CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

In buildings, composite open web steel girders with RCC are widely used as structural elements. In the past composite open web steel girders are also used in bridges, but codal provisions for the design of this type of bridge are not yet comprehensive. Since the sixties, composite action of open web steel girder bridges with concrete slab has been studied, both theoretically and experimentally.

In this chapter, the history of the failure of bridges and their causes are reviewed chronologically. Development in the composite open web steel girder bridge technology and use of shear connectors is also reviewed. Use of high tensile strength steel as construction material and its merits and demerits are surveyed. The existing literature related to reasons for failure of steel bridges and composite open web steel girder bridge with RCC deck is categorized in the following three subheads.

i. A brief history of the failure of bridges.

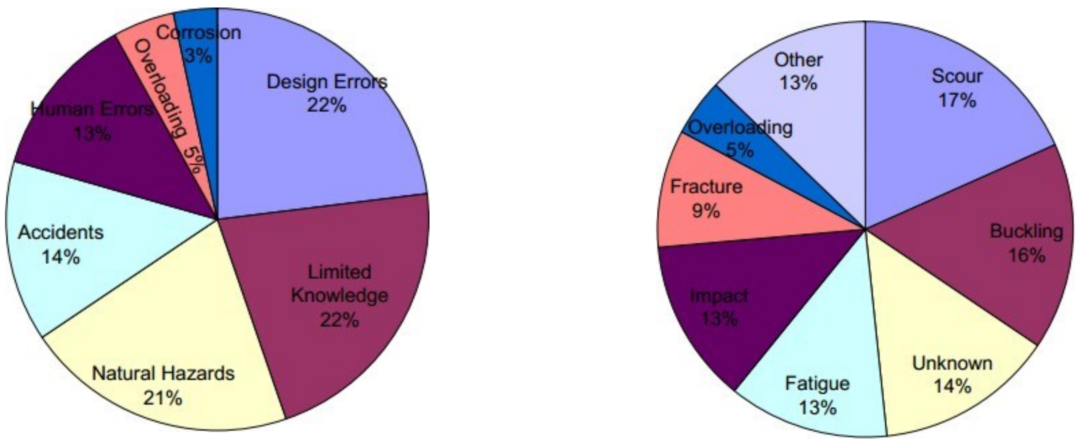
ii. Composite open web steel girder technology for buildings and bridges.

iii. Shear connectors in composite open web steel girder bridges.

2.2 BRIEF HISTORY OF FAILURE OF BRIDGES

Historically, numerous steel bridges have collapsed at various stages of construction and service. Some failures were partial while others were total collapse. Prominent causes of failure include design defects, overstressing of structural elements, failure due to faulty construction practices, fatigue and creep, lack of inspection and care, accidental impacts on bridges, earthquakes, fire damages among other unforeseen events. In the case of open web steel bridges, failure of connections and buckling of compression members are the most common causes of failure.

A review of failure case studies of 164 bridges was done by Imam and Chryssanthopoulos (2010). Out to the bridges studied, a large portion comprised highway bridges (53%), 34% of the bridges were railway bridges and the rest were footbridges. The same paper classified 87 bridges as collapsed, 73 bridges were classified as 'no collapse' and four bridges were in the 'failure unknown' category. The 'collapsed bridges' category had bridges that were rendered unserviceable due to one or more structural elements falling off the bridge. Out of the collapsed bridges, the majority of the metallic bridges were from the USA (36%) and UK (20%). This was due to the large number of metallic bridges used in the above countries. The causes of the failure of metallic bridges are shown in Figure 2.1(a). The major factors contributing to the collapse of the bridges were design errors (22%) and natural hazards (21%).



a. Causes of failure b. Modes of failure

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

The distribution of modes of failure is shown in Figure 2.1(b). The most common mode of failure was due to scour (17%) of the foundation and buckling of the compression members (16%). In case of failure of the superstructure, the most common mode was due to buckling (16%) which was closely followed by fatigue (13%).

According to the listing of 31 bridges that failed in the USA, on average one bridge failed every 8 months from 1980 to 2009 and from 2010 to 2012, 27 bridges were reported to have failed with an average of one bridge failing every month (construction risk management, 2012).

The online repository of http://www.bridgeforum.org has details of bridges which has failed from 1444 to 2009. Following are some of the examples which highlights a fundamental issue of buckling in steel under compression. In 1881, a compression member of a steel open web steel girder bridge near Miramont in France buckled causing the bridge to collapse. In 1883, also a bridge on the Toss river in Switzerland collapsed due to the buckling of compression members causing the death of one person and leaving five persons injured. An open web steel girder bridge near Bergbucke in Austria also collapsed in the year 1891. The cause of the failure was also attributed to the buckling of compression members and lack of lateral support. A semi-parabolic truss arch bridge near the city of Ljubičevo over river Morava in Serbia also failed due to buckling of connection of the compression chord (Z. Šavor, et al., 2011). In 1907, 74 people lost their life in Quebec over river St. Lawrence, when the bottom chord compression members of an under-construction cantilever bridge buckled. In many cases, the bridges failed during construction and service conditions due to an unexpected increase of load on the bridges.

Buckling of non-redundant critical compression members causes the sudden collapse of open web steel girder bridges. 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. The behaviour of tension and compression members under loading is completely different and therefore, compression members need a different approach for their design.

Analysis of bridge failures in India is done by, R. K. Garg et al. (2020). It is reported that between 1977 to 2017, more than 2130 bridges (excluding culverts and pedestrian bridges) have collapsed. Although the bridges were designed for a life span of 100 years, the average life of these failed bridges was only 34.5 years. This average life of failed bridges is higher than in China (23.6 years) but lower than USA (51.7 years). The analysis also stated that 123 bridges failed during different stages of construction pointing out deficiencies in designs and constructional practices. 86.7% of these bridges were reinforced concrete or pre-stressed concrete bridges.

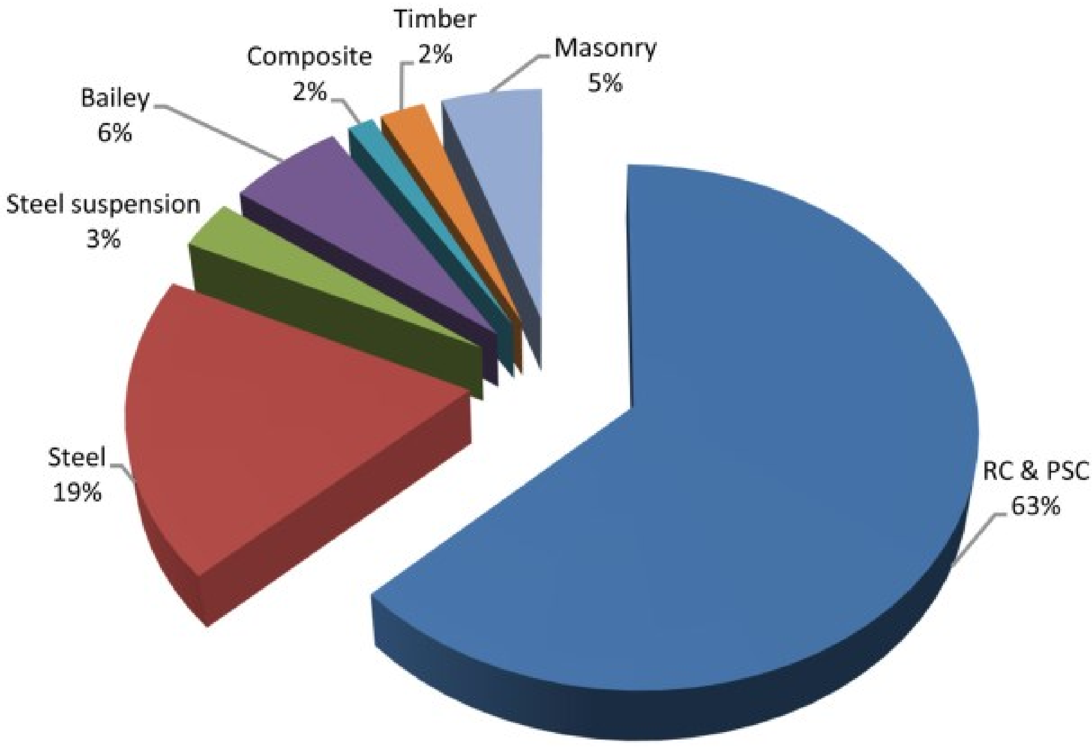


Figure 2.2 Distribution of failed bridges in India from 1977 to 2017

[R. K. Garg et al. (2020)]

Of 2130 failed bridges which they considered, 63% were RCC and PSC bridges, 19% were steel bridges, 6% were bailey bridges, 5% were masonry and 3% were steel suspension bridges as shown in Figure 2.2. It was also observed that the most common cause of bridge failure was due to natural disasters contributing to 80.3% of the collapses (Figure 2.3). Among natural disasters, 51.9% of the bridges failed due to flooding. This can be attributed to changing ecology, incorrect hydrological data and uncontrolled sand mining. 26.9 % of the bridges were also lost to earthquakes.

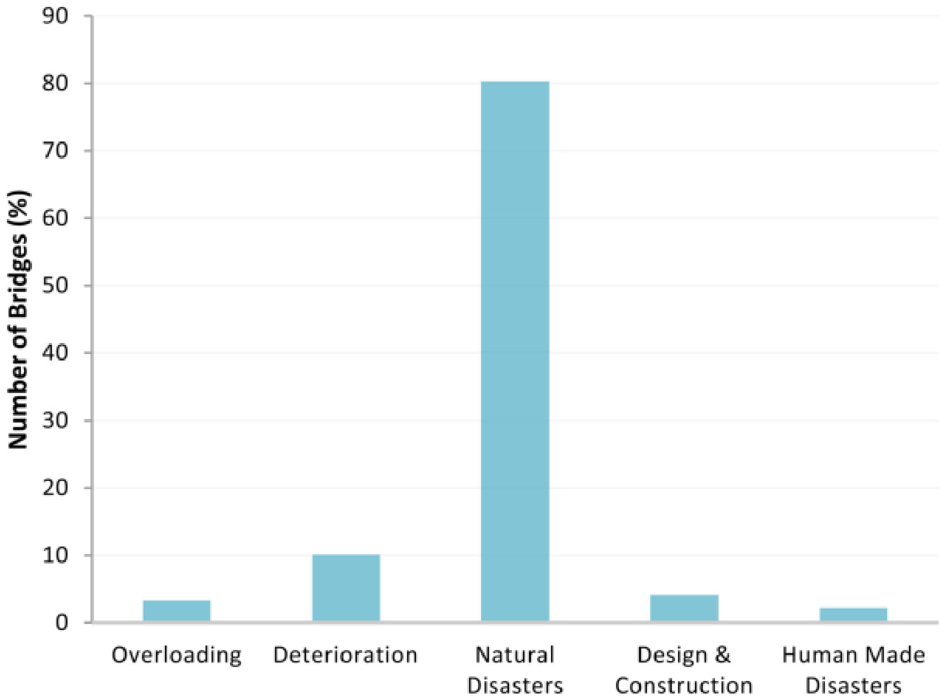


Figure 2.3 Causes of bridge failure in India. [R. K. Garg et al. (2020)]

In the study, the reason for the failure showed that 72% of the bridges were left unusable due to the failure of the superstructure and 10% due to failure of substructure among other reasons as shown in Figure 2.4.

