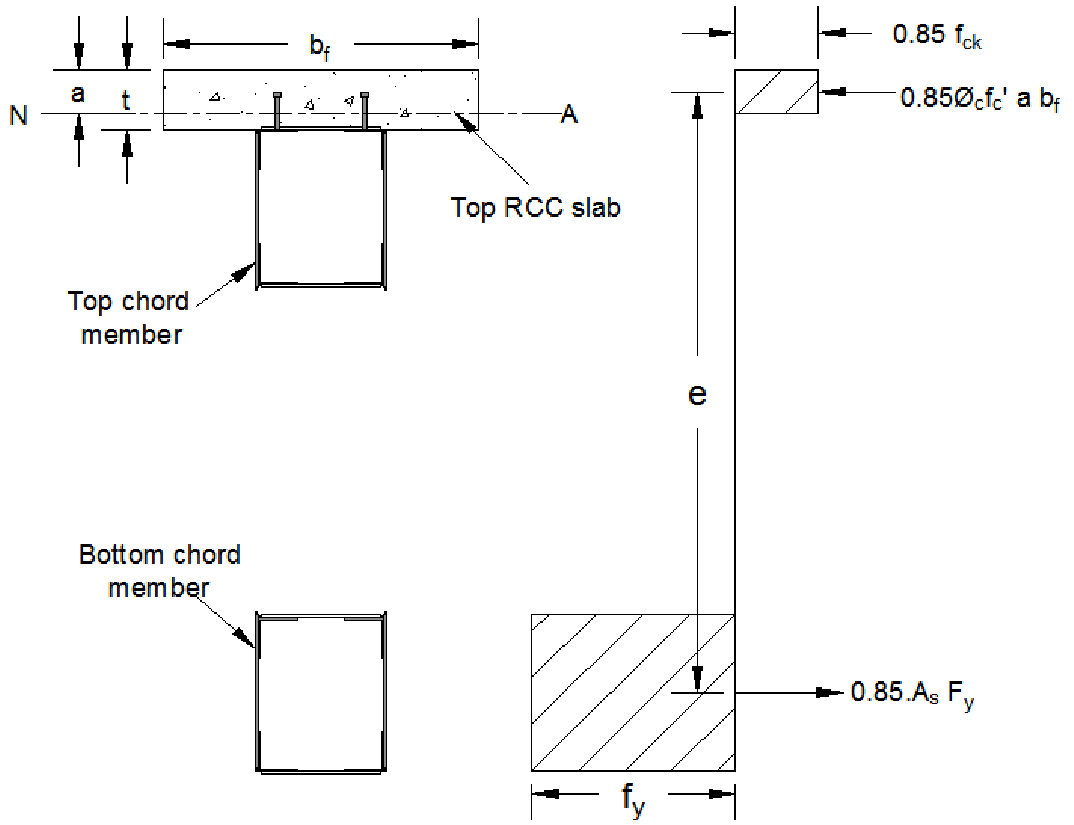


Figure 4.4 Force equilibrium of composite truss (CAN/CSA S16.1)

4.5 PROPOSED ANALYSIS AND DESIGN GUIDELINES FOR COMPOSITE STEEL TRUSS BRIDGES

There are no design guidelines available for design of steel concrete composite bridge in any design code. In the present research work, an attempt has been made to propose innovative design guidelines for the design of steel concrete composite truss bridge. The following steps are proposed to be followed for the design of composite steel truss bridges.

A. Modeling of the bridge

B. Analysis under various loading conditions

C. Design of the bridge

4.5.1 Modelling of the bridge

Traditionally trusses are designed assuming that the nodes of truss are pin jointed and frictionless. This assumption leads to the design of trusses only for axial forces. Members of a truss meeting at a common joint are generally connected by using gusset plates with the help of multiple rows of rivets or welds. These riveted or welded joints in a truss do not form a pin jointed and frictionless joint which leads to presence of bending moment in the truss members in addition to the axial forces. Therefore, 3D space frame analysis shall be carried out for composite steel truss bridges.

4.5.2 Analysis under various loading conditions

For design of composite truss bridges following primary loads and their combinations are recommended for analysis.

a. Primary loads

Primary loads considered for the analyses are given in Table 4.1.

Table 4.1 Primary Loads

Load no.	Name	Remark
1	EQZ	Earthquake load in transverse direction
2	EQX	Earthquake in longitudinal direction
3	SW	Self-weight of truss members including 20% extra for lacings gussets plates
4	SL	3.5 kN/m^2 extra load during deck slab casting due to shuttering and equipment.
5	DS	Weight of deck slab and wearing coat applied on top chord and stringers as distributed longitudinally

6	CB	Load due to crash barrier on top chord members as UDL
7	WL	Wind load in transverse direction (2.0kN/m^2)
8	FPLL	4.0 kN/m^2 load on Foot path.
9	DL	Total dead load =1.2xSW+DS+WC+CB
10	LL	LL as per design standards for vehicle load on bridge deck

b. Load Combination

Following load combinations shall be followed for design of composite steel truss bridge.

i. Service condition

Under service condition design shall be carried out for - [DL + LL] case.

During service condition, DL includes self weight of truss and deck slab, and super imposed dead load. Maximum live load effects shall be taken in the combination.

ii. Construction stage

Launching of the truss shall be assumed to have been safely carried out using false pier or launching nose etc., as the case may be. Safety of the truss during deck slab casting shall be checked with self weight, deck slab, shuttering and equipment loads, for which additional load factor of 1.5 is recommended. For the 1.5 x [DL + SL] case, 30% increase in permissible stress as per cl. 202.3 of IRC:6-2010 is taken.

In the construction stage, design shall be checked for - $\frac{1.5}{1.3}$x[DL + SL] condition.

iii. Load combination with seismic load

Design shall be checked under seismic load, for longitudinal and transverse seismic cases with 20% live load (cl. 202.3, IRC:6-2010) and permissible increase in stress of 50% (cl. 202.3, IRC:6-2010).

In seismic condition design shall be checked for $\frac{1}{1.5}$ x[DL+0.2xLL+ EQZ/EQX].

iv. Load combination with wind load

Design shall be checked under wind load condition, for transverse wind load case with full live load (cl 202.3, IRC:6-2010) and permissible increase in stress of 33% (cl 202.3, IRC:6-2010).

In wind load condition design shall be checked for - $\frac{1}{1.33}$x [DL + LL +WL] condition.

v. Limit state of strength for overload condition

At limit state of strength for overload condition, the design shall be checked with a load factor of 2.25 for - 2.25 x [DL + LL] condition.

Permissible stress in this condition shall be taken without the fatigue factor of 1.5 (cl. 507.1.2.1, IRC: 24-2010). If the permissible stress is taken for service condition with a fatigue factor of 1.5, then the load factor will effectively reduced to 1.5.

Since strength is the main criterion and elasto-plastic deformation of the bridge is a desirable feature, premature failure of the structure is prevented by suitably increasing cross sectional areas of laterally unsupported compression members. It is assumed that tension members and the gusset plates designed for service condition shall have sufficient reserve strength and their designs for service condition are safe in the overload condition also. Further, it shall be assumed that compression members, if laterally supported as in the case of deck type composite truss bridge, have sufficient reserve strength for the overload condition. Therefore, areas of only those compression members which are not laterally supported need be increased for the overload condition requirement.

c. Critical load combination

Envelops for different load combinations may be created in STAAD Pro. and envelopes corresponding to the most critical load combinations may be used for design.

