CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

Composite steel and concrete trusses are widely used both in buildings as primary or secondary beams. In the past composite open web steel trusses are also used in bridges, but no code provisions for this type of bridge exist in the World. Behaviour of steel trusses in buildings acting compositely with concrete slab has been investigated since the sixties, both experimentally and theoretically.

In this chapter, history of failure of bridges and its causes is reviewed chronologically. Development in the steel concrete composite truss technology and use of shear connectors in it is also reviewed. Use of high tensile steel and structural steel as a construction material and their merits and demerits are surveyed. The existing literature related to steel concrete composite truss bridge is categorized in the four subheads.

i. Brief history of failure of bridges.

ii. Composite truss technology for buildings and bridges.

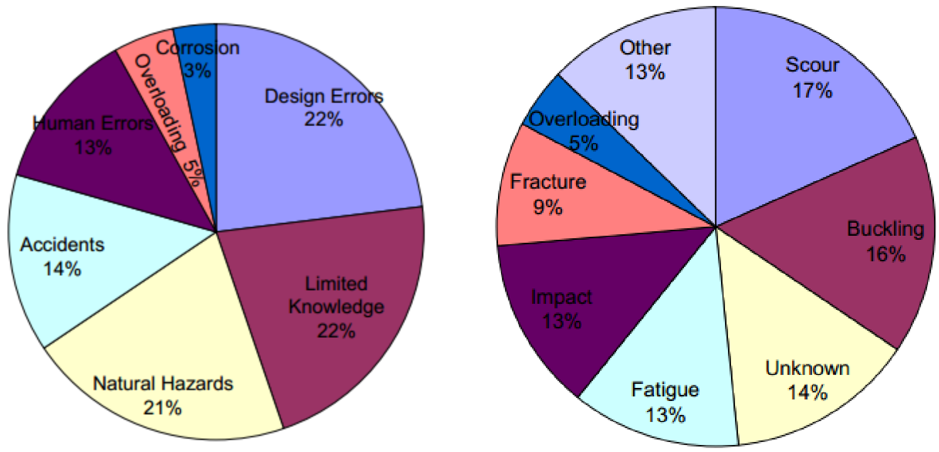
iii. Shear connectors in composite truss bridge.

iv. Use of High Tensile Steel (HTS) in composite truss bridges.

2.2 BRIEF HISTORY OF FAILURE OF BRIDGES

In the past a number of steel truss bridges have failed during various stages of construction or service. The failures have been partial, or total collapses have taken place. The most common causes of bridge failure include: overstress of structural elements due to section loss, design defects and deficiencies, long-term fatigue and fracture, failures during construction, accidental impacts from ships, trains and aberrant vehicles, fire damage, earthquakes, lack of inspection and unforeseen events. In case of truss bridges, failure of gusset plates connecting members of truss, and buckling failure of compression members are the most happening failures.

Imam and Chryssanthopoulos (2010) reviewed case studies of 164 failed metallic bridges. Of these 53% were highway bridges, 34% were railway bridges and a small percentage comprised foot bridges. Of the 164 reported cases, 87 bridges were classified as collapse, 73 bridges as ‘no collapse’ and 4 bridges as failure unknown. For the purposes of this study collapse was defined as one or more structural elements falling down from the bridge as a result of the failure rendering the structure incapable of remaining in service. The majority of the 87 cases of metallic bridge collapses occurred in the USA (36%) and UK (20%), partly due to the large number of such bridges in these countries. The distribution of failure causes for the collapse database was shown as reproduced in Figure 2.1(a). The most important factors (almost equally) contributing to collapse were design errors (22%), limited knowledge (22%) and natural hazards (21%)**.** The distribution of failure modes for the 87 cases of bridge collapses were shown as reproduced in Figure 2.1(b). It can be seen that for metallic bridges the most frequently encountered modes are scour of piers/foundations (17%), buckling (16%), fatigue (13%), impact (13%) and fracture (9%). Fatigue and fracture taken in combination appeared to be the most critical failure mode, closely followed by buckling.



a. Causes of failure b. Modes of failure

Figure 2.1 Failure causes and modes of failure leading to collapse (B. M. Imam, 2010)

According to listing of USA bridge failures 31 bridges, on average one bridge every 8 months have failed during 1980 to 2009, and from 2010 to 2012, 27 bridges are reported to have failed with an average of one bridge every month (Construction Risk Management, 2012).

In 1881, steel truss bridge near Miramont in France completely failed due to buckling of compression members. In 1883, truss bridge over Töss river in Switzerland failed due to buckling of compression members leading to death of one person and injury to five others. In 1891, truss bridge near Bergbucke in Austria failed due to buckling and lack of lateral support. In 1892, the Semi-parabolic truss arch bridge near Ljubičevo over river Morava in Serbia, failed during load testing. The cause of failure was buckling of compression chord due to defective connection of two part compression members (Z. Šavor, *et. al.*, 2011). In 1907, Quebec truss bridge over river St. Lawrence in Canada failed due to buckling of bottom compression members during cantilevered construction phase leading to death of 74 people. In many cases failure during construction and service is due to unexpected increased load on the bridge which many times might be beyond the scope of structural designer's knowledge.