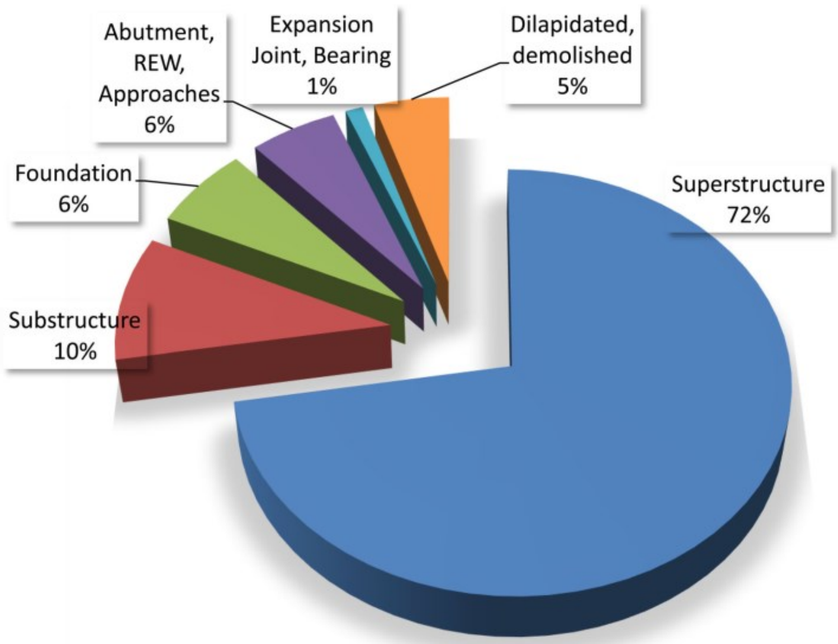


Figure 2.4 Component wise failure of bridges in India. [R. K. Garg et al. (2020)]

The cases of bridge failure have increased in the recent decade, with approximately 129 bridges failed each year during 2007-2017 (Figure 2.5). This distinct pattern of failure of bridges is different from the rest of the world. Surprisingly most of the failure of in-service bridges is between the age of 1 to 5 years (Figure 2.6).

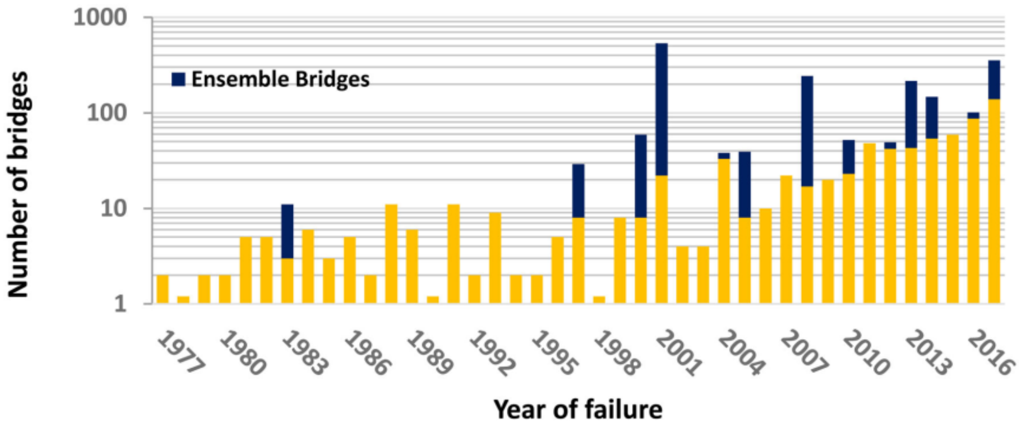


Figure 2.5 Yearly distribution of failed bridges. [R. K. Garg et al. (2020)]

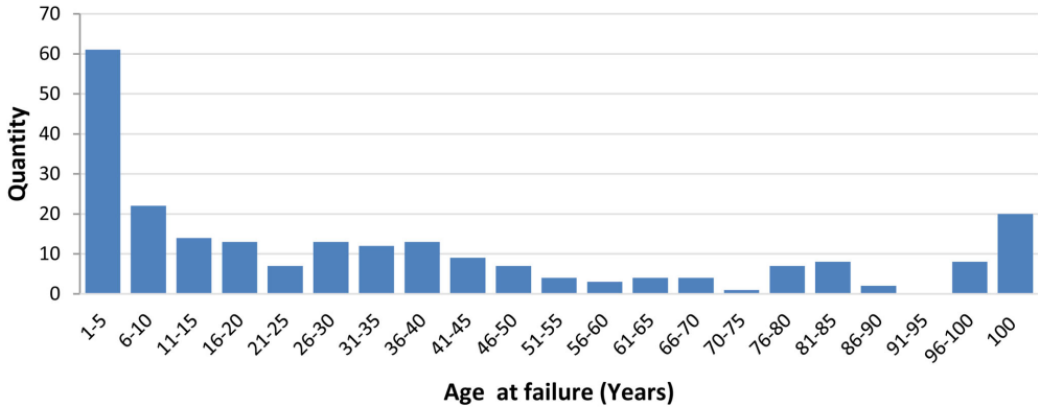


Figure 2.6 Distribution of failed bridges as per their age.

[R. K. Garg et al. (2020)]

In the case of steel bridges, buckling of the compression members was the leading cause of failure. The buckling resulted due to overloading and faulty design or construction practices.

2.3 SIGNIFICANT FAILURE OF BRIDGES DUE TO BUCKLING

In the case of open web steel girder bridges, the most common cause of failure of the superstructure is due to buckling of top chord member or due to buckling of connecting plates. Either the failure of a structure can be sudden or it can be progressive. In the case of progressive failure, judging the failure of the structure is easy and preventive measures can be taken to address the issue. Progressive failures also give us time to avoid life losses. However, in case of sudden failure, the whole structure may collapse within seconds without giving any warning. This may lead to life loss along with property loss.

2.3.1 Brief history of I-35W Bridge.

I-35W bridge was constructed over the Mississippi River in Minneapolis, Minnesota. The eight-lane bridge had two continuous trusses placed on three spans of 81m, 139m and 81m. On 1st August 2007, the bridge suddenly collapsed causing the death of 13 people and more than 100 people were injured (M. Liao, et al., 2011).

Figure 2.7 shows failed superstructure of the I-35W bridge (A. Astaneh-Asl, 2008).

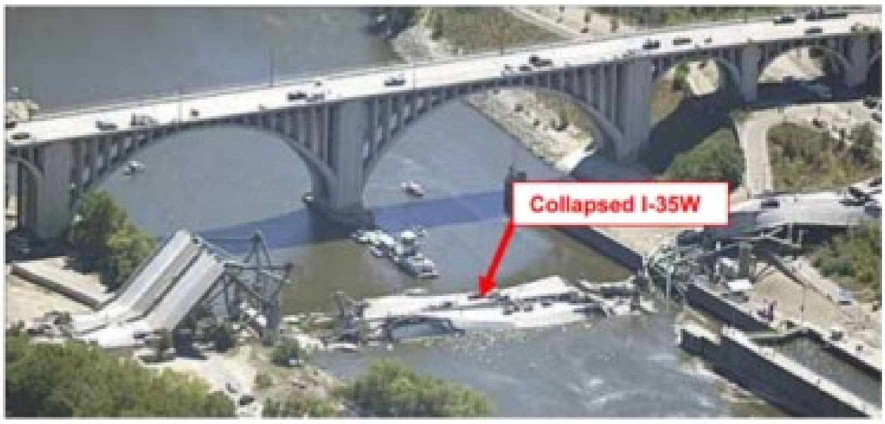


Figure 2.7 Failed superstructure of I-35W bridge [A. Astaneh-Asl, 2008]

During the service life of the bridge, a lot of modifications and repairs were done. A couple of the significant changes in the bridge structure, which may have become the cause of failure, are listed below.

- In 1977, a new deck was laid on the bridge and the thickness of the deck slab was increased from 6.5 inches to 8.5 inches.
- In 1998, concrete parapet and steel guardrails were introduced in the bridges.

At the time of the inauguration of the bridge, the weight of the deck slab contributed 70% of the total weight of the bridge. Increasing the thickness of the deck slab and concrete parapets further increased the weight of concrete by 30%. To give a perspective of the extra weight put on the bridge, the 2 inches of the deck on an 8-lane bridge is equivalent to doubling the weight of the steel used in the bridge (A. Astaneh-Asl, 2008). The members of the open web steel girder bridge were found to be adequately safe for the loads placed on the structure.

Structural analysis of the I-35W bridge was done to determine the members which failed and caused the collapse of the bridge. A 1-inch thick gusset plate was used to connect the members of the open web steel girder bridge except for the joint U10 where half-inch thick gusset plates were used (Figure 2.8). Finite element modelling and field investigations concluded that buckling of plates used at U10 joint was the cause of sudden failure of the whole structure (M. Liao. et al., 2011).

Figure 2.8 Failed gusset plate at U10 joint [M. Liao et al., 2011]

2.3.2 Failure of under-construction Chauras Bridge

A 190m long bridge was to be constructed on the Alaknanda River near Srinagar city in Uttrakhand, India. The bridge was to be constructed between the cities of Srinagar on the left bank and Chauras on the right bank. The structure of the bridge was a three-span continuous open web steel girder of 40m, 110m and 40m. The bridge failed on 24th March 2012, during the laying of concrete deck slab on the bridge (H.S.Birajdar et al., 2014) (Figure 2.9). Six lives were lost in the accident.