4.5.3 Design for service and overload conditions

Design of the bridge shall be carried out for limit state of serviceability as per IRC:24-2010. In the limit state of serviceability a fatigue factor of 1/1.5 is used in the permissible stresses. In the design for overload condition, an additional load factor of approximately 1.5 for (DL+LL) shall be used, but the fatigue factor of 1.5 is not used. Therefore, limit state of serviceability and limit state of strength design results are essentially same.

Due to shrinkage, the RCC deck slab is not considered composite with the steel truss until shrinkage strain of the deck slab is overcome by the flexural compressive strain due to live load or overload condition. This is an assumption on the safe side for the design, as the presence of reinforcing steel in the deck slab will cause its composite action to some extent. However, the deck slab is considered to provide lateral support to the top chord compression members, and thereby effectively prevents their premature buckling and theoretically permitting stress in the members up to their ultimate strength.

Optimum design of the truss members is based on Interaction Ratio (IR). Interaction ratio is the ratio of stress in the truss member due to combined axial forces and biaxial bending moments to the permissible stress.

Therefore, Interaction Ratio (IR) = $\frac{\sigma_{a,cal}}{\sigma_a} + \frac{\sigma_{bx,cal}}{\sigma_{bx}} + \frac{\sigma_{by,cal}}{\sigma_{by}}$

Where,

$\sigma_{a,cal}$ = Calculated axial stress

$\sigma_{bx,cal}$ = Calculated bending stress about local axis X.

$\sigma_{by,cal}$ = Calculated bending stress about local axis Y.

σ_a = Permissible axial stress

σ_{bx} = Permissible bending stress about local axis X.

σ_{by} = Permissible bending stress about local axis Y.

In the design for the overload condition, interaction ratio is limited to 1.0 in all tension members and laterally restrained top chord compression members. For laterally unsupported compression members it is limited to 0.66.

4.6 RESERVE STRENGTH AT PLASTIC COLLAPSE

There is no direct literature available for calculation of reserve strength of the composite steel truss bridges in plastic stage (J. Machaceka, *et. al.*, 2013). Therefore, design of the bridge at collapse in plastic condition is carried out as per provisions given for composite truss and OWSJ (Open Web Steel Joist) for buildings in CISC 1997 (CAN/CSA S16.1, 1997) and ASCE Task Committee on Design Criteria for Composite Structures in Steel and Concrete (ASCE Task Committee, 1996). The design provisions have been modified in accordance with IS: 456-2000 and IRC: 24-2010.

For capacity calculation of the bridge at plastic collapse, it has been assumed that the web members are sufficiently strong enough and will not fail before failure of other members. As given by Cran (J. A. Cran, 1972) and Bouchair, J. Bujnak and P. Duratna (A. Bouchair, *et. al.*, 2012) area of steel top chord members is neglected in calculating the moment of resistance.

Details of the composite bridge model for the plastic stage are given in Figure 4.5.

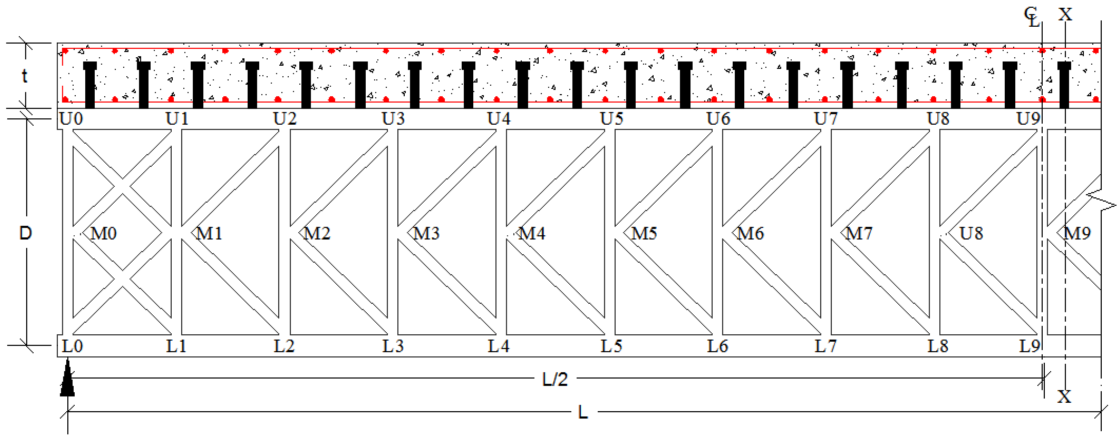
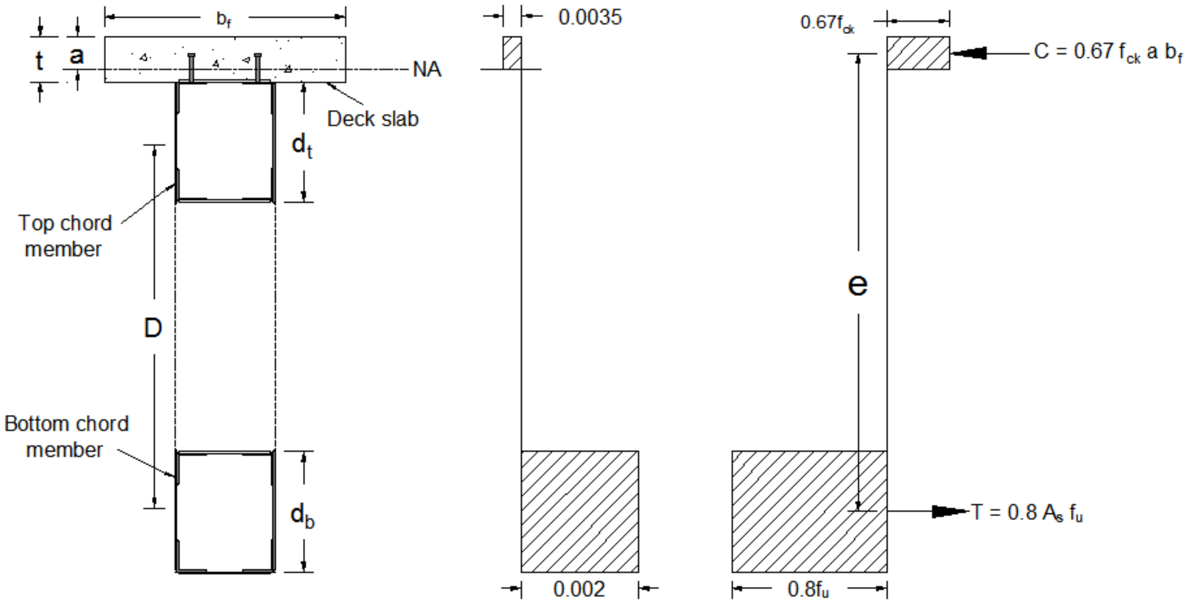


Figure 4.5 Composite truss bridge model for plastic failure

The bridge for plastic analysis is considered at section X-X (Figure 4.5), which is adjacent to the centre line. Thus, at the section X-X only bottom chord member and composite top chord carry axial forces.