Possibility of sudden collapse of truss bridges has always been due to buckling of non redundant critical compression members. Unlike compression members, tension members do not usually fail suddenly since they experience noticeable elongation and can take stress up to ultimate stress beyond the yield stress. Behavior of tension and compression members under loading is completely different and therefore, compression members need a different approach for their design.

2.3 DEVELOPMENTS COMPOSITE TRUSS TECHNOLOGY

Experimental investigations by Tide and Galahbos (1968) and Azmi (1972) together with practical applications by Iyengar and Zils (1973) pointed to new opportunities in structural design and enabled the use of composite trusses as large span floor beams. Extensive research was performed in Canada and USA by Brattland and Kennedy (1992) resulted in wide use of steel and concrete composite trusses in North America.

Lembeck (1965) performed five tests on conventional and composite open-web steel joists for buildings with 6.1m span. Composite interaction of the composite joists resulted from the projection of the web members into the concrete slab above the top chord. Lembeck concluded that composite joists were stronger and stiffer than non-composite joists with the same tension chord and web system, and as well, that the size of the steel top chord of a composite joist could be considerably reduced.

After four tests on composite and non-composite open-web steel joists, Wang and Kaley (1967) also reported that member stresses and deflections of composite joists were appreciably lower than those of non-composite members. Their tests were conducted in the elastic range.

Tide and Galambos (1968) tested five 4.9m span composite open-web steel joists both cold formed and hot rolled Steel chords. Most of these specimens failed due to the provision of an inadequate number of stud shear connectors for full composite action. Tide and Galambos observed that web members of a composite joist carry most of the vertical shear in the member. Interface slips measured at the connector locations were larger near the ends of the span, indicating, as would be expected, that connectors near the reactions carry larger shear forces and that slip accumulates towards the ends.

Cran (1972) made recommendations for the ultimate strength of composite open web steel joists for buildings, based on the results of full-scale tests on three different joists with spans of 12.2m, 15.2m and 6.1m. All exhibited very ductile behavior, although only two long span joists were tested to failure. In both these cases, failure was initiated by buckling of web member, which Cran attributed to secondary stresses due to the large deflections. Based on the observation that top chord compressive strains diminished to zero or even became tensile as load was applied to the test specimens, Cran concluded that contribution of the top steel chord of a composite open-web steel joist could be neglected for strength calculations. For design purposes, the concrete slab acts as the top chord of the truss. Cran also recommended that the calculated elastic deflections of a composite open-web steel joist be increased by 10% to account for shear deflection, with further increase of 10 to 20% to account for the effects of interface slip, especially for slabs on ribbed deck

Kravanja and Silih (*2003)* presented the economical comparison between composite welded I beams and composite trusses designed from hollow sections. The comparison was made for simply supported beams for different spans and different loads. Composite I beams and composite trusses were designed in accordance with Eurocode 4 for the conditions of both ultimate and serviceability limit states. The aim of the comparison was to find out the spans and loads, at which each of the different structures shows its advantages. In order to carry out a precise comparison between the two different composite systems, they applied a structural optimization method rather than classical structural analysis. The optimization was performed by the nonlinear programming approach. The economic objective function of self-manufacturing costs was defined for the optimization. The comparison showed that composite I beams were economically appropriate at higher values of variable imposed loads, while composite trusses were viable at lower imposed loads.

Use of High Strength Steel for design of double composite truss bridge was discussed by Shim, *et. al.*, (2011). High performance steel for bridges (HSB) which has higher performance in tensile, yield strength, toughness, weldability than common steels has been developed in Korea. HSB800 has a minimum tensile strength of 800MPa. Design of long span steel bridges with high strength steels is limited because of buckling and fatigue. Two concepts of composite and hybrid were utilized to solve the obstacles. Combination of steel box girders and double-composite truss girders along the length of the bridges was also utilized to enable the design of longer span bridges. New continuous bridges with more than 100m span length were designed using the proposed concept and HSB. Effectiveness of each combination was discussed to suggest recommendations for the design of composite bridges with high performance steels.

Bouchair, *et. al.,* (2012) discussed about connection in steel-concrete composite truss. The composite steel-concrete construction is one of the most economical systems for building and bridge floors, especially for greater spans. In their study the influence of the degree of connection, represented by the connector diameter, the influence of the top chord section and the material characteristics of steel and concrete, were analyzed considering the stiffness and the resistance of the beams and the shear forces in the connectors. The parametric studies showed that the top chord members had no significant effect on the flexural stiffness.

Xue, *et. al.*, (2011) presented a model test and numerical finite element analysis (FEA) on the mechanical behaviour of a composite joint in a truss cable-stayed bridge. The model test with the scale of 1:2.5 for the truss joint was conducted to fully understand the safety and serviceability. In the experiment, stress distribution, crack resistance ability and shear resistance of headed studs were measured to investigate the mechanical performance and force transmission of the joint part. The maximum strain of the steel plate and concrete chord remained in the linear elastic region until 1.7 times the design load, which means there is a significant safety margin for such composites. On the basis of the experimental results of composite truss joints, three-dimensional finite element models were established. The results of the finite element analysis were in good agreement with those of the tests in terms of strength and stiffness. It was also expected that the results presented in their paper would be useful as references for the further research and the design of composite truss bridges and composite joints.