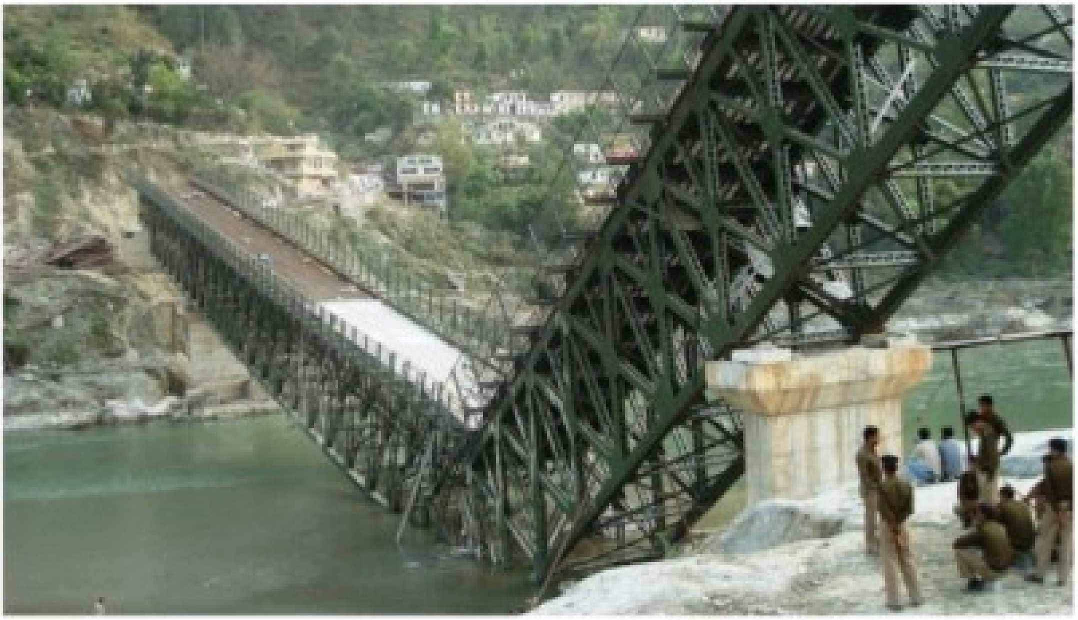


Figure 2.9 Failed Chauras bridge during concreting. [H.S.Birajdar et al., 2014]

The bridge had a carriageway width of 7.5m and a 1.5m wide footpath on either side and was designed to carry 2-lanes of traffic. Analysis of the bridge structure was carried out to identify the members where the collapse initiated. The bridge was modelled in STAAD Pro V8i software. From the analysis, it was found that at the time of the collapse, the member U13U14 carried an axial force of 6000.1 kN and making the member stresses reach 173.8 N/mm^2 as shown in Figure 2.10. Buckled view of member U13U14 that was taken place at the collapse of the Chauras bridge is shown in Figure 2.11. The overstressing of the member U13U14 is also observed in the analysis.

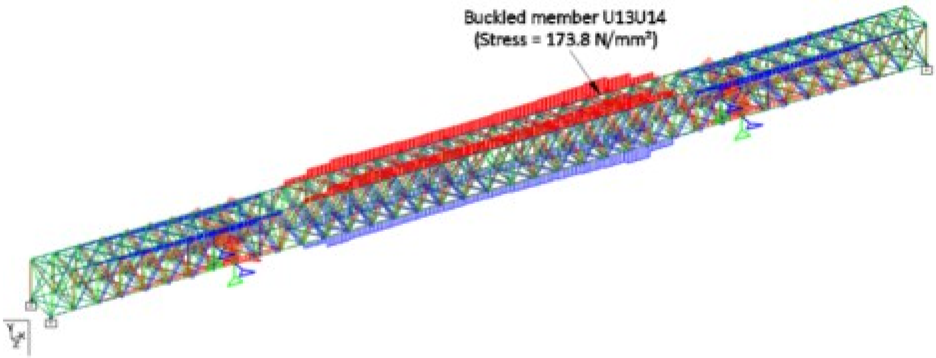


Figure 2.10 Member stresses at the time of the collapse [H.S.Birajdar et al., 2014]

Figure 2.11 Buckled member U13U14 [H.S.Birajdar et al., 2014]

The cause of the failure was a faulty sequence of the casting of the deck slab. The laying of wet concrete began from the middle of the 110m span which caused lifting at the end spans. The proper casting of the deck slab might have avoided the collapse of the bridge.

2.4 DEVELOPMENTS IN COMPOSITE OPEN WEB STEEL GIRDER TECHNOLOGY

Experimental investigation on composite open web steel girder bridges began in the 1960s (Azmi (1972)). In 1965, five experimental investigations were conducted on 6.1m span conventional and composite open-web steel joists for buildings (Lembeck (1965)). The system was made composite by embedding the projection of web of members into a concrete slab above the top chord. It was observed that composite joists with the same tension chord had more strength and stiffness when compared to non-composite joists. In addition, Lembeck also concluded that the use of steel in the top chord of the joist can be considerably decreased. Wang and Kaley (1967) conducted four tests on composite and non-composite steel trusses in the elastic range. They also reported a significant reduction in deflection and stresses in the members. Five models of 4.9m composite open web steel joists made from both hot rolled and cold formed sections were tested by Tide and Galambos (1968). During the loading, most of the models failed due to connection failure between the steel member and concrete deck due to insufficient shear studs provided. They also concluded that most of the vertical shear in the structure is taken by the web members of the joist. They also recorded larger slips at the end spans indicating that shear connectors near the span end carry more shear force than the shear connectors in the mid-span.

Cran (1972) conducted full-scale tests on three composite open web steel joists spanning 12.2m, 15.2m and 6.1m. All the models showed ductile behaviour and failure was caused due to buckling of web members. This happened due to secondary stresses accumulated due to large deflections. Based on the observation that top chord compressive strains diminished to zero or even became tensile as the load was applied to the test specimens, Cran established that the top chord steel member contributed very little to the overall strength of the structure and can be neglected while calculating the overall strength. He suggested that for design purposes, the concrete slab can be considered as the top chord of the structure. He further recommended that the elastic deflection should be considered 10% more to account for the shear deflection and interface slip between the concrete slab and steel top chord.

Iyengar and Zils (1973) showed the practical application of composite trusses as large span floor beams. Research works by Brattland and Kennedy (1992) made the USA and Canada the leaders in the research on steel-concrete composite truss construction in the 1990's. Kravanja and Silih (2003) did an economical comparison between simply supported composite open web steel girders and composite I beam. The objective of the study was to identify the advantages of both the structural systems at various spans and loads. Both the beams were designed as per Eurocode 4 for serviceability as well as ultimate strength limit states. For increasing the accuracy of the study, instead of using classical structural analysis, a nonlinear structural optimization method was used. The comparison showed that for higher loads composite I beams were economical and for lower imposed loads composite trusses are more economical.

Shim et al. (2011) showed the use of high strength steel for the design of a double composite truss bridge. High-performance steel (HSB) has higher tensile strength and toughness than common steel. HSB is also more weld friendly than common steel. The yield strength of HSB 800 is 800N/mm^2. Due to buckling and fatigue, the design of long-span steel bridges using HSB is not common. For overcoming the obstacles, concepts of composite and hybrid bridges were used. For designing long-span bridges, a combination of double composite truss girders along with steel box girders were used. Using these techniques, bridges with spans up to 100m were designed. The advantages of composite bridges along with HSB were discussed.

Bouchair et al., (2012) explored the properties of the shear connection between concrete and open web steel girder bridge. He suggested that steel-RCC composite construction will be one of the most economical solutions in buildings and bridges for larger spans. In their study, the effect of connector diameter and top chord sections were studied along with different materials characteristics for steel and concrete. The shear force in connectors was calculated. The performance of beams in terms of strength and deflection was also established. The parametric study concluded that the flexural stiffness of the beam is not significantly affected by the change of steel top chord member's cross-section.

For understanding the mechanical behaviour of a composite joint in a cable-stayed bridge, model test and finite element analysis were done by XUE et al. (2011). For examining the truss joint, a scaled model test (1:2.5) was done. During the experiment, emphasis was given to the crack resistance ability and stress distribution in the joint was studied. The performance of headed studs was also investigated to establish the effectiveness of transmission of force by the joint. It was recorded that the strain in the steel plate and concrete became non-linear only after the load increased 1.7 times the design load. This means that the composite joints offer a significant factor of safety. The experimental results were then compared with the numerical finite element analysis results. Both experimental and numerical results were in good agreement with the strength and stiffness of the joint. The work done in the paper may lead to further research in composite joints for forming effective design guidelines for composite joints in cable-stayed bridges.

Cracks in the concrete deck of a composite open web steel girder bridge is an important issue to address for improving the life of the bridge. There are two primary causes of cracks in the deck, the physical and chemical properties of the concrete and tension produced in the deck due to external loads. Since 1970 a lot of stress was given to tension cracks due to external loads (Brozzetti, 2000). As the concrete deck in composite bridges is cast in-situ and is restrained due to shear connectors, tensile cracks develop due to two major factors:

i. The heat of hydration initially increases the temperature of concrete and then when the concrete cools down, minor cracks are formed.

ii. Due to shrinkage of concrete, which is due to decrease in volume of concrete as water evaporates.

Berthellemy J. (1993) observed cracks with width up to 0.3mm at the early stage of the setting. Mixtes P. (1995) identified the reasons and gave recommendations to minimize the cracks in the early stage of the concrete setting. Some parameters, which contribute to the initial cracks, are the type of cement, water-cement ratio, humidity in the atmosphere, admixtures used and temperature at the time of casting. Another major cause of cracking in the concrete in the composite bridge is the development of negative moments in the deck. This can however be addressed by providing adequate reinforcement in the tensile region.

2.5 LABORATORY EXPERIMENTS FOR ANALYSIS OF COMPOSITE ACTION

Research on composite open web steel girder bridges has picked up pace in the last couple of decades, but most of the research was focused on composite truss construction in buildings. Following are a few case studies of published lab experiments regarding under-slung and composite deck construction in bridges.

2.5.1 Composite open web steel girder bridge deck: *Instituto Superior Técnico, TULisbon, Portugal (J.P.R. Braz 2008)*

An experimental investigation was performed on a 3-D steel truss made composite with an RCC deck on the top. The objective of the test was to understand the behaviour of the composite structure. During the test, the model was statically loaded till failure. The system was analyzed using the moment-curvature method and numerical and experimental results showed good precision. The structure however failed due to local failures at various critical zones.

Experimental Setup

The tested model had a span of 7.5 m, with the middle span of 6m and 1.5 m span was of the cantilever type. The three-dimensional model was made using hollow circular steel sections. The concrete deck on the top had a width of 1100mm and thickness of

70mm as shown in Figure 2.12. The deck slab consisted of 5 mm diameter longitudinal reinforcement bars at a spacing of 100mm. The concrete deck was made composite with the steel truss using two rows of shear studs welded to the steel member at a spacing of 180mm. All the members of the open web steel girder had welded joints. Dimensions and arrangement of open web steel girder bridge members are shown in Figure 2.12.

The materials used in the fabrication of the model was tested in the laboratory to determine its properties. The stress-strain behaviour of the steel tubes and reinforcement bars are shown in Figure 2.13. The average compressive strength of concrete cube after 28 days of casting was 38.4N/mm^2 and the strength of concrete at the time of the test was 48.1 N/mm^2.

Member strains were recorded using electrical strain gauges and vertical deflections were measured under the locations of application of load using linear displacement potentiometers. For validating the data observed from the experiment, a moment-curvature analysis of the structure was carried out.