Force equilibrium diagram at the mid span section of the bridge is given in Figure 4.6.



a. Cross section at x-x   b. Strain diagram   c. Stress diagram

Figure 4.6 Force equilibrium at plastic collapse

Where,

a	=	depth of neutral axis from top of the deck slab
A_{sb}	=	cross-sectional area of steel bottom chord,
A_{st}	=	cross-sectional area of steel top chord
D	=	Center to center distance between top and bottom chord members
d_t	=	depth of top chord member
d_b	=	depth of bottom chord member
t	=	thickness of deck slab
e	=	lever arm
b_f	=	effective width of slab

f_{ck} = characteristic strength of concrete

f_u = specified ultimate strength of steel.

Equating compression and tension forces; $ab_f \frac{f_{ck}}{\gamma_c} = \frac{A_s f_u}{\gamma_{ml}}$

Taking, $\gamma_c = 1.5$ and $\gamma_{ml} = 1.25$

$$0.67 ab_f f_{ck} = 0.8 A_s f_u$$

or
$$a = \frac{0.8 A_s f_u}{0.67 b_f f_{ck}}$$

The plastic moment of resistance (M_p) of the composite section is computed as,

$$M_p = A_{sb} \times f_u \times e = A_{sb} \times f_u \times (D + d_t/2 + t - a/2) = M_p \text{ kNm}$$

For the complete bridge, the total plastic moment of resistance = 2 x M_p kNm

Corresponding equivalent udl on the bridge 'W_p' is given by; $\frac{WpL^2}{8}$ = 2 x M_p

or $W_p = \frac{16 M_p}{L^2}$ kN/m

Applied udl in service condition, (DL+LL) = W_s kN/m

Applied udl at limit state of strength, 1.5x(DL+LL) = 1.5xW_s kN/m

Therefore, factor of safety at plastic collapse with respect to service load = W_p/W_s

Factor of safety at plastic collapse with respect limit state of strength load = $W_p/1.5W_s$

Thus, in composite steel truss bridge, there is a factor of safety against plastic collapse in comparison to service condition as well as in comparison to the limit state of strength condition.

4.7 COMPARISON BETWEEN NON - COMPOSITE AND COMPOSITE DECK TYPE TRUSS BRIDGES.

In this section two deck type steel truss bridges (one composite and other composite type) of span 90m are analyzed and designed for overload condition of 1.5

times service load condition. In one bridge composite action between steel truss and RCC deck is taken into consideration and in other case bridge is considered as non-composite. These two bridges are designed as per the design recommendations made in the present research.

4.7.1 Geometric details and bridge model

Geometric details of the 90m span bridge model are given below.

i. Height of Truss (C/C distance between top chord and bottom chord members) = 10m.

ii. C/C distance between two trusses = 6.0 m

iii. Width of roadway = 5.5 m

iv. Panel length = 5m

v. Number of 5m top panels = 18

vi. Number of 5m bottom panels = 18

vii. C/C distance between cross girders = 5m

Half elevation of the 90m span deck type truss bridge is given in Figure 4.7.

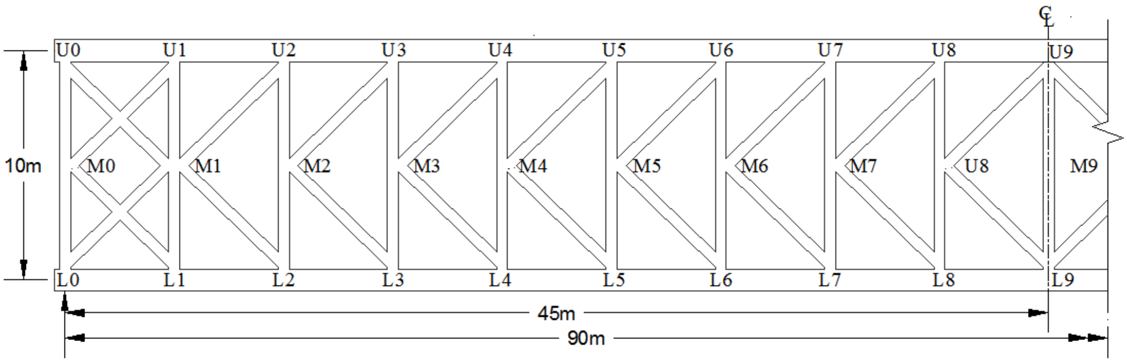


Figure 4.7 Elevation showing half span of 90m truss bridge

A 3D space frame model (Figure 4.8) is prepared using STAAD Pro. v8i software for the analysis of the 90m span deck type composite and non-composite truss bridge. In non composite case deck slab is not considered as structural element in the model whereas in composite case, desk slab is integrated part of the truss.

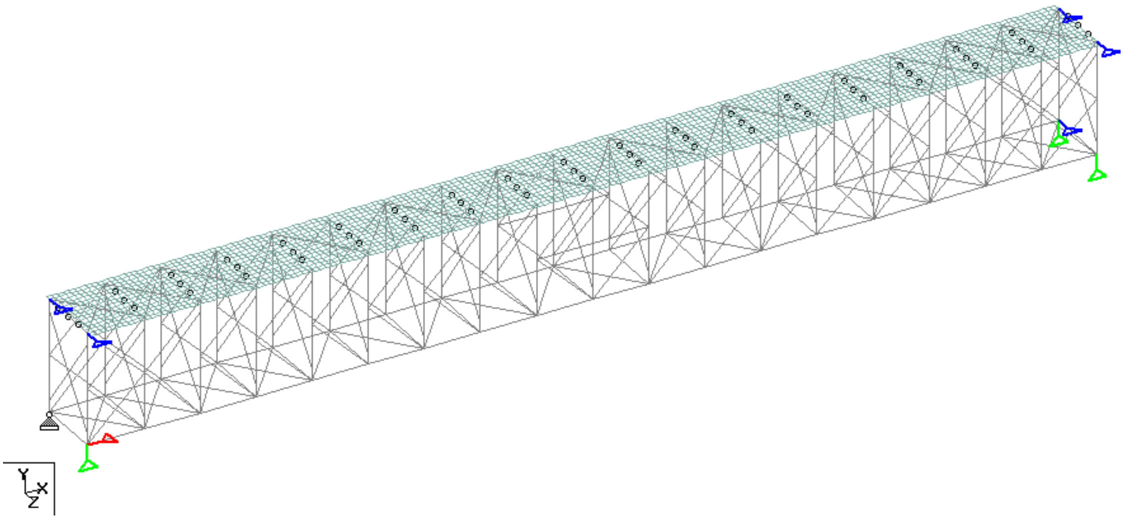


Figure 4.8 STAAD model for composite truss bridge

Cross section of the 90m span simply supported composite truss bridge is given below in Figure 4.9.