2.4 COMPOSITE TRUSS BRIDGE

Trusses are efficient structural systems, since the members experience essentially axial forces and hence the materials are fully utilized (R. P. Johnson, *et. al.,* 2001). Steel as a structural material is equally strong; both in tension and compression, and the truss members are fully stressed, hence steel trusses are more efficient. They tend to be economical to support loads over larger span lengths. However, the members in the compression chord of the simply supported steel truss (top chord) may prematurely buckle before the stresses reach the material strength. In this context composite action of the RCC slab with the truss compression chord becomes useful and prevents its buckling. A reinforced concrete or composite deck floor is required in bridges to provide a flat surface. Using it as a part of the compression member in truss system is an economical proposition. Concrete has a lower strength compared with steel and hence requires larger cross section to sustain a given compression. Consequently, the concrete floor slab used as a part of the compression chord of the truss is not vulnerable to buckling failure. Further, concrete can more economically carry compression. Thus, in a composite truss system the relative merits of steel and concrete as construction materials are fully exploited. It is one of the most economical systems in longer span bridge construction. Composite truss systems are structurally efficient and economical. Considering functional and structural efficiency and economy, it is only natural that composite steel-concrete trusses are a popular choice for medium span bridges. Shear transfer between the steel truss and concrete deck slab is mobilised usually using shear studs.

2.4.1 Types and examples of composite steel truss bridges

Following types of composite steel truss bridges are constructed in the past.

i. Simply supported composite steel truss bridge.
ii. Single deck continuous composite steel truss bridge.
iii. Double deck continuous composite steel truss bridge.
iv. Composite steel truss bridge with prestressed deck
v. Hybrid composite steel truss bridge.

i. Simply supported composite steel truss bridge

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. However, not much literature is available for composite steel truss bridges. In India construction of composite steel truss bridges is not common, and hence research in this area is required to avail benefits of the composite construction in case of bridges.

Steel truss bridges are commonly constructed throughout the World in which single span simply supported bridges are the simplest and easiest to design and construct. In India generally through type truss bridges are constructed and deck slab rests over the cross girder and stringer system. Generally shear studs are not provided to make the cross girder and stringer system composite with the deck slab. As the bottom chord of the trusses are in tension in simply supported spans, composite decks with the bottom chord members is of little help. In the case of deck type simply supported truss bridges, top chord members are under compression and RCC deck slab, if made composite with the top chord members, prevents buckling of the top chord compression members and adds in cross sectional area of the compression members. In order to make the RCC deck slab composite with the top chord members, shear studs between the top flange of the top chord members and RCC deck are required. Thus, benefits of composite construction can be easily obtained in case of composite deck type truss bridges.

In the Czech Republic twelve simply supported composite truss bridges with spans between 21m to 63 m were completed during last decade and similarly in other countries this type of truss bridges have been constructed (Figure 2.2) (J. Machacek, *et. al.,* 2011).

Figure 2.2 Simply supported composite truss bridge of span 36m
(J. Machacek, *et. al.,* 2011)

In the deck type composite bridge due to shrinkage strain in the deck slab, composite action between the steel truss and the deck slab starts only when shrinkage strain is overcome by the flexural strain in the deck slab due to part live load. Thus, advantage of the composite section in terms of increased cross sectional area is derived only at the late stage of live load. Therefore, the steel truss may be designed for full dead load plus live load condition, and advantage of the composite section may be derived in the limit state of strength condition under 1.5x(DL+LL) case. As a result, under limit state of strength condition, sections of the laterally supported top chord compression members need not be increased from the service condition requirement.

ii. Single top deck continuous composite steel truss bridge.

Continuous steel truss bridges can be constructed with the advantage of approximately 1/3rd mid span sagging moment, and 2/3rd support hogging moment, of the total simply supported span moment. This type of construction was adopted in case of Chauras and Garudchatti bridges. Three span continuous geometry was adopted in these cases, where side to main ratio was kept as 0.364. In the Chauras bridge uplifting of the side span during deck slab casting caused its failure as described in Chapter - 3.

RCC top deck can be made composite with the continuous span steel truss. Due hogging moments at the supports, the deck slab in this case will be under tension and its contribution to bridge cross section at the support section will be limited to the reinforcement in the deck slab, and its structural contribution will be little.

An example of this type of bridge is the Lully viaduct composite bridge with steel tube truss (Figure 2.3) located near village Lully in the Canton of Fribourg in Switzerland . It is a single deck continuous composite truss bridge with a typical span of 42.75m (H. G. Dauner, *et. al.,* 1998).