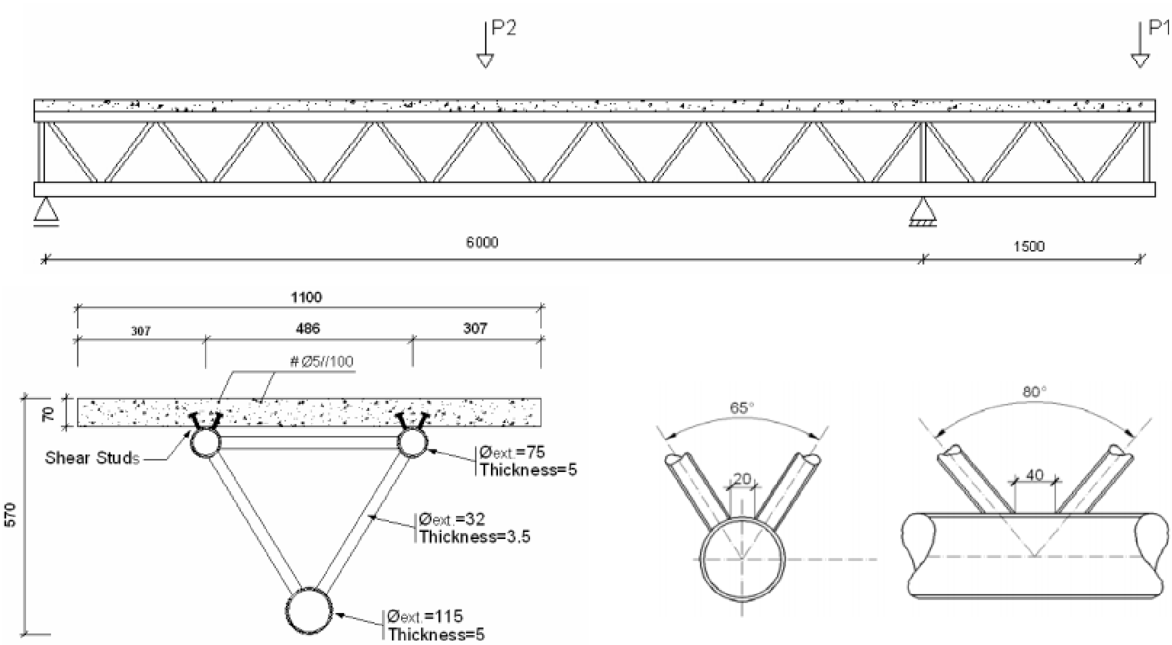


Figure 2.12 Truss arrangement and member dimensions for the test model

[J.P.R. Braz, 2008]

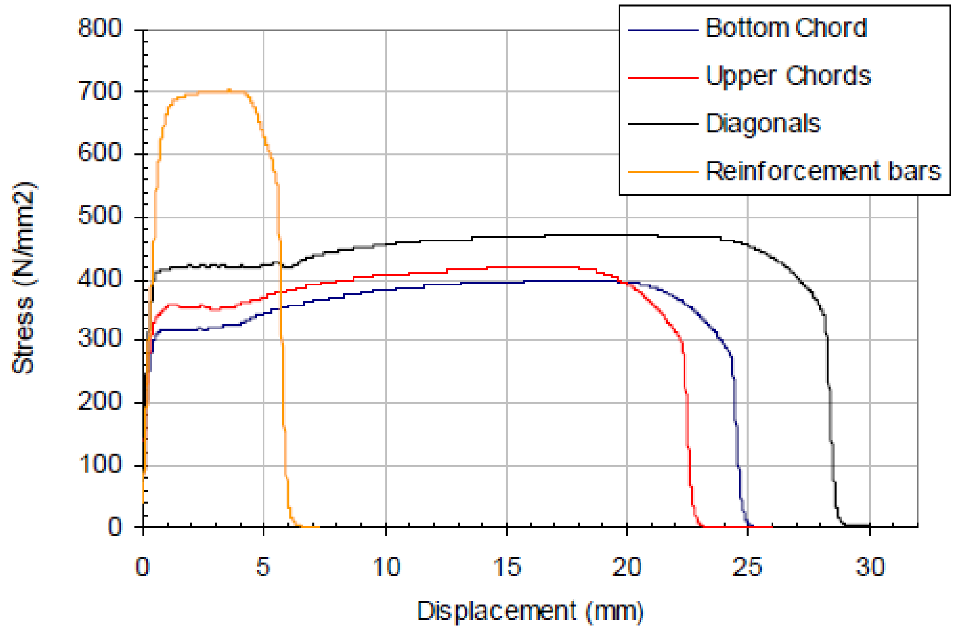


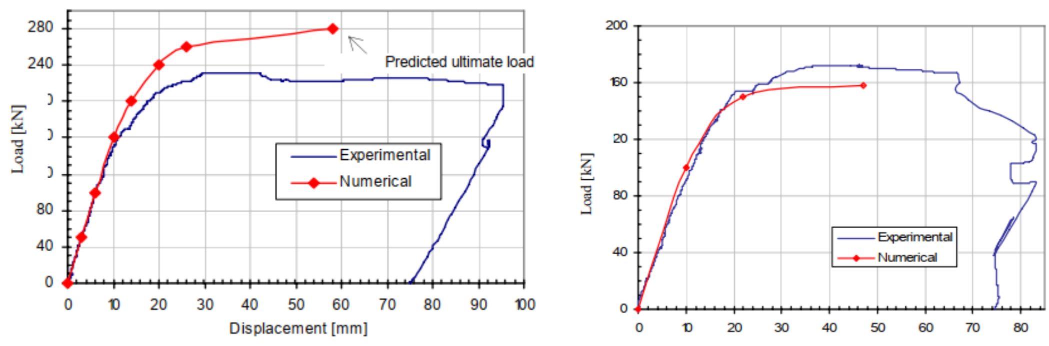
Figure 2.13 Stress-displacement variation for the steel members and reinforcement. [J.P.R. Braz, 2008]

Results and Conclusion

From the laboratory experiment, the following results were obtained.

- The deck slab cracked at a load of 42 kN. The numerical analysis also showed 50kN as the load for cracking of deck slab.
- The cantilever portion of the model failed at a load of 167 kN and the middle portion of the bridge failed at a load of 235 kN (Figure 2.14).
- The model failed due to the shear failure of the bottom chord and buckling of diagonal members.
- Local failure in the truss sections avoided stress redistribution in the middle section and hence required ductility was not achieved.
- Welds at the joints performed well throughout the experiment. No failure of joints was observed.

- Deck slab was under reinforced as full redistribution of bending moment was not achieved.
- Moment curvature analysis predicted the ultimate moment at failure fairly.



(a) (b)

Figure 2.14 Load-displacement relation for (a) middle section (b) end section. [J.P.R. Braz, 2008]

2.5.2 Composite steel and concrete bridge trusses

(Josef Machacek, Martin Cudejko, 2011)

Due of the benefits of composite construction is increasingly being used to build bridges nowadays. Because of the advantages composite construction offers, it is becoming a popular choice for bridge construction nowadays. Many bridges are constructed with the arrangement of parallel trusses connected with a concrete deck using shear connectors. Most of such bridges use fixed supports at the end. Although such support systems demand more concern for design and installation, they offer lower bending moments along the span of the bridge. The distribution of longitudinal shear in shear connectors is also better in such structural configuration.

An experimental study on a steel-concrete composite truss system was performed to understand the elastic and elastic-plastic shear distribution in shear connectors. In the study, numerical analysis was done on ANSYS software and the Eurocode approach for the distribution of longitudinal shear in shear connectors. The experimental results were compared to the numerical results. The analysis was done for geometric and material non-linear analysis using ANSYS software. Non-linear distribution of shear was considered for accurate design of shear connector's rigidity and spacing of the connectors near joints. Following the experimental studies and numerical analysis design recommendations for shear connectors were suggested. The concrete slab and steel flange were modelled as two-dimensional elastic beams. For analyzing the longitudinal shear in connectors, the tensile zone in the concrete slab was ignored.

Experimental Setup:

The 2D frame for the experiment was fabricated with a span of 21m. The steel members used were flat strip members of the following dimensions.

Upper flange: 250mm x 20 mm
Bottom flange: 300 mm x 40mm
Diagonal members 250mm x 40mm.

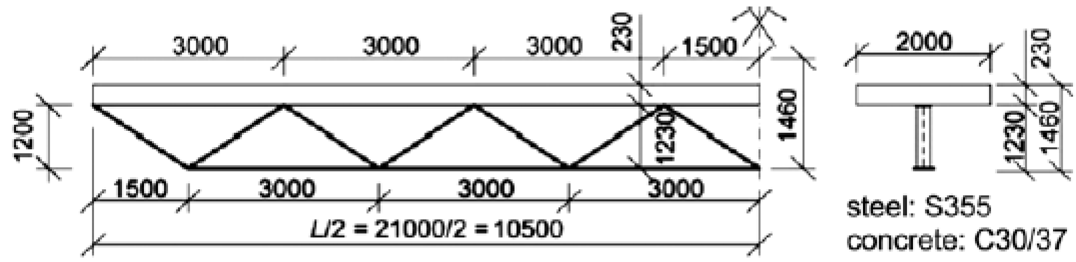


Figure 2.15: Truss arrangement for experimental study.
[J. Machacek, M. Cudejko, 2011)]

The geometrical configuration of the truss is shown in Figure 2.15.

Headed studs were used as shear connectors. The diameter of the studs was 19mm and the ultimate strength of the stud was estimated to be 340 MPa. The studs were provided in 3 rows at a spacing of 200mm longitudinally. The shear connectors were designed as per Eurocode 4. The grade of concrete for the deck was C30/37. The material properties of steel and concrete used are shown in Figure 2.16. The shear strength of the connectors was calculated as 3 × 77 100/200 = 1156 N/mm (T1).

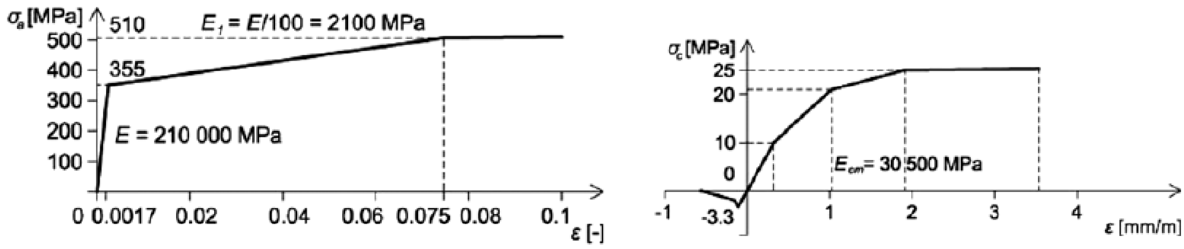


Figure 2.16 Stress-Strain relation for the materials used.
[J. Machacek, M. Cudejko, 2011)]

The focus of the study was to understand the shear distribution in shear connectors. Simplified two-dimensional analysis shows that the shear peaks at connectors gives higher values and closely tally to Eurocode values. In Figure 2.17, a comparison of all the analyses is shown.