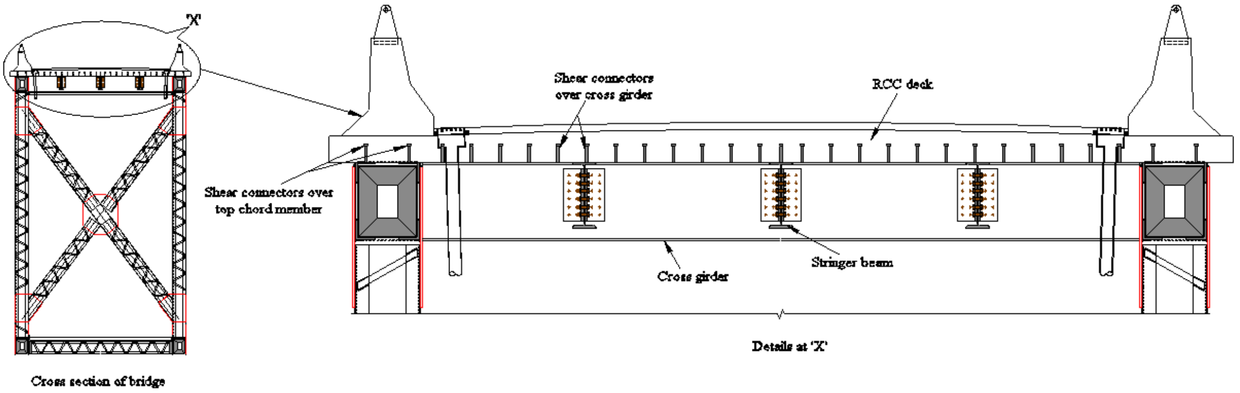


Figure 4.9 Cross section of 90m span truss bridge

In the non-composite truss bridge case, deck slab is not made composite with the top chord members of the truss, and hence top chord members of the truss are designed for 2.25x(DL+LL) case in the overload condition to avoid buckling.

Similarly, for the composite truss bridge a space frame model is prepared where deck slab is modeled with plate elements. It is assumed that the shear connectors are spaced at 300mm c/c in four rows, and accordingly deck panels are meshed in rectangular shape with edge length of 300mm. The purpose of meshing of deck slab is to provide connectivity between RCC deck and steel members due to shear studs. Figure 4.5 shows STAAD model for composite truss bridge.

4.7.2 Material properties

Properties of structural steel:

Grade of steel	= E250
Yield strength (f_y)	= 250 N/mm^2
Ultimate stress (f_u)	= 410 N/mm^2
Young's modulus of elasticity, E_s	= 2.0 x 10^5 MPa
Poisson's ratio	= 0.3
Shear modulus	= 76.9 x 10^3 MPa
Coefficient of thermal expansion	= 1.2 x 10^{-5} /°C

Properties of concrete:

Grade of concrete	= M40
Modulus of elasticity	= 31622.7 N/mm^2

4.7.3 Loading on the bridge

Following primary loads on the bridge are considered.

Dead load

i. Self weight of the truss including 20% additional weight due to gusset plate, lacings, batten plates and rivets or bolts.

ii. Load due to deck slab.

   Thickness of deck slab is 250mm at centre and uniformly decreases to 200mm towards curb. Therefore, average thickness of the deck slab is taken as 225mm.

   Deck slab is resting on stringers, cross girders and top chord members. Loads from the deck slab are primarily taken as UDL on top chord members and stringer beams.

iii. Load due to wearing coat.

Wearing coat is 56mm thick and its unit weight is taken as 22 kN/m^3.

iv. Self weight of crash barrier

Cross sectional area of the crash barrier is $0.27m^2$ and loading on top chord member due to this is taken as 7.5 kN/m

v. Temporary load due to shuttering and equipments

During casting of the deck slab, temporary load due to shuttering and equipments is taken as 3.5 kN/m^2.

Live load

The bridge is analyzed for three trains of single lane Class-A wheeled vehicles in 3.5m width of bridge and pedestrian live load of 4 kN/m^2 is considered in remaining 2m width of carriageway as per IRC: 6-2010. Two trains of Class-70R loading are also considered.

Wind load

Wind pressure is taken as 2.0 kN/m^2 on exposed area of bridge across wind direction.

Seismic Load

Seismic loading on the bridge is taken for seismic zone-V.

4.7.4 Load Combinations

Following load combinations are taken as proposed under section 4.5.2.

1. Service condition - DL+LL

2. Construction stage - $\frac{1.5}{1.3}$x[DL + Load due to shuttering and equipments]

3. Seismic condition - $\frac{1}{1.5}$ x[DL+0.2xLL+ EQZ/EQX]

4. Wind load condition - $\frac{1}{1.33}$x [DL + LL +WL]

5. Overload condition - 2.25x[DL+LL] (without taking fatigue factor of 1.5)

4.7.5 Design of the bridge

As per IRC:24-2010, Standard Specifications and Code of Practice for Road Bridges, Section V, Steel Road Bridges, design of road bridges is carried out for limit state of serviceability and checked for limit state of strength and other limit states. In the limit state of serviceability the load factor for (DL+LL) case is 1.0 and in the permissible stresses a fatigue factor of 1/1.5 is used. In the limit state of strength design the corresponding load factor is approximately 1.5 and no fatigue factor is used. Therefore, both the approaches result in same design. As discussed in para. 3.4.1.2 of Chapter-3, higher load factor at the limit state of strength of 2.25 in place of 1.5 is recommended. This modified load factor of 2.25 is proposed in the present research work.

Design results of the 90.0m span non-composite and composite bridges have been compared for the following conditions.

i. Service condition under (DL+LL) case with 1/1.5 fatigue factor.

ii. Limit state of strength condition under 2.25x(DL+LL) case without 1/1.5 fatigue factor.

In the service condition, both for non-composite and composite bridges, interaction ratio in all the members is limited to 1.0.

At limit state of strength, non-composite truss bridge is designed using permissible stresses for service condition, but the interaction ratio in all compression members is limited to 0.66 and for tension members to 1.0. In the composite bridge case interaction ration for the web compression members only is limited to 0.66 and for all other members it is 1.0.

4.7.6 Optimized design results

Optimized design based on trial and error is carried out for the 90.0m span bridge in composite and non-composite cases as per detailed analysis and proposed design guidelines. Following are the design results for the 90.0m span deck type truss bridge in non composite and composite case.

Non-composite bridge under service condition and at overload condition

Optimized design results for half portion of the bridge for Non-composite bridge under service condition and at limit state of strength are given in Table 4.2.