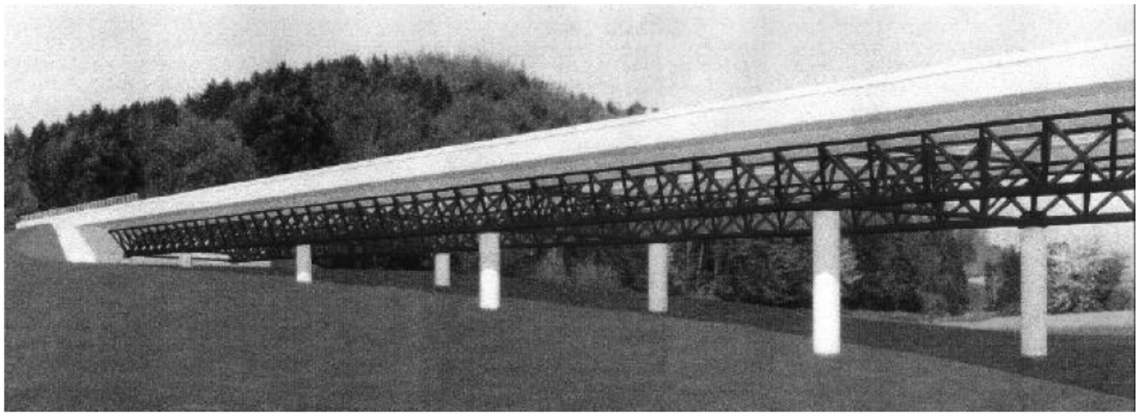
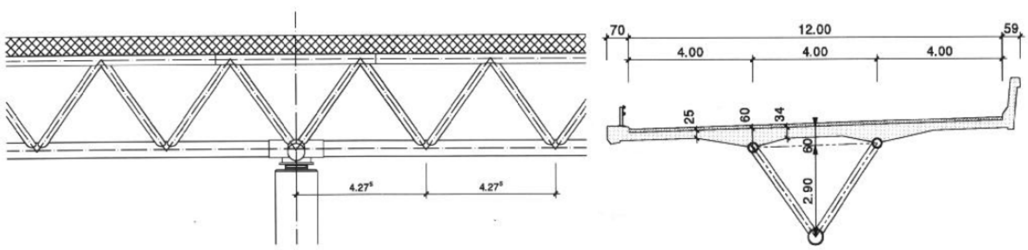


Figure 2.3 General view of the Lully Viaduct Composite Bridge
(H. G. Dauner, *et. al.,* 1998)

The bridge consist two triangular trusses fabricated entirely from unstiffened circular tubes and connected together with the help of tubular brace truss at each support location (Figure 2.4). Each transversal triangular cross-section is 2.9 m high and 4.0 m wide, and is supported by a single slender pier. The largest diameters and thickness of the tubes are over 500 mm and nearly 70 mm respectively.



(a) Elevation (b) Cross-section

Figure 2.4 Longitudinal view and standard cross-section (H. G. Dauner, *et. al.,* 1998)

The bridge was made composite with upper chord members with the help of welded shear connectors as shown in Figure 2.5.

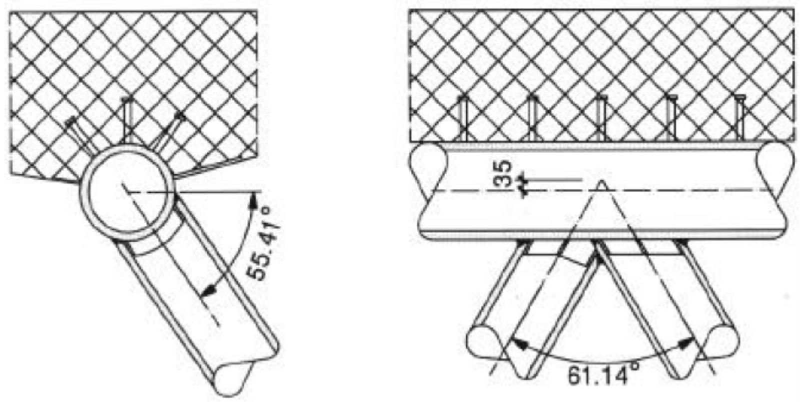


Figure 2.5 K-shaped joint geometry with shear studs (H. G. Dauner, *et. al.,* 1998)

Another example of this type of bridge is under construction Bogibeel bridge (Figure 2.6), which is a combined road and rail bridge in the Dibrugarh district of 4.94 kilometres length in Assam.