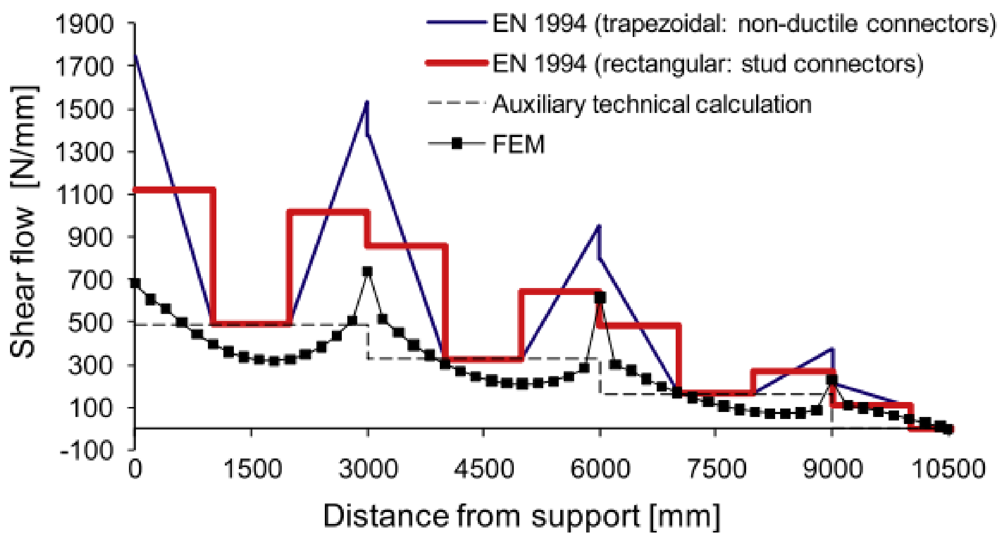


Figure 2.17: Comparison of Eurocode 4 results with numerical results.

[J. Machacek, M. Cudejko, 2011)]

In the simplified approach, headed stud connectors were simulated as elastic cantilevers. The analysis gave reasonable results and can be used to find the optimum shear connection. For the analysis, the parameters of the concrete slab, the rigidity of the shear connector and the flange of the steel member play an important role in calculating the shear flow.

2.6 COMPOSITE OPEN WEB STEEL GIRDER BRIDGE

In open web steel girder bridges, the members are subjected to axial forces which results in better utilization of materials hence making them an efficient structural system (R.P.Johnson et al., 2001). Structural steel behaves almost similarly in compression and tension. However, the members under compressive loads may prematurely buckle before the compressive stresses reach material strength. In this context, the composite action with the compressive member of the open web steel girder bridge may prevent buckling and hence making the structure more efficient.

In highway bridges, a flat deck is required for the movement of traffic. If the deck is made composite with the compressive members of the open web steel girder bridge, the system may become more economical and may even be used in long-span bridges. The shear transfer between the concrete deck and steel sections can be attained by using well-designed shear connectors. Concrete is cheaper than steel and can carry compression more economically. In addition, due to the lower strength of concrete compared to steel, larger sections of concrete are required. Such large sections of concrete make it less susceptible to buckling failure. This makes composite open web steel girder bridges structurally efficient and economical, making them a popular choice for medium span bridges.

Types and examples of composite open web steel girder bridges

Following types of composite open web steel girder bridges are constructed in the past.

i. Simply supported composite open web steel girder bridge.

ii. Single deck continuous composite open web steel girder bridge.

iii. Double-deck continuous composite open web steel girder bridge.

iv. Composite open web steel girder bridge with prestressed deck

i. Simply supported composite open web steel girder bridge

Composite construction of RCC floors with trusses is common in the case of building construction, and with steel plate girders in the case of composite plate girder bridges. However, not much literature is available for composite steel open web steel girder bridges. In India construction of composite steel open web steel girder bridges is not common, and hence research in this area is required to avail benefits of the composite construction in the case of bridges.

Open web steel girder 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 open web girder bridge is in tension in simply supported span, a composite deck 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 compression members, prevents the buckling of top chord members and adds in cross-sectional area of the compression members. To make the RCC deck slab composite with the top chord members, shear studs between the top flange of the top chord members and the RCC deck are required. Thus, the benefits of composite construction can be easily obtained in the case of composite deck type truss bridges.

Twelve simply supported composite truss bridges were built in the Czech Republic during the last decade. The bridges had spans between 21m to 63 m. Similarly, in other countries, this type of truss bridge has been constructed (Figure 2.18) (J. Machacek, et al., 2011).

Figure 2.18 Simply supported composite truss bridge of span 36m

[J. Machacek, M. Cudejko, 2013)]

In the deck type composite bridge due to shrinkage strain in the deck slab, composite behaviour between the steel truss and deck slab may start only when shrinkage strain is overcome by the flexural strain in the deck slab due to part live load. Thus, the 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 the 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 the 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 the case of the Chauras and Garudchatti bridges in Uttrakhand, India. Three-span continuous geometry was adopted in these cases, where the ratio of side span to the main span was kept as 0.364. In the Chauras bridge uplifting of the side span during deck slab casting caused its failure.

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 the bridge's 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 2.19) 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).

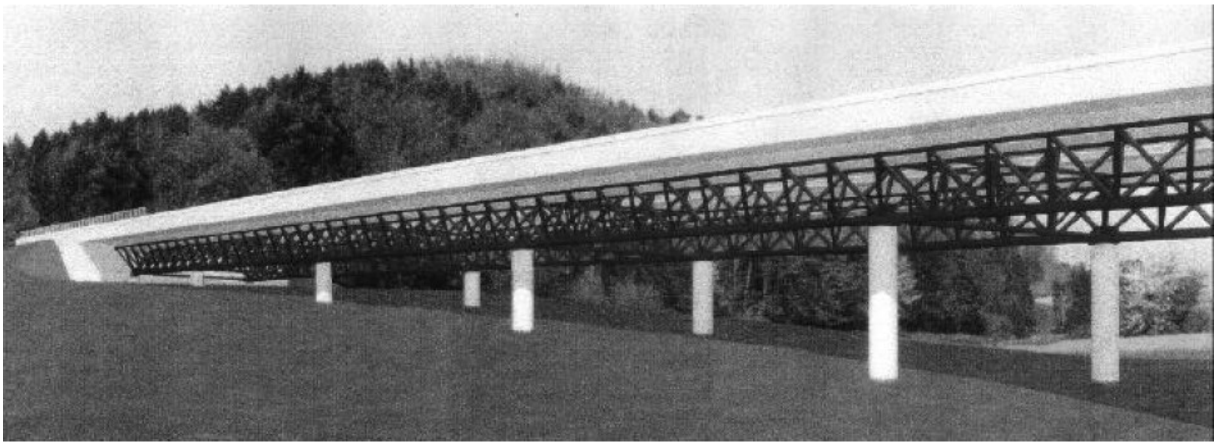
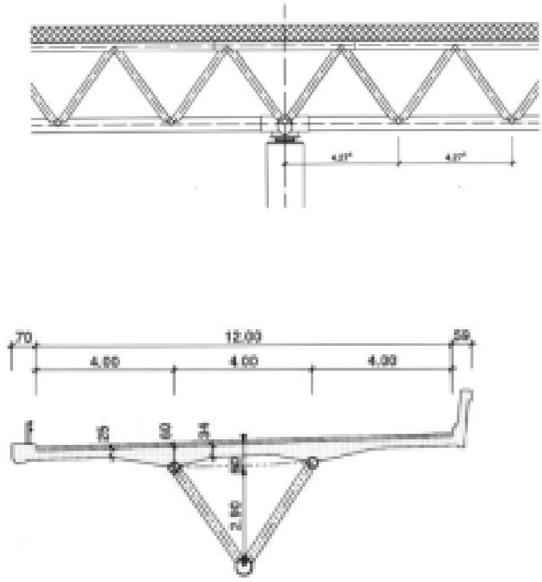


Figure 2.19 General view of the Lully Viaduct composite bridge

[H. G. Dauner, et al., 1998]

Unstiffened circular tubes were used to fabricate the two triangular trusses Both the trusses were then connected with the help of a tubular brace truss at each support location (Figure 2.20). 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 2.20 Longitudinal view and standard cross-section

[H. G. Dauner, et al., 1998]

Welded shear connectors were used to make the deck composite with the upper chord members as shown in Figure 2.21.

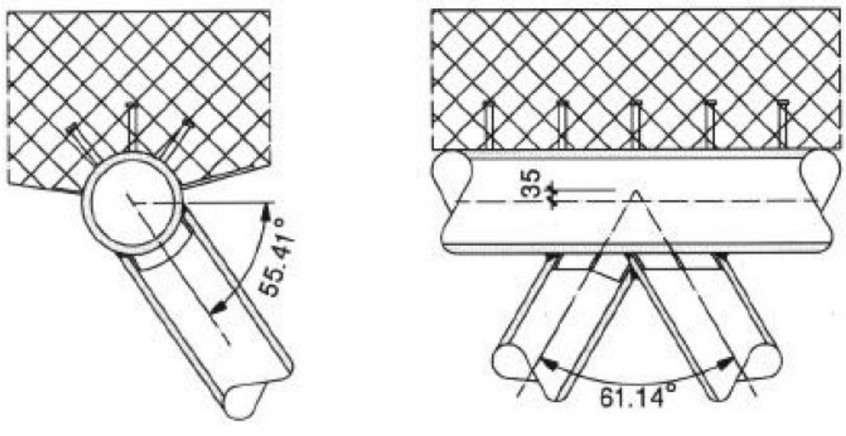


Figure 2.21 K-shaped joint geometry with shear studs [H. G. Dauner, et al., 1998]

Another example of this type of bridge is Bogibeel bridge (Figure 2.22), which is a combined road and rail bridge in the Dibrugarh district of 4.94 kilometres length in Assam. The bridge was opened for traffic in December 2018.

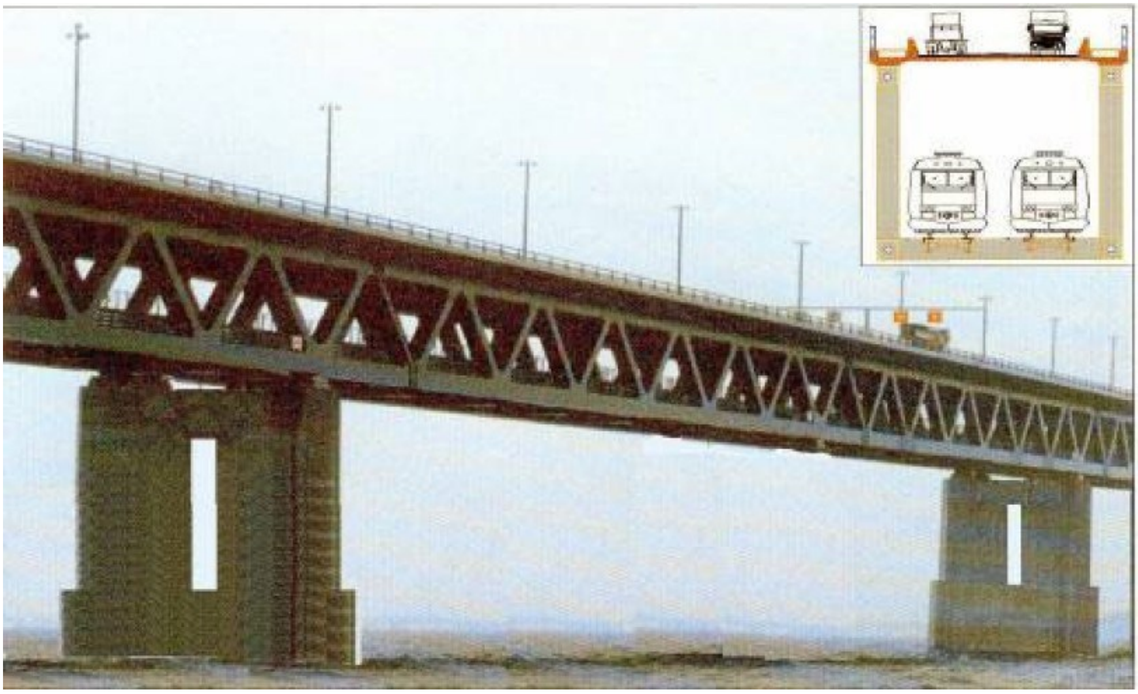


Figure 2.22 Bogibeel bridge [Google images]

This is the first composite truss bridge in India (A. K. Goyal). The rail-road bridge double-deck bridge consists of a three-lane road on the upper level and a two-line railway track on the lower level. The distance between the road level and the railway line is be 10.5m. The superstructure is made of steel trusses spanning 125m and 32.6m. 125m span truss is a warren type truss with 10 panels of 12.16m each. Welded plates are used to make members of either the I section or Box sections. For the railway deck on the lower level, cross-girders are placed at a spacing of 12.16m centre to centre. The connections between the stringers and cross girders are welded. All lateral and cross-bracing members also have welded joints. The upper deck for the roadway is made composite with the top chord and cross girders. All cross girders at this level are at a spacing of 3.04m. End spans of 32.6m, as well as 39spans of 125m, are fabricated at the site and then launched.