Table 4.2 Design of non-composite bridge under service load and at overload

Member	Service condition			Overload condition		
	Section	Design axial force (kN)	Interaction ratio	Section	Design axial force (kN)	Interaction ratio
L0L1	4-ISA 150x150x10 2- PL 575x10	-1048.4	0.391	4-ISA 150x150x10 2- PL 575x10	-1332.6	0.413
L1L2	4-ISA 150x150x10 2- PL 575x10	-1564.3	0.548	4-ISA 150x150x10 2- PL 575x10	-1821.0	0.62
L2L3	4-ISA 150x150x10 2- PL 575x10	-2353.4	0.725	4-ISA 150x150x10 2- PL 575x10	-2511.3	0.779
L3L4	4-ISA 150x150x12 2- PL 575x12	-3312.0	0.956	4-ISA 150x150x12 2- PL 575x12	-3525.6	0.968
L4L5	4-ISA 150x150x16 2- PL 575x16	-4242.1	0.885	4-ISA 150x150x16 2- PL 575x16	-4517.1	0.953
L5L6	4-ISA 150x150x18 2- PL 575x18	-4922.1	0.957	4-ISA 150x150x18 2- PL 575x18	-5235.2	0.997
L6L7	4-ISA 150x150x20 2- PL 575x20	-5439.6	0.936	4-ISA 150x150x20 2- PL 575x20	-5780.3	0.968
L7L8	4-ISA 150x150x18 2- PL 575x18	-5820.7	0.961	4-ISA 200x200x18 2- PL 575x18	-6182.4	1.001
L8L9	4-ISA 200x200x18 2- PL 575x18	-6040.8	0.984	4-ISA 200x200x18 2- PL 575x18	-6402.5	1.028
U0U1	4-ISA 150x150x10 2- PL 575x10	472.9	0.343	4-ISA 150x150x10 2- PL 575x10	373.2	0.26
U1U2	4-ISA 150x150x10 2- PL 575x10	968.2	0.444	4-ISA 150x150x10 2- PL 575x10	857.6	0.427
U2U3	4-ISA 150x150x10 2- PL 575x10	2353.9	0.903	4-ISA 150x150x20 2- PL 575x20	2556.6	0.496
U3U4	4-ISA 150x150x12 2- PL 575x14	3394.9	0.924	4-ISA 150x150x20 2- PL 575x20	3618.3	0.644
U4U5	4-ISA 150x150x16 2- PL 575x16	4234.1	0.897	4-ISA 200x200x25 2- PL 575x25	4511.5	0.544
U5U6	4-ISA 150x150x18 2- PL 575x18	4913.7	0.916	4-ISA 200x200x25 2- PL 575x25	5227.8	0.615
U6U7	4-ISA 150x150x20 2- PL 575x20	5445.0	0.936	4-ISA 200x200x25 2- PL 575x25	5787.0	0.681

U7U8	4-ISA 200x200x18 2- PL 575x18	5819.2	0.931	4-ISA 200x200x25 2- PL 575x32	6185.3	0.661
U8U9	4-ISA 200x200x18 2- PL 575x18	6043.8	0.983	4-ISA 200x200x18 2- PL 575x32	6411.6	0.692
M0L0	2-ISMC 350 2- PL 370x25	1726.4	0.326	2-ISMC 350 2- PL 370x25	1544.2	0.575
M1L1	2-ISMC 250 2- PL 270x20	496.8	0.458	2-ISMC 250 2- PL 270x20	458.3	0.391
M2L2	2-ISMC 350 2- PL 370x14	882.6	0.762	2-ISMC 350 2- PL 370x14	758.4	0.415
M3L3	2-ISMC 350 2- PL 370x16	984.0	0.857	2-ISMC 350 2- PL 370x16	1047.0	0.548
M4L4	2-ISMC 350 2- PL 370x10	877.2	0.809	2-ISMC 350 2- PL 370x10	914.8	0.561
M5L5	2-ISMC 350 2- PL 370x10	706.4	0.945	2-ISMC 350 2- PL 370x10	750.8	0.479
M6L6	2-ISMC 250 2- PL 270x14	576.3	0.777	2-ISMC 250 2- PL 270x14	612.4	0.52
M7L7	2-ISMC 250	413.0	0.83	2-ISMC 250	429.1	0.537
M8L8	2-ISMC 200	287.2	0.518	2-ISMC 200	296.2	0.515
M9L9	2-ISMC 200	167.5	0.294	2-ISMC 200	149.9	0.269
M0U0	2-ISMC 350 2- PL 370x25	708.2	0.275	2-ISMC 350 2- PL 370x25	586.8	0.316
M1U1	2-ISMC 250 2- PL 270x20	-266.2	0.379	2-ISMC 250 2- PL 270x20	-243.0	0.312
M2U2	2-ISMC 350 2- PL 370x14	-1185.1	0.992	2-ISMC 350 2- PL 370x14	-1438.9	0.664
M3U3	2-ISMC 350 2- PL 370x16	-794.1	0.783	2-ISMC 350 2- PL 370x16	-864.2	0.424
M4U4	2-ISMC 350 2- PL 370x10	-641.6	0.684	2-ISMC 350 2- PL 370x10	-683.9	0.444
M5U5	2-ISMC 350 2- PL 370x10	-485.9	0.654	2-ISMC 350 2- PL 370x10	-516.3	0.398
M6U6	2-ISMC 250 2- PL 270x14	-340.0	0.529	2-ISMC 250 2- PL 270x14	-364.4	0.276
M7U7	2-ISMC 250	-210.4	0.412	2-ISMC 250	-198.5	0.364
M8U8	2-ISMC 200	194.1	0.467	2-ISMC 200	178.7	0.396
M9U9	2-ISMC 200	162.9	0.403	2-ISMC 200	145.1	0.373
L0M1	4-ISA 150x150x16 2- PL 575x16	1601.6	0.556	4-ISA 150x150x16 2- PL 575x16	2054.5	0.463
M0L1	2-ISMC 350	-728.9	0.731	2-ISMC 350	-701.0	0.671
M1U2	4-ISA 150x150x16 2- PL 575x16	2091.7	0.794	4-ISA 150x150x16 2- PL 575x16	2475.2	0.589
M2U3	2-ISMC 350 2- PL 370x25	1439.2	0.888	2-ISMC 350 2- PL 370x25	1524.0	0.608
M3U4	2-ISMC 350 2- PL 370x16	1223.8	1.048	2-ISMC 350 2- PL 370x16	1316.0	0.608
M4U5	2-ISMC 350 2- PL 370x10	1041.7	0.884	2-ISMC 350 2- PL 370x10	1094.4	0.598
M5U6	2-ISMC 300 2- PL 320x10	812.8	0.91	2-ISMC 300 2- PL 320x10	858.5	0.609
M6U7	2-ISMC 250 2- PL 270x14	615.4	0.861	2-ISMC 250 2- PL 270x14	652.7	0.602
M7U8	2-ISMC 250	418.8	0.607	2-ISMC 250	416.1	0.595
M8U9	2-ISMC 200 2- PL 220x10	275.0	0.746	2-ISMC 200 2- PL 220x10	310.2	0.657

M0U1	2-ISMC 350 2- PL 370x8	740.3	0.762	2-ISMC 350 2- PL 370x8	728.6	0.526
U0M1	2-ISMC 350	-675.0	0.487	2-ISMC 350	-546.3	0.499
M1L2	2-ISMC 350	-1195.0	0.842	2-ISMC 350	-1033.9	0.731
M2L3	2-ISMC 350 2- PL 370x10	-1443.6	0.668	2-ISMC 350 2- PL 370x10	-1528.4	0.67
M3L4	2-ISMC 350	-1227.3	0.863	2-ISMC 350	-1306.7	0.936
M4L5	2-ISMC 350	-1041.3	0.752	2-ISMC 350	-1091.8	0.769
M5L6	2-ISMC 300	-814.5	0.703	2-ISMC 300	-855.3	0.715
M6L7	2-ISMC 250	-616.5	0.635	2-ISMC 250	-651.5	0.684
M7L8	2-ISMC 250	-420.8	0.454	2-ISMC 250	-419.6	0.449
M8L9	2-ISMC 200	-220.8	0.383	2-ISMC 200	-216.1	0.383

In Service condition,

Maximum deflection = 143.7 mm

Steel off take as per STAAD result = 3122.4 kN

In Limit State of Strength case,

Maximum deflection = 183.2 mm

Steel off take as per STAAD result = 3730.2 kN

Composite bridge at service condition

In the composite bridge shear connectors are used between the top chord members and the deck slab as shown in Figure 4.10.