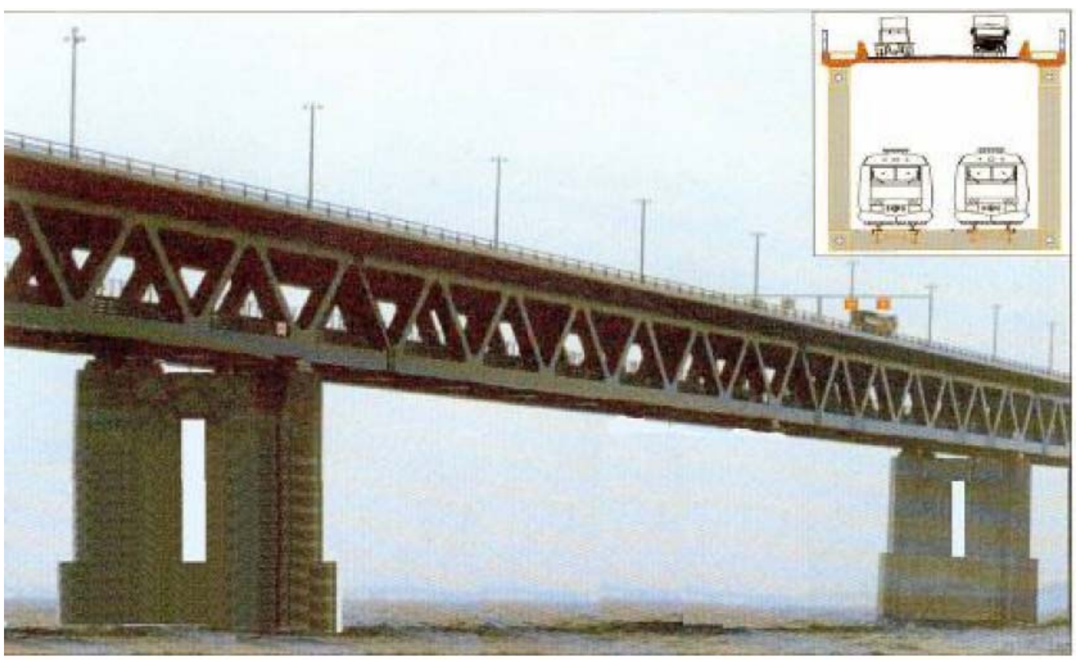


Figure 2.6 Proposed bridge at Bogibeel with 125m span (A. K. Goyal, 2013)

This is the first composite truss bridge in under construction in India (A. K. Goyal). The Bogibeel rail-road bridge is a double-deck bridge with a two-line railway track on the lower deck and a three-lane road on the upper. The road level will be 10.5m above the railway line. The superstructure consists of steel trusses of 125m span and 32.6m span, for double track on bottom level and three lane road deck at top level. The warren type truss of 125m span consists of 10 panels of 12.16m each with centre to centre distance of bearing as 121.6m. The members shall consist of welded plates of either 'I' or 'Box'-sections. The Railway deck shall consist of cross girders at a spacing of 12.16m at the bottom chord joints of truss. The stringers and cross girders are of welded type with welded connections for lateral and cross bracing members. The roadway deck consists of composite top chord and cross girders. The roadway cross girders are at the same spacing of 3.04m. Two spans of 32.6m, one each at shores shall be required to be fabricated and launched along with 39 spans of 125m each.

In the continuous composite top deck bridge, due to tensile shrinkage strain, and flexural strain under live load condition, cracking of the deck slab at the support sections is unavoidable. This leads to ingress of water causing corrosion of the rebar, and accelerated fatigue deterioration.

iii. Double deck continuous composite truss bridge

In case of continuous bridges bottom chord members at the supports are in compression due to approximately two times hogging moment in comparison to span moment, and top chord members at mid span are in compression. In order to prevent buckling of the top chord and bottom chord compression members and to increase cross sectional areas of the truss, RCC deck slabs are helpful, both at top as well as at bottom of the truss.

An example of this type of double deck continuous composite truss bridge is the Ulla river viaduct (Figure 2.7) over the river Ulla in Spain. It is a steel lattice with double steel and concrete composite work at hogging zone. It consist three main spans of 225m+240m+225m long and several approaching spans measuring 120m long. The four central piers are rigidly connected to the lattice deck (F. Millanes, 2010).

Figure 2.7 View of the Ulla river viaduct (F. Millanes, 2010)

Upper chord members are made composite with the upper deck slab suing shear connectors near the mid span, and lower chord members are made composite with the lower level slab at the supports with the help of shear connectors.

Cracking of the deck slab near the supports takes place due to shrinkage and tensile flexural strain, which leads to accelerated deterioration of the deck slab as discussed in the case of single top deck continuous composite steel truss bridge. Further, live load moment at the supports is twice that of mid span moment, and therefore, variable thickness of slabs may be required.

iv. Composite steel truss bridge with prestressed deck

In case double deck system continuous composite truss bridges, upper deck at the support locations and lower deck at the mid span locations are subjected to tension. Therefore, prestressing of the deck slabs at these locations is helpful. Examples of this type of construction are Sarutagawa and Tomoegawa Bridges in Japan (Figure 2.8) (K. Ohgaki, 2011).

Figure 2.8 Sarutagawa Bridge (K. Ohgaki, 2011)

This prestressed concrete composite truss bridge consists of a web structure built with tubular steel truss members in place of a standard concrete web and has upper and lower concrete slabs made composite with steel truss (Figure 2.9).