Due to shrinkage strain in deck slab and flexural strain in the case of composite top deck type bridge, micro-cracks will develop in the slab. These cracks will lead to the seepage of water which will cause corrosion of the rebar and hence accelerate the fatigue of the structure.

iii. Double-deck continuous composite truss bridge

In the 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. To prevent buckling of the top chord members and bottom chord compression members and to increase the cross-sectional area of the truss, RCC deck slabs are helpful, both at the top as well as at the bottom of the truss.

An example of this type of double-deck continuous composite truss bridge is the Ulla river viaduct (Figure 2.23) over the river Ulla in Spain. It is a steel lattice with double steel and concrete composite work at the hogging zone. It consists of three main spans of 225m+240m+225m long and several approaching spans measuring 120m long. The four central piers are rigidly connected to the latticed deck (F. Millanes, 2010).

Figure 2.23 View of the Ulla river viaduct [F. Millanes, 2010]

Upper chord members are made composite with the upper deck slab using 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 a single top deck continuous composite steel truss bridge. Further, the live load moment at the supports is twice that of the mid-span moment, and therefore, variable thickness of slabs may be required.

iv. Composite steel truss bridge with pre-stressed deck

In the case of double-deck system continuous composite truss bridges, the upper deck at the support locations and the 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 2.24) (K. Ohgaki, 2011).

Figure 2.24 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 2.25).

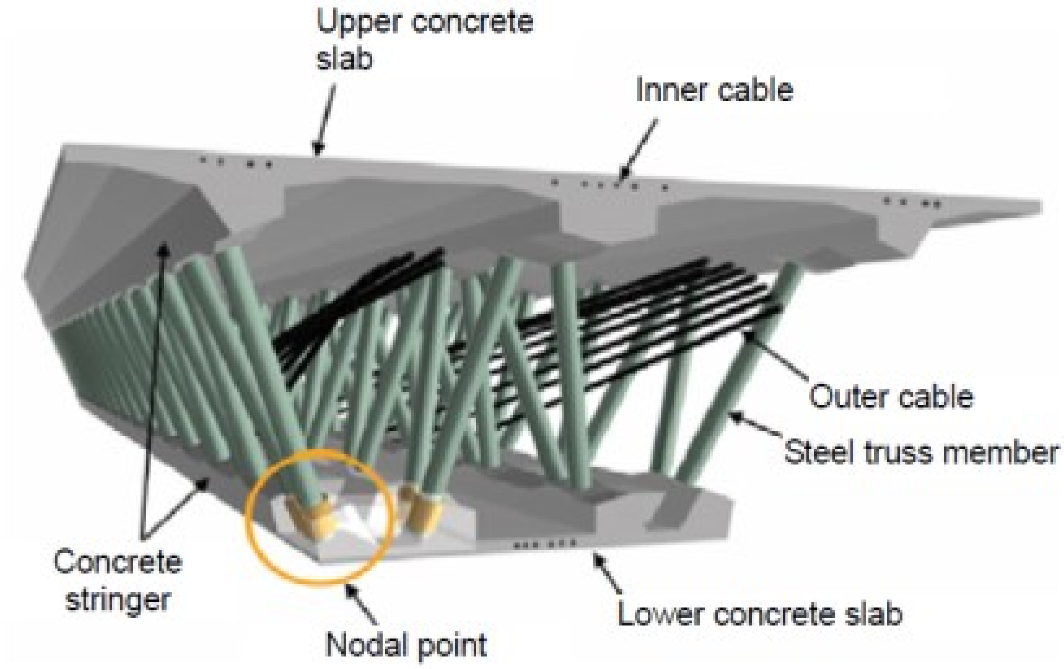


Figure 2.25 Composite construction with prestressing of upper and lower slabs [K. Ohgaki, 2011]

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

Prestressing the deck slab using cables embedded in the deck slab is not desirable. 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 the top and bottom slabs does not provide a suitable solution particularly to prevent cracking of the top deck slabs.

Advantages and disadvantages of continuous span composite truss bridges

Continuous span 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 the case of the Chauras and Garudchatti bridges in Uttrakhand, India. Three-span continuous geometry was adopted in these cases, where the side span to main span ratio was kept as 0.364. In the Chauras bridge uplifting of the side span during deck, slab casting caused its failure.

RCC top deck can be made composite with the continuous span steel truss. Due to hogging moments at the supports, the deck slab in this case is under tension and its contribution to the 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 conditions, 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.

Controlling the propagation of cracks is vital for continuous steel and composite bridges (J. He, et al., 2010). Such cracks permit harmful substances to seep into the concrete decks. Due to the cracks corrosive agents like chlorides, sulfates along will water permeates into the deck slab of the bridge can cause rapid deterioration. With increased deterioration, larger cracks may appear which will cause spalling of concrete and hence decrease the cross-section of the deck slab and hence the load-bearing capacity of the bridge rendering it unsafe for use. Because bridges are important links, repair of such decks is often difficult and expensive. Therefore, avoiding crack formation in the early stage is very important.

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

There are some methods available for controlling the crack growth for example use of a relatively large amount of rebar, prestressing of the deck in hogging or negative bending moment zone. To limit the Crack 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 off the side span will take place as evidenced in the case of the Chauras bridge. Thus, small continuous side spans with long main spans are not advised. Therefore, simply supported medium spans between 30m to 100m of deck type composite truss bridges are most suitable, especially for deep valley conditions in mountainous terrains.

2.7 SHEAR CONNECTION IN COMPOSITE TRUSS BRIDGES

According to cl 606.1 of IRC 22-2015, the acting together of the girder and slab as a unit is ensured by the use of a mechanical device known as shear connectors.

The three types of shear connectors are given below.

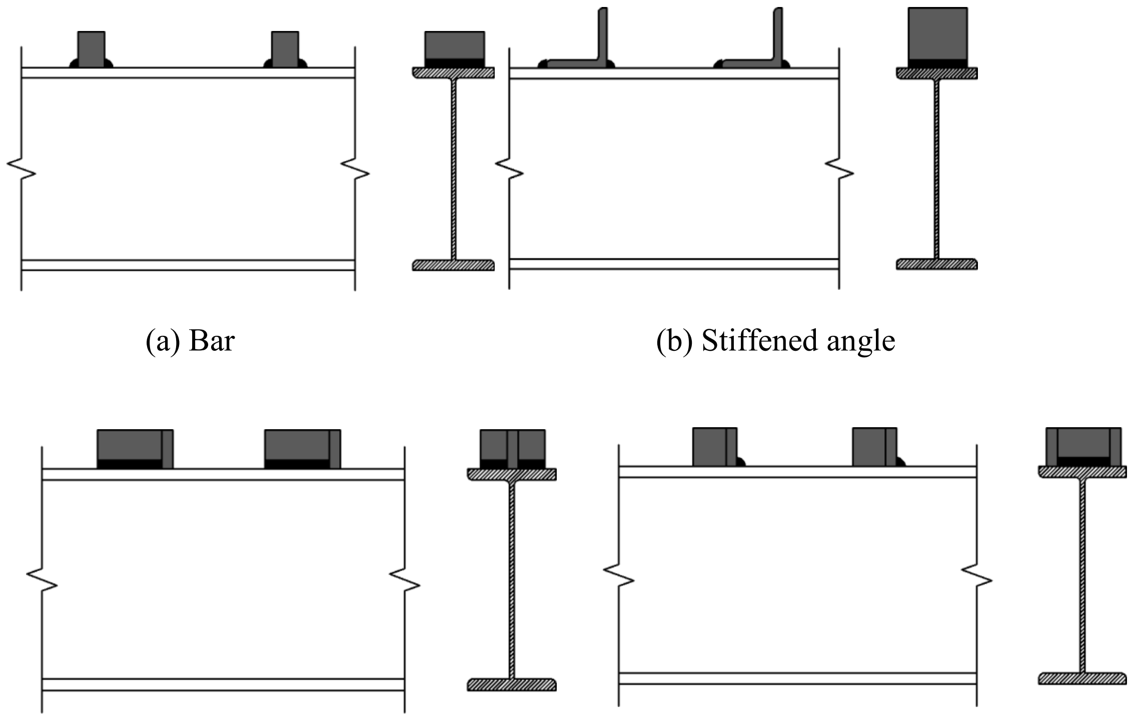
i. Rigid shear connectors

ii. Flexible shear connectors

iii. Anchorage shear connectors

Rigid shear connector

Rigid shear connectors are made of short stiff bars, channels, stiffened angles, or tees welded on top of the flange of the steel girders. These connectors develop resistance to horizontal shear by bearing against concrete. Anchorage devices should be provided with these connectors as shown in Figure 2.26.