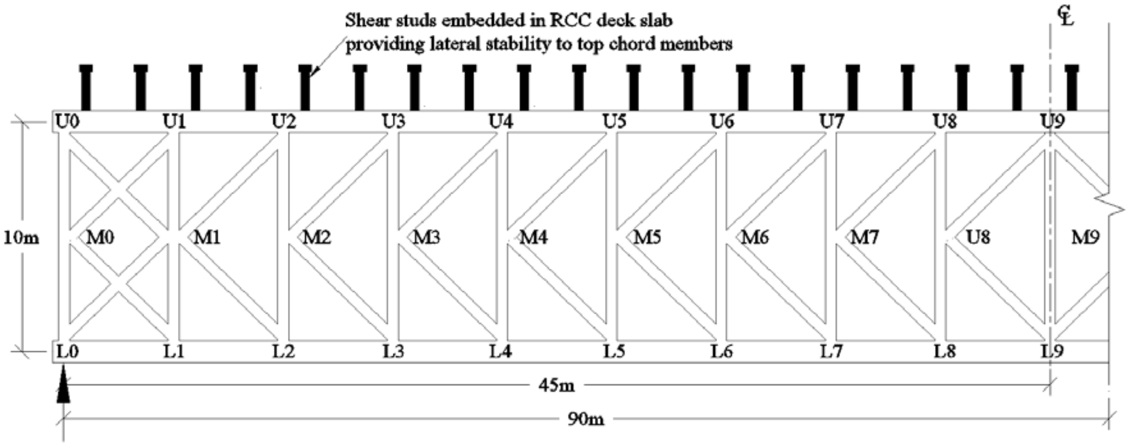


Figure 4.10 90m span composite truss bridge details

As per FEM analysis of a typical 90.0m span composite truss bridge in STAAD, maximum stress in the deck slab concrete at the mid span under only live load with impact condition is 3.91 N/mm^2 (Figure 4.11).

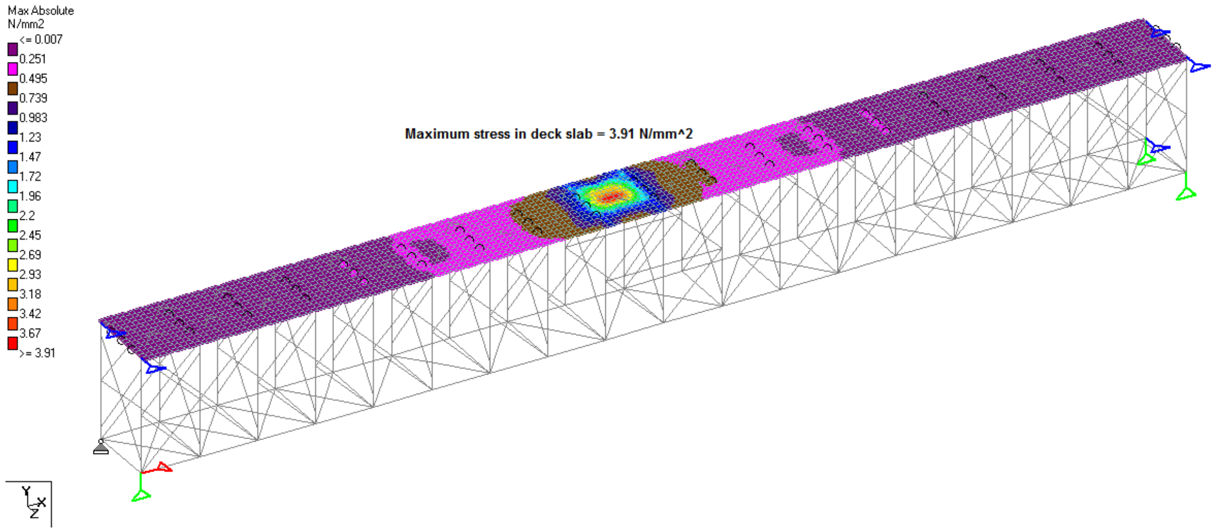


Figure 4.11 Deck slab stress under LL

Shrinkage strain in pre tensioned M40 grade concrete is specified to be 0.0003 in cl. 5.2.4 of IS: 1343-1980, Code of Practice for Prestressed Concrete.

Stress in the deck slab required to overcome shrinkage strain = 0.0003x5000x$\sqrt{fck}$

= 9.48 N/mm^2 > 3.91N/mm^2

As actual stress of 3.91 N/mm^2 in the deck slab under live load is less than the stress due to shrinkage strain of 9.48 N/mm^2, composite action even under full live load will not take place. Although, composite action is not possible between deck slab and truss, top chord compression members of the truss are laterally supported due to shear connectors and their buckling is prevented, and thereby higher compressive stress than the buckling stress up to the ultimate strength will take place in these members.

Due to shrinkage strain in the deck slab, the design remains unchanged from the non composite bridge case. Therefore, design results as produced above for the non composite case shall apply in this case also.

Composite bridge at overload condition

Details of the composite bridge model for overload condition are given in Figure 4.12.

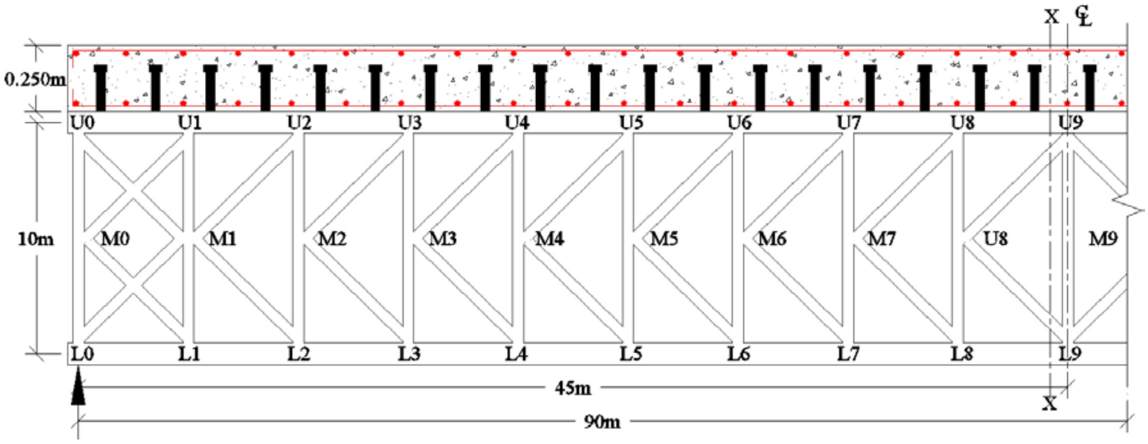


Figure 4.12 90m span composite truss bridge for 1.5x(DL+LL) case

Optimized design results for the half portion of the bridge are given in Table 4.3