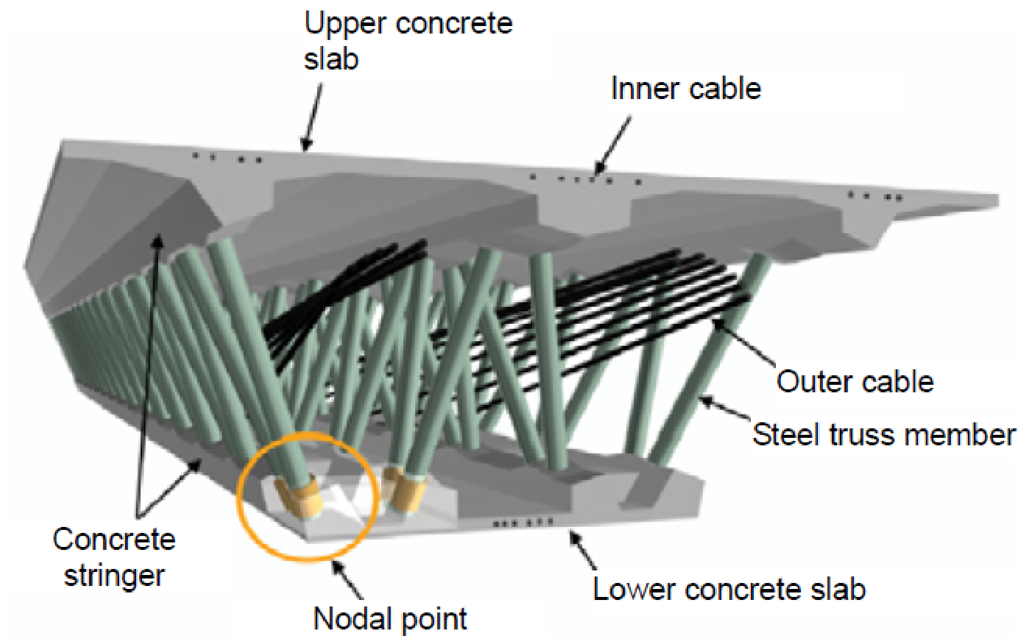


Figure 2.9 Composite construction with prestressing of upper and lower slabs (K. Ohgaki, 2011)

In addition to composite action of steel truss with upper concrete slab and lower concrete slab, prestressing of the slabs was also carried out.

Prestressing of the deck slab using cables embedded in the deck slab is not desirable (IRC: reference). Top deck slab under live load condition is in flexural tension at supports and flexural compression at the mid span, and bottom slab is under flexural compression at supports and flexural tension at mid span, therefore, uniform prestress in top and bottom slabs does not provide a suitable solution particularly to prevent cracking of the top deck slabs.

v. Hybrid composite steel truss bridge.

In the roadway bridges, there are a lot of cases that the hybrid structures are adopted, especially in the expressway, and the one that steel and concrete were combined in a structural cross section is called a composite structure, and on the other hand, the one that steel and concrete were combined in a structural member is called a mixed structures, and generally these are called hybrid structures.

Advantages and disadvantages of continuous span composite truss bridges

Continuous span steel truss bridges can be constructed with the advantage of approximately $1/3^{rd}$ mid span sagging moment, and $2/3^{rd}$ support hogging moment, of the total simply supported span moment. This type of construction was adopted in case of Chauras and Garudchatti bridges. Three span continuous geometry was adopted in these cases, where side to main ratio was kept as 0.364. In the Chauras bridge uplifting of the side span during deck slab casting caused its failure as described in Chapter - 3.

RCC top deck can be made composite with the continuous span steel truss. Due hogging moments at the supports, the deck slab in this case is under tension and its contribution to bridge cross section at the support section is limited to the reinforcement in the deck slab, and its structural contribution is little.

In the continuous composite truss bridge, due to tensile shrinkage strain, and tensile flexural strain under live load condition, cracking of the deck slab at the support sections is unavoidable. This leads to ingress of water causing corrosion of the rebar, and accelerated fatigue deterioration.

Crack control is an important issue in steel and composite continuous bridges (J. He, *et. al.,* 2010). These cracks permit the ingress of harmful substances into concrete bridge decks. With the presence of cracks in concrete bridge decks, water, sulphates, chlorides, and other potentially corrosive agents are able to permeate to the interior of the bridge deck and cause further deterioration in the form of even larger cracks, spalling, potholes and eventually a loss of cross section of the bridge deck or reinforcing steel, which ultimately leads to an unsafe bridge. The repair of concrete bridge decks is often difficult and expensive because alternate routes are sometimes difficult or impossible to come by. To prevent deterioration from starting in the first place, concrete must not be allowed to crack, especially at an early age. In addition to tensile cracks, a number of different types of shrinkage can contribute to the development of cracking and durability concerns in concrete. These include plastic, autogenous, and carbonation shrinkage.

Although, there are many advantages of continuous composite truss bridges, RCC deck slab of a continuous composite truss bridge is susceptible to cracking due to negative moment at the intermediate supports, leading to increased fatigue and ingress of water and corrosion of the rebar, which adversely affects its durability.

Despite of available methods for crack control viz. use of relatively large amount of rebar, prestressing of the deck in hogging or negative bending moment zone, cracks within the limit of acceptable widths, use of double-deck at top and bottom in continuous composite truss bridge, use of shear pockets and precast deck slab, deck flexing and thermal prestressing methods have been tried but no effective and feasible method has yet been proposed for steel–concrete composite structures (S. H. Kim, *et. al.,* 2007).

If side span length is less in comparison to the main span length, then lifting of the side span will take place as evidenced in the case of Chauras bridge (Chapter-3). Thus, small continuous side spans with long main spans is not advisable.

Therefore, simply supported medium span (30m to 100m) deck type composite truss bridges are most suitable, specially for deep valley condition in mountainous regions.