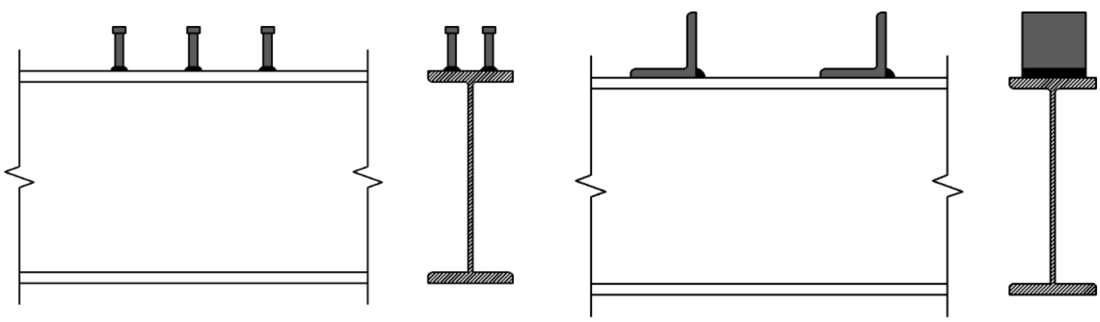
(a) Bar (b) Stiffened angle

(c) Tee (d) Channel

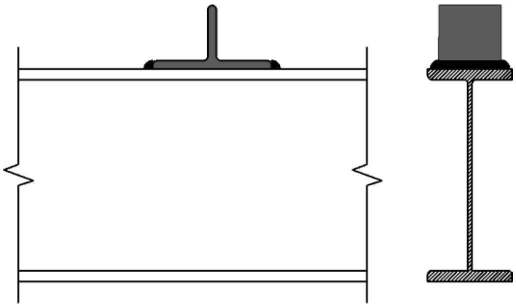
Figure 2.26 Rigid connectors

Flexible shear connector

Flexible shear connectors are made of studs, angles or tees welded on the top flange of the steel girder. These connectors resist horizontal shear by bending the connectors some of this type are shown in figure 2.27. Where necessary, such shear connectors shall be provided with anchorages devices.



(a) Headed stud (b) Angle



(c) Tee

Figure 2.27 Flexible connectors

Anchorage shear connector

Anchorage connectors (Figure 2.28) are also effective in resisting horizontal shear along with separation of the concrete deck with girder at the interface through the bond.

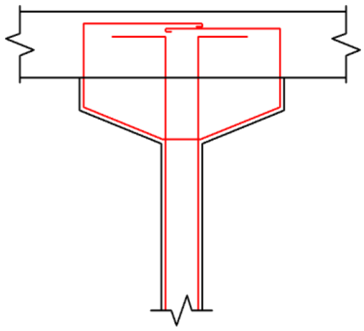


Figure 2.28 Anchorage shear connectors

Headed shear studs are most suitable for composite steel truss bridges as these facilitate reinforcement layout and concreting in the deck slab. Figure 2.29 shows the headed shear connection in a plate girder bridge.

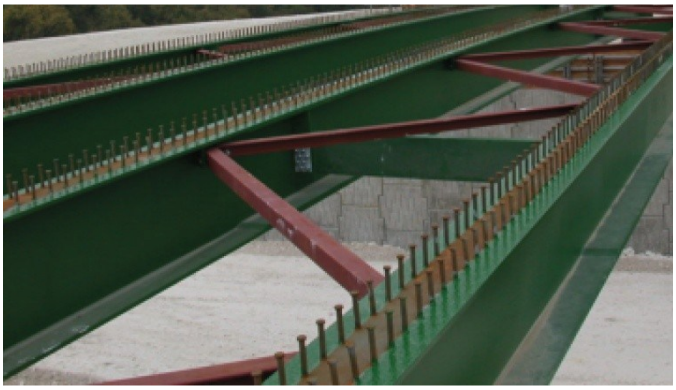


Figure 2.29 Headed shear connectors

[A41 Aston Clinton Bypass, www.steelconstruction.info]

A lot of research has been conducted for the calculation of longitudinal shear in shear studs of composite open web steel joists for buildings. The challenges in the design of shear connectors are:

1. *Choosing the type of shear connector:* The types of shear connectors depending on the behavior of connectors are mentioned above. It is a designer's choice to choose appropriate shear connector for the project.
2. *Strength of shear connectors:* The strength of shear connectors depend on a number of parameters like load on the section, material properties of concrete and steel, type of connector used etc.. The spacing of shear connectors is also dependent on these parameters.
3. *Loss of shear strength due to fatigue:* Due to repetitive cycles of loading and unloading, the shear connectors may fail in providing adequate shear connection.
4. *Shear concentration near joints:* Various literature have suggested that in case of open web steel trusses, the shear stress does not vary linearly. The shear has larger force near the joints and the force decreases as we move away from the joints. This factor should be taken into account while designing shear studs for

open web steel girder bridge decks.

Design procedures are evolved based on experiments and numerical studies. Such results are used in the Canadian (CAN/CSA S16.1, 1997) and American codes (SJI-CJ-2010). But, the given design guidelines are limited to the composite trusses for the buildings.

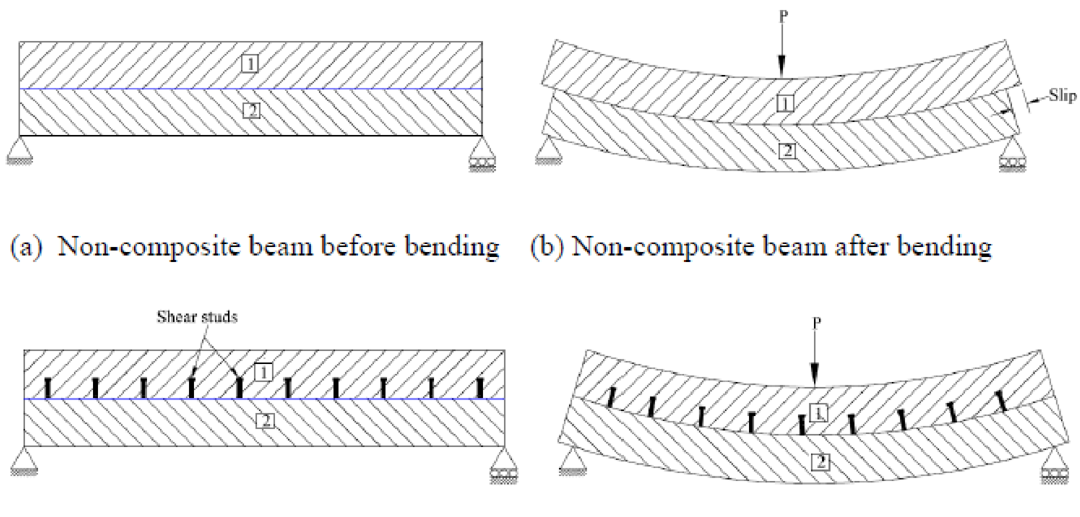
The design of composite steel and concrete truss girders have been extensively discussed by Machacek and Cudejko (2009) and Machacek and Charvat (2011). Special emphasis has been given to the connection of steel truss and concrete slabs.

They conducted an experimental investigation on two steel-concrete composite truss girders. Perforated shear connectors were used to transfer horizontal shear. The experimental results were then used to calibrate a three-dimensional nonlinear finite element model made using the ANSYS software package. Using load slip relationships from previous researches, they analyzed more than 30 variants of shear connections for a simple truss at the mid-span. Within elastic behaviour, peaks of shear were observed above truss nodes. Shear connectors loaded in fatigue and during plastic redistribution in plastic range showed distinctive peaks near truss nodes. The numerical results were then compared with the provisions of Eurocode 4 and densification of shear connectors near truss nodes were suggested.

Machacek and Cudejko (2011) studied longitudinal shear flow in shear connectors of composite truss bridges. Nonlinear shear flow distribution between steel truss and concrete slab along the span of composite truss girder was found. A wide range of design aspects important for steel and RCC composite trusses were studied by Neal and Johnson in the nineties. The research gave recommendations for the plastic design of shear connectors for plate girders. They pointed out the importance of the ductility of shear connectors and flexural rigidity of the upper flange of the steel girder. The effects of longitudinal force introduced due to prestressing of the concrete slab were discussed by Johnson and Ivanov (2001). Their study was later used to form formulae for the local effect of the concentrated longitudinal force in concrete decks in Eurocode 4 (ENV 1994-2, 2001). Although, the results were based on linear analysis of shear connectors and gave conservative results.

2.8 BEHAVIOUR OF SHEAR CONNECTORS

The purpose of the shear connector is to prevent slip between the deck slab and the steel member. Considering a simply supported non-composite beam (Figure 2.30.a) comprising two different beams 1 and 2, when loaded with external load 'P' (Figure 2.30.b), due to shear at the interface of the beams, these will behave individually and slippage will occur at the interface. When beams ‘1’ and ‘2’ are made composite with the help of shear studs (Figure 2.30.c), the shear at their interface is transferred through shear studs and these act together under external loading (Figure 2.30.d).



(a) Non-composite beam before bending (b) Non-composite beam after bending

(c) Composite beam before bending (d) Composite beam after bending

Figure 2.30 Non-composite and composite beams

2.8.1 Mechanism of flexible shear connectors

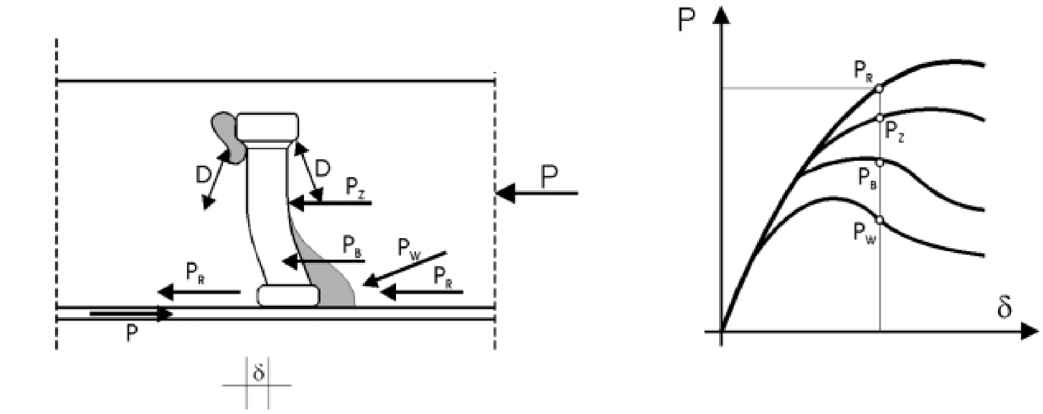


Figure 2.31 Mechanism of shear force distribution for stud type shear connector [S. Rankovic, D. Drenic (2002)]

The shear force is transferred from the steel member to the concrete slab through the shear stud. As shown in Figure 2.31, let us assume the shearing force P is to be transmitted to the concrete slab. P force enters the base of the shear stud at the base of the connector. Due to this, force P_W is introduced at an angle at the weld connecting the stud with the flange of the member. Due to increased pressure, the concrete near the base of the stud gets crushed and the shear force transferred to the shaft of the connector is P_B. This force causes the shaft of the stud to bend and plastic deformation occurs. This bending also induces tensile force in the stud and hence it inhibits vertical lift. Concrete present under the head of the connector experiences pressure due to the formation of tensile stress in the connector. This pressure increases the frictional force at the contact between the steel flange and concrete.

The shaft of the connector experiences the horizontal component of the tensile force 'P_Z'. Due to tensile force and shear force acting together, the shear stud fails at the weld location. At the yield point, the behaviour of flexible and rigid shear connectors is the same. Both the connectors at higher stresses don't allow deformation to occur and hence movement between the steel and concrete is restricted.

2.8.2 Longitudinal shear flow in solid and open web steel girder bridges.

Shear flow in shear connectors for solid sections and open web steel girder is shown in Figure 2.32 in elastic and plastic stages (J. Machaceka, et al., 2013).). In the case of solid web bridges (Figure 2.32.a) under uniformly distributed load shear transfer between the web and the deck takes place at a constant rate during the elastic condition, and in the plastic condition, the shear transfer becomes constant through all the shear connectors.