Table 4.3 Design results at limit state of strength

Member	Section	Design force (kN)	Interaction ratio
L0L1	4-ISA 150x150x10 2- PL 575x10	-1304.1	0.404
L1L2	4-ISA 150x150x10 2- PL 575x10	-1779.2	0.621
L2L3	4-ISA 150x150x10 2- PL 575x10	-2451.5	0.77
L3L4	4-ISA 150x150x12 2- PL 575x12	-3442.7	0.963
L4L5	4-ISA 150x150x16 2- PL 575x16	-4407.2	0.942
L5L6	4-ISA 150x150x18 2- PL 575x18	-5099.1	0.982
L6L7	4-ISA 150x150x20 2- PL 575x20	-5627.1	0.951
L7L8	4-ISA 150x150x18 2- PL 575x18	-6014.9	0.986
L8L9	4-ISA 200x200x18 2- PL 575x18	-6228.2	1.00
U0U1	4-ISA 150x150x10 2- PL 575x10	365.2	0.261
U1U2	4-ISA 150x150x10 2- PL 575x10	840.4	0.423
U2U3	4-ISA 150x150x10 2- PL 575x10	2445.1	0.901
U3U4	4-ISA 150x150x12 2- PL 575x14	3517.9	0.938
U4U5	4-ISA 150x150x16 2- PL 575x16	4387.1	0.924
U5U6	4-ISA 150x150x18 2- PL 575x18	5079.2	0.944
U6U7	4-ISA 150x150x20	5625.2	0.95

	2- PL 575x20		
U7U8	4-ISA 200x200x18 2- PL 575x18	6009.1	0.962
U8U9	4-ISA 200x200x18 2- PL 575x18	6227.9	1.00
M0L0	2-ISMC 350 2- PL 370x25	1520.7	0.565
M1L1	2-ISMC 250 2- PL 270x20	447.9	0.385
M2L2	2-ISMC 350 2- PL 370x14	740.7	0.429
M3L3	2-ISMC 350 2- PL 370x16	1022.3	0.551
M4L4	2-ISMC 350 2- PL 370x10	886.2	0.571
M5L5	2-ISMC 350 2- PL 370x10	729.5	0.481
M6L6	2-ISMC 250 2- PL 270x14	593.4	0.526
M7L7	2-ISMC 250	415.7	0.539
M8L8	2-ISMC 200	291.4	0.522
M9L9	2-ISMC 200	145.2	0.263
M0U0	2-ISMC 350 2- PL 370x25	582.0	0.314
M1U1	2-ISMC 250 2- PL 270x20	-229.7	0.307
M2U2	2-ISMC 350 2- PL 370x14	-1408.2	0.594
M3U3	2-ISMC 350 2- PL 370x16	-840.0	0.375
M4U4	2-ISMC 350 2- PL 370x10	-664.9	0.397
M5U5	2-ISMC 350 2- PL 370x10	-507.3	0.313
M6U6	2-ISMC 250 2- PL 270x14	-353.4	0.263
M7U7	2-ISMC 250	-199.7	0.3
M8U8	2-ISMC 200	180.7	0.436
M9U9	2-ISMC 200	140.1	0.368
L0M1	4-ISA 150x150x16 2- PL 575x16	2004.7	0.451
M0L1	2-ISMC 350	-687.6	0.661
M1U2	4-ISA 150x150x16 2- PL 575x16	2408.6	0.567
M2U3	2-ISMC 350 2- PL 370x25	1493.3	0.597
M3U4	2-ISMC 350 2- PL 370x16	1285.0	0.589
M4U5	2-ISMC 350 2- PL 370x10	1064.6	0.573
M5U6	2-ISMC 300 2- PL 320x10	841.6	0.612
M6U7	2-ISMC 250 2- PL 270x14	633.9	0.575
M7U8	2-ISMC 250	409.1	0.586
M8U9	2-ISMC 200	304.4	0.658

	2- PL 220x10		
M0U1	2-ISMC 350 2- PL 370x8	714.9	0.519
U0M1	2-ISMC 350	-536.0	0.492
M1L2	2-ISMC 350	-1003.2	0.701
M2L3	2-ISMC 350 2- PL 370x10	-1488.6	0.653
M3L4	2-ISMC 350	-1268.6	0.904
M4L5	2-ISMC 350	-1054.2	0.743
M5L6	2-ISMC 300	-831.0	0.697
M6L7	2-ISMC 250	-630.4	0.661
M7L8	2-ISMC 250	-410.4	0.438
M8L9	2-ISMC 200	-214.5	0.387

Maximum deflection of the bridge at limit state of strength = 206.2 mm

Steel off take as per STAAD result = 3407.9 kN

4.7.7 Reserve strength at plastic collapse

Reserve strength at plastic collapse is calculated as per equations given in section Details of the composite bridge model for the plastic stage are given in Figure 4.13.

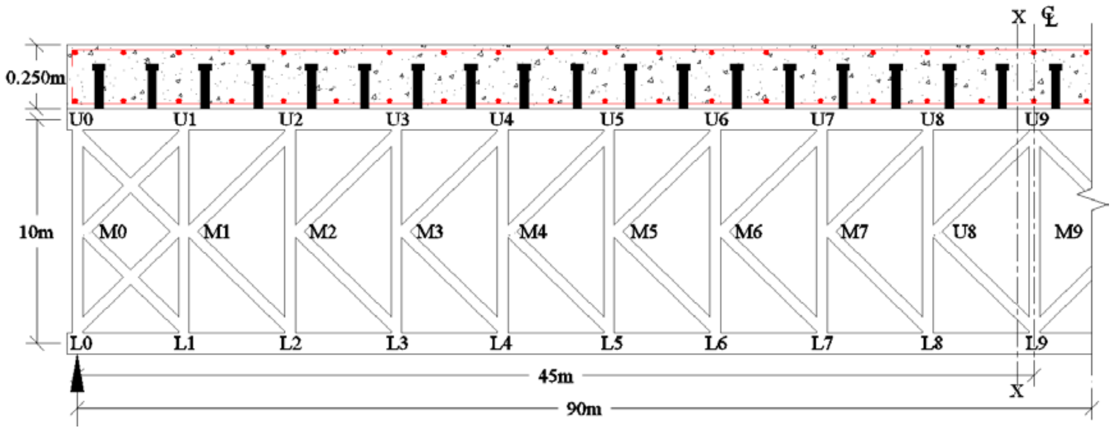
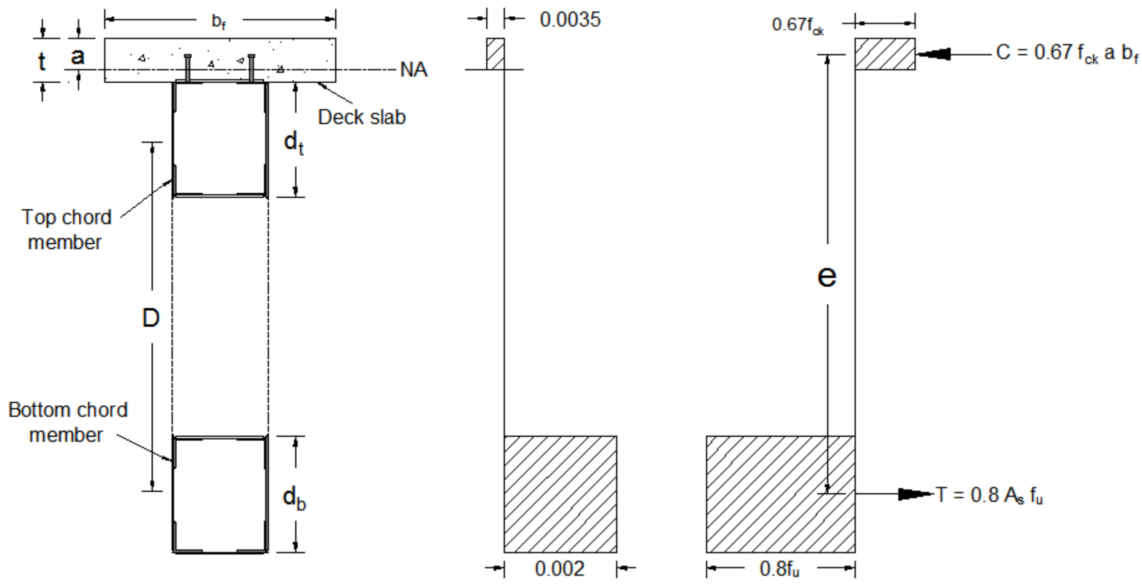