2.5 SHEAR CONNECTION IN COMPOSITE TRUSS BRIDGES

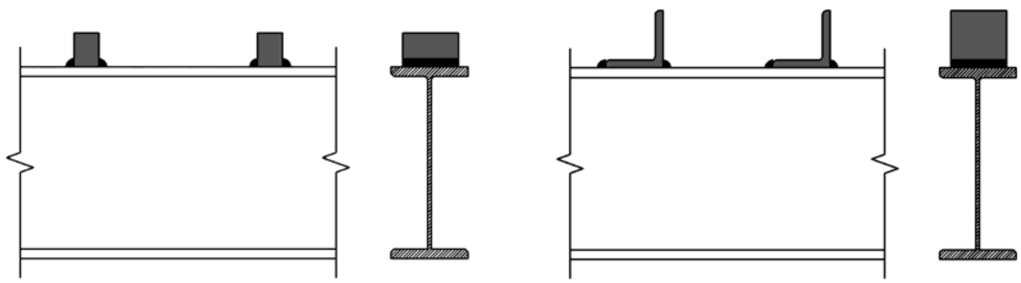
According to cl 601.2 of IRC: 22-1986, the acting together of the girder and slab as a unit is ensured by the use of mechanical device known as shear connectors.

There are three types of shear connectors as given below.

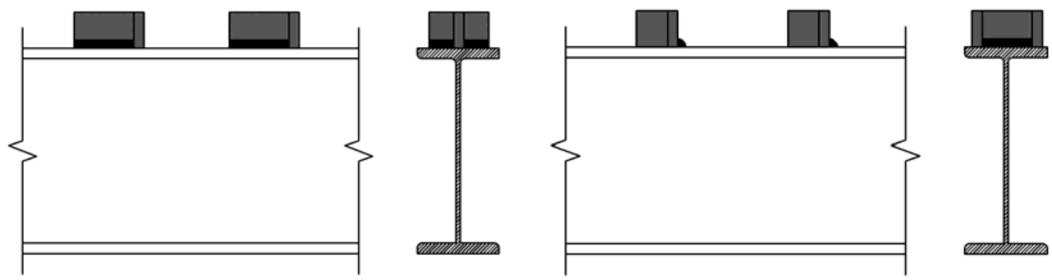
i. Rigid shear connector
ii. Flexible shear connector
iii. Anchorage shear connector

Rigid sheer connector

Rigid shear connector consists of short length bars, stiffened angles, channels or tees welded on to the flange of the steel girders and derives resistance to horizontal shear by bearing against concrete. Such connectors should be provided with anchorage devices as shown in Figure 2.10.



(a) Bar (b) Stiffened angle

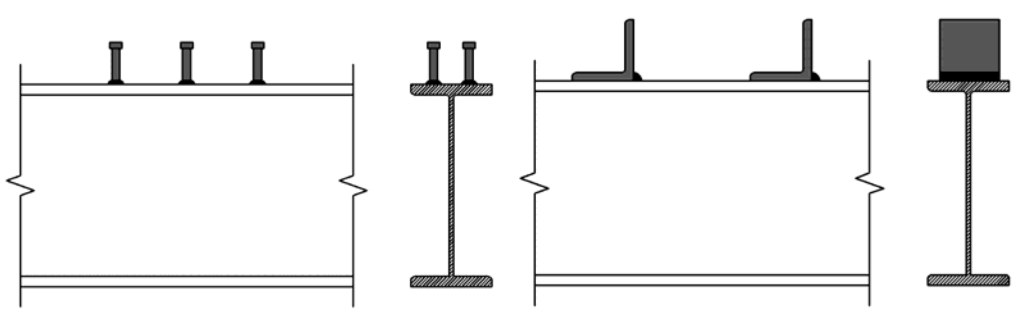


(c) Tee (d) Channel

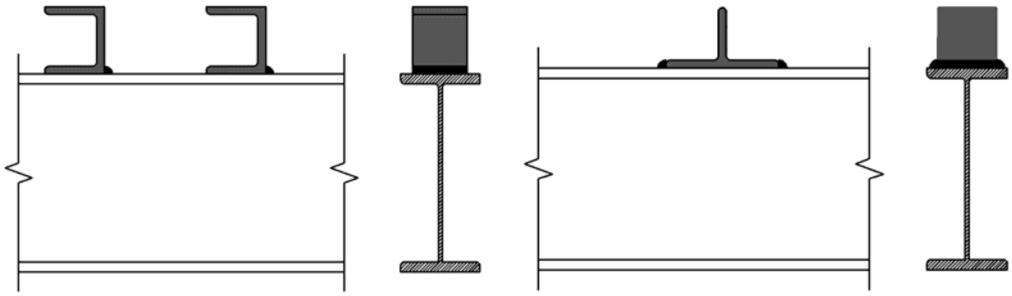
Figure 2.10 Rigid connectors

Flexible shear connector

Flexible shear connector consists of studs, channels, angles or tees welded to steel girders and derives resistance to horizontal shear through bending of the connectors (some of this type are shown in Figure 2.11). Where necessary, such shear connectors shall be provided with anchorages device.



(a) Headed stud (b) Angle



(c) Channel (d) Tee

Figure 2.11 Flexible connectors

Anchorage shear connector

Anchorage connectors (Figure 2.12) are used to resist horizontal shear and to prevent separation of the girder from the concrete slab at the interface through bond.