Figure 4.13 Composite truss bridge (90m span) model for plastic failure

The bridge for plastic analysis is considered at section X-X (Figure 4.9), which is adjacent to the centre line. Thus, at the section X-X only bottom chord member and composite top chord carry axial forces.

Force equilibrium diagram at the mid span section of the bridge is given in Figure 4.14.



a. Cross section at x-x　　b. Strain diagram　　c. Stress diagram

Figure 4.14 Force equilibrium at plastic collapse

Where,

a	= depth of neutral axis from top of the deck slab
A_{sb}	= cross-sectional area of steel bottom chord (=48224 mm^2)
A_{st}	= cross-sectional area of steel top chord (=48224 mm^2)
D	= Center to center distance between top and bottom chord members
d_t	= depth of top chord member (=600mm)
d_b	= depth of bottom chord member (=600mm)
t	= thickness of deck slab (=250mm)
e	= lever arm
b_f	= effective width of slab (=3450mm)
f_{ck}	= characteristic strength of concrete (=40N/mm^2)
f_u	= specified ultimate strength of steel (=410N/mm^2)

Equating compression and tension forces; $0.67ab_f f_{ck} = 0.8A_s f_u$

or 0.67x40 x a x 3450 = 0.8x48224 x 410

Therefore, a = 171.07 mm

The plastic moment of resistance (M_p) of the composite section is computed as,

$$M_p = A_{sb} \times 0.8 \times f_u \times e$$

$$= A_{sb} \times 0.8 \times f_u \times (D+d_t/2+t -a/2)$$

$$= 0.048224 \times 0.8 \times 410 \times 10^9 \times 10.46$$

$$= 1,65,450.0 \text{ kNm}$$

For the complete bridge, the total plastic moment of resistance = 2x1,65,450.0

= 3,30,901.0 kNm

Corresponding equivalent udl on the bridge 'w' is given by; $\frac{wl^2}{8} = 3,30,901.0$

or w = 326.8 kN/m

Total applied load in service condition, (DL+LL) = 12132.0 kN

Equivalent udl = 12132.0 /90.0 = 134.8 kN/m

Therefore, factor of safety at plastic collapse with respect to service load

= 326.8/134.8 = 2.4

Factor of safety at plastic collapse with respect to limit state of strength load

= 326.8/202.2 = 1.6

Thus, in composite steel truss bridge, there is a factor of safety against plastic collapse of 2.4 in comparison to service condition, and 1.6 in comparison to the limit state of strength condition.

4.7.8 Deflection and steel off take comparison

Deflection comparison for the non-composite and composite bridges in service condition and at the limit state of strength are given in Table 4.4

Table 4.4 Comparison of deflection

Load Case	Non-composite or composite truss (DL+LL)	Composite Truss Bridge 2.25x(DL+LL)	Non-Composite Truss Bridge 2.25x(DL+LL)
Maximum deflection under DL (SW+Deck Slab)	84.0	124.2	113.5
Maximum deflection under (SIDL+LL+FPLL)	59.7	61.2	69.45
Total deflection	143.7	185.4	183.0
Permissible deflection	150.0 mm	150 mm	150 mm

Steel off take for the non-composite and composite bridges are given in Table 4.5.

Table 4.5 Comparison of steel off take

	Non composite truss in service condition for (DL+LL) case	Composite Truss Bridge at limit state of strength for 2.25x(DL+LL) case	Non-Composite Truss Bridge at limit state of strength for 2.25x(DL+LL) case
Steel off take	3.53 t/m	3.86 t/m	4.28 t/m
Total steel for 90m span (t)	317.7	347.4	385.2

From Table 4.5 it is seen that total steel required for non composite truss bridge designed for service condition under (DL+LL) case is 317.7 t. Total steel required for non composite bridge designed for the overload condition of 2.25x(DL+LL) case is 347.4 t. Total steel required for composite truss bridge designed for the overload condition of 2.25x(DL+LL) case is 385.2 t.

In case of the composite truss bridge designed for the overload condition of 2.25x(DL+LL) case, total steel required (385.2 t) is higher by 21.2% in comparison to the total steel required (317.7 t) for non composite bridge designed in service condition for (DL+LL) case, where as its load carrying capacity is higher by 50.0%.

4.8 CONCLUDING REMARKS

In simply supported composite truss bridges, the top chord members are under compression. When deck slab is made composite with the top chord members, it provides effective lateral restraint to these members, and in the overload condition permissible stress in the top chord members can be assumed to be up to the ultimate stress. Thus, deck type simply supported composite steel truss bridges have very good advantage at overload condition due to the composite action.

Due to shrinkage of the deck slab concrete, cross sectional area of the deck slab concrete does not directly provide strength to the bridge until it is overcome by the flexural stresses due to various loadings on the bridge. Reinforcing steel in the deck slab concrete, however, takes the flexural stresses even when the deck slab concrete is cracked, which is neglected being on the safer side.

The design of the truss bridge at service condition and at limit state of strength condition gives approximately same design results as per IRC: 24-2010. Therefore, in the present study higher load factor of 2.25 at the overload condition in place of 1.5 without fatigue factor is recommended for design of laterally unsupported compression members which gives higher safety factor at plastic collapse stage.

Behavior and cost of simply supported 90.0m span non-composite and composite steel truss bridges have been compared. In case of the composite truss bridge designed at limit state of strength for overload condition of 2.25x(DL+LL) case, total steel required is higher by 21.2% in comparison to the total steel required for non composite bridge designed in service condition for (DL+LL) case, where as its load carrying capacity is higher by 50.0%.

There is a factor of safety of 2.4 in comparison to load at service condition, and 1.6 in comparison to the load at overload condition at plastic collapse for the 90m span composite truss bridge.