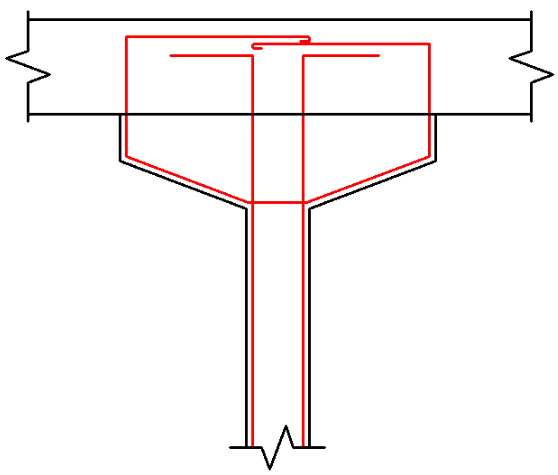


Figure 2.12 Anchorage shear connectors

Headed shear studs are most suitable for composite steel truss bridges as these facilitate reinforcement layout and concreting in the deck slab. Figure 2.13 shows headed shear connection in a plate girder bridge.

Figure 2.13 Headed shear connectors (A41 Aston Clinton Bypass)

In past extensive research has been carried out for the calculation of longitudinal shear in shear studs of composite open web steel joists for buildings. Design procedures have been evolved on the basis of these experimental results and analytical models and included in Canadian (CAN/CSA S16.1, 1997) and American codes (SJI-CJ-2010). But, the given design guidelines are limited to composite trusses used in buildings.

Machacek and Cudejko (2009) and Machacek and Charvat (2011) discussed design of composite steel and concrete truss girders with an emphasis on longitudinal connection of the steel truss and a concrete slab. They experimentally investigated the behavior of two steel and concrete composite truss girders, using perforated shear connector to reach full shear connection. Experimental results served to calibrate the non-linear 3D numerical FE model formulated using ANSYS software package. More than 30 variants of shear connections of a simple truss with Vierendeel panel at mid span have been studied, having various load slip relationships obtained from previous research. Distinctive peaks of shear flow in the connection above truss nodes have been found within elastic behavior, especially important for connectors loaded in fatigue followed by plastic redistribution in plastic region. Comparison of the numerical results with proposals given in Eurocode 4 and influence of shear connection densification above truss nodes were discussed in a detail. Finally, recommendations for practical design were presented.

Machacek and Cudejko (2011) studied longitudinal shear flow in shear connectors of composite truss bridges. Distribution of the shear flow between steel truss and concrete slab along the span of a composite truss girder was found to be highly non-linear and simplified approaches were searched for. In the nineties, the research by Neal and Johnson and SCI publication led to design recommendations, showing a wide range of design aspects important for composite steel and concrete trusses. In compliance with these recommendations, the plastic design can be done identically as for a common plate girder, including the design of a steel–concrete shear connection, provided, the shear connectors are adequately ductile and the bending rigidity of the upper steel flange of the truss is sufficient. Local effects of a concentrated longitudinal force introduced into the concrete slab of a composite continuous girder due to prestressing were investigated by Johnson and Ivanov (2001) and introduced into Eurocode 4 (ENV 1994-2, 2001) in more detail. The Eurocode proposes formulae for the local effect of a concentrated longitudinal force and distribution of the longitudinal shear force into shear flow between steel section and concrete slab, which may appropriately be used in the design of composite trusses. The analysis was based on linear behaviour of shear connection and gives largely conservative results.

2.6 CRITICAL OBSERVATIONS

1. Buckling of compression members in steel truss bridges causes sudden collapse without warning claiming life and property with it. Composite action of RCC deck slab with top chord compression members prevents their premature buckling as they get laterally restrained throughout their length with the help of shear studs.
2. Tension and compression members have different behavior under loading. Tension members can take load up to ultimate stress of material beyond yield stress but load carrying capacity of compression members is restricted due to its buckling tendency.
3. Though composite action of RCC deck with steel truss in deck type truss bridges is advantageous, no separate design guidelines are stipulated in any design standard for the design of composite truss bridges. Therefore, keeping in mind advantages of composite truss bridge, standard design guidelines for it are necessary.
4. Shrinkage strain in RCC deck do not allow composite action of deck slab with steel truss unless it is overcome by flexural stresses due to live load. Therefore, advantage of composite action is available only at overload condition or at plastic collapse.
5. HTS in composite truss bridges may prove advantageous.
6. Due hogging moments at the supports, the deck slab in the case of continuous composite truss bridges is under tension and its contribution to bridge cross section at the support section is limited to the reinforcement in the deck slab, and its structural contribution is little. Therefore, simply supported composite truss bridges are advantageous over continuous composite truss bridges.

2.7 SCOPE OF PRESENT STUDY

1. Analysis of Chauras Bridge at collapse stage and learning lessons from it for construction of truss bridges in future: Case study
2. Strengthening and load testing of Garudchatti bridge to make it safe for maximum service load: Field study
3. Recommendation of design load factor for the design of truss bridges for overload condition.
4. Recommendations for the design of composite steel truss bridges on the basis of composite action between steel truss and RCC deck.
5. Design recommendations for the efficient design of shear studs in composite truss bridge.
6. Comparative study of non composite and composite truss bridge for four different truss configurations.
7. Proposal of a new truss configuration named as semi deck type composite truss bridge.
8. Feasibility study for the use of high tensile steel (HTS) in composite truss bridges.