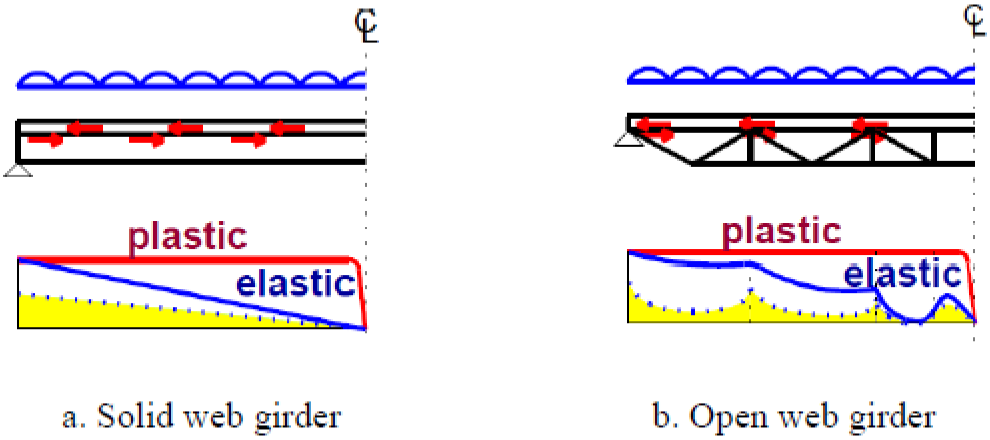
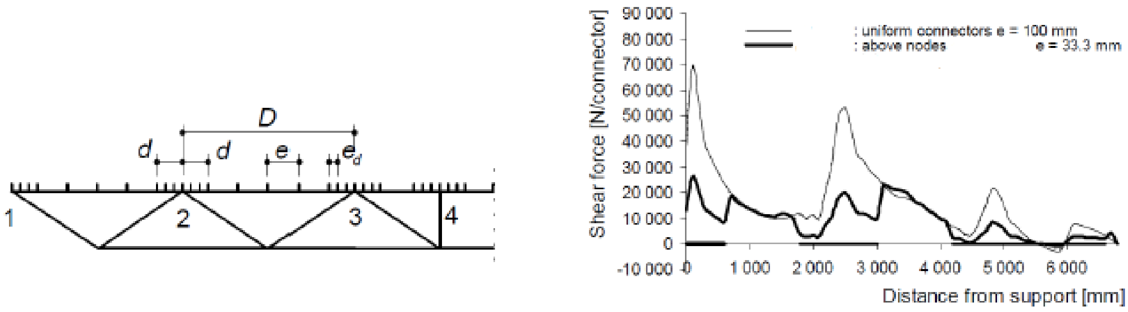


Figure 2.32 Shear flow in shear connectors [J. Machaceka, et al., 2013]

In the case of open web girder bridges (Figure 2.32.b), shear flow in elastic conditions between the web and the deck is concentrated near the truss joints, for which densification of studs is required.

Densification of shear connectors over truss nodes required for shear transfer in the elastic condition is shown in Figure 2.33. Figure .33 a shows arrangement of shear studs in the case of the composite truss where the concentration of studs is shown in distance ‘d’ from the nodes and ‘e’ is the spacing of studs in the remaining length. Figure 2.33.b shows the variation in shear force per connector for uniform spacing of shear connectors and connectors above nodes.



a. Arrangement of shear studs

b. Variation in shear force per connector

Figure 2.33 Densification of shear connectors [J. Machaceka, et al., 2011]

In composite steel truss bridges casting of the deck, the slab is cast after launching of the steel truss. In the composite system, shrinkage strain occurs in the deck slab during the hardening process. The average shrinkage strain in M40 grade concrete of the deck slab is of the order of 0.0003 (IS 1343-1980). Until this shrinkage strain is overcome by the flexural stress under the live load condition, the studs can mainly offer lateral support to the top chord members of the truss, and full advantage of the composite deck will not be available. Therefore, spacing of the shear studs in the case of steel truss bridges will be mainly guided by the lateral restraint consideration and not by purely shear transfer consideration.

2.9 CODAL PROVISIONS FOR SHEAR CONNECTORS

Various provisions in codes for shear connectors in composite building trusses and composite solid web bridges are discussed here.

2.9.1 Indian code provisions.

Indian code IRC 22-2015 provides details of shear connectors mainly in composite solid web girder bridges. The mathematical formula given in this code for calculating longitudinal shear does not apply to composite steel truss bridges. Therefore, longitudinal shear for composite truss bridges has to be defined by any other suitable method.

In Indian code, the fatigue strength of stud connectors is given by the following expression

Design as per fatigue characteristics

$$Q_r = \alpha \text{ x } A \ 10^{-3} \tag{3.1}$$

Where,

Q_r = Allowable range of horizontal shear per stud connector (kN)

A = Area of Stud (mm^2)

α = 55 MPa for 2 x 10^6 cycles

Design as per strength characteristic

$$\boldsymbol{V_L} = \sum\left[\frac{\boldsymbol{V.A_{ec}.Y}}{\boldsymbol{I}}\right]_{\boldsymbol{dl,ll}} \tag{3.2}$$

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.

Apart from the above two methods, spacing of the shear connectors may also be calculated for full shear connection as per section 606.4.1.1 of IRC 22-2015.

2.9.2 Canadian code provisions.

In Canadian code, provisions given composite truss and Open Web Steel Joist (OWSJ) for buildings are given in CISC 2003 (CAN/CSA S16.1-1997). As per this code, the design of OWSJs are for loads applied to the top chord and which are acting in the plane of the joist. It is assumed that the top chord is prevented from lateral buckling. For calculating the axial forces in members, it is assumed that the members are pin-jointed. Actual loads are converted to statically equivalent loads, which are assumed to act at the panel points.

The total horizontal shear for full shear connection, V_h, at the junction of the steel truss or joist and the concrete 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. It is assumed in this code that the entire tension is taken by the bottom chord and equal compression is taken by the composite deck slab above the neutral axis.

Between the point of maximum bending moment (positive or negative) and zero moment, the number of shear connectors is given by the following equation.

$$n = \frac{V_h}{q_{rs}} \tag{3.3}$$

Where, q_{rs} is the factored shear resistance for headed end welded studs in solid slabs and is given by

$$q_{rs} = 0.50\, \varphi_{sc}\ A_{sc} \sqrt{f_c^{`} E_c} \leq \varphi_{sc}\ A_{sc}\, F_u \tag{3.4}$$

Where

F_u = 415 MPa or commonly available studs

A_{sc} = Cross-section of a steel shear connector

φ_{sc} = The resistance factor to be used with the shear resistances = 0.80

E_c = Modulus of elasticity of concrete = 4500 $\sqrt{f_c^`}$

2.9.3 American code provisions.

Provisions in American codes for composite steel truss for bridges are not provided and for buildings, these are given in American National Standard for Standard Specification for Composite Steel Joists C-J series, SJI-CJ-2010.

Shear capacity of a single shear stud for studs in 1.5 in. (38 mm), 2 in. (51 mm), or 3 in. (76 mm) deep decks with $d_{stud}/t_{top\ chord} \leq 2.7$ is given by,

$$Q_n = \text{Min}\left[0.5\ A_{stud}\sqrt{f_c' E_c}\ or\ \left(\frac{R_p R_g A_{stud} F_{u\,stud}}{1000}\right)\right]\text{kN} \quad (3.5)$$

Shear capacity of a single shear stud for studs in 1.5 in. (38 mm), 2 in. (51 mm), or 3 in. (76 mm) deep decks with $2.7 \leq d_{stud}/t_{top\ chord} \leq 3$ is given by,

$$Q_n = \text{Min}\left[0.5\ A_{stud}\sqrt{f_c' E_c}\ or\ \left(\frac{R_p R_g A_{stud} F_{u\,stud}}{1000}\right) - 6.67\left(\frac{d_{stud}}{t_{top\ chord}} - 2.7\right)\right]\text{kN} \quad (3.6)$$

Where,

A_{stud} = Cross-sectional area of shear stud, in^2 (mm^2)

d_{stud} = diameter of shear stud, in. (mm)

E_c = Modulus of elasticity of concrete, ksi (MPa)

f_c' = specified minimum 28-day concrete compressive strength, ksi (MPa)

$F_{u\,stud}$ = minimum tensile strength of stud, 65 ksi (450 MPa)

Q_n = shear capacity of a single shear stud, kips (kN)

R_p = shear stud coefficient

R_g = 1.00 for one stud per rib or staggered position stud

= 0.85 for two studs per rib side by side

= 0.70 for three studs per rib side by side

$t_{top\ chord}$ = thickness of top chord horizontal leg or flange, in. (mm)

2.9.4 Euro code provisions.

Design of composite bridges as per European codes is dealt in ENV 1994-2-2001, 'Design of Composite Steel and Concrete Structures, Part - 2, Composite Bridges' (Eurocode-4).

As per Euro code 4, in a solid slab, the capacities of shear connectors to resist longitudinal shear should be taken as smaller of values obtained by the following equations.

$$Q_m = 0.29\ \mathrm{d}^2 \sqrt{f_c^{`} E_{cm}} \qquad \text{or} \qquad Q_m = 0.8\, f_u\, ({\pi d^2}/{4}) \tag{3.7}$$

Where,

$f_c^{`}$ is the concrete compressive strength;
E_{cm} is the mean elastic modulus of the concrete;
d is the diameter of the stud shaft;
f_u is the tensile strength of the stud shank.

The value of E_{cm} is given by:

$$E_{cm} = 22\left(\frac{f_{cy}}{10}\right)^{0.3} = 32500 \text{ MPa} \tag{3.8}$$

Where, f_{cy} is the cylinder strength of the concrete.

2.10 LITERATURE SURVEY

1. Buckling of compression members in steel truss bridges causes sudden collapse. This failure gives no warning, causing economical and life loss. The composite action of the 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. Though composite action of RCC deck with steel truss in deck type truss bridges is advantageous, detailed design guidelines for the design of composite

truss bridges is not available. Therefore, keeping in mind the advantages of a composite truss bridge, standard design guidelines for it are necessary.

3. Shrinkage strain in RCC deck may not allow composite action of deck slab with steel truss unless it is overcome by flexural stresses due to live load. Therefore, the advantage of composite action may be available only at overload conditions or at plastic collapse.

4. In the case of continuous composite truss bridges, the hogging moment appears at the supports. The deck slab in this case is under tension and its contribution to the 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.

5. Shear studs near the supports and nodes of the trusses experience a sudden increase in horizontal shear in them.

6. Due to composite action, the tendency of failure of top chord compression members is reduced. The mode of failure of composite bridge will shift to the tensile failure of bottom chord or shear failure of web members.

The explored literature review and current practices of composite construction indicates that the composite bridge construction is going to become mainstream in near future. The motivation behind the research work carried out is to evaluate the performance of composite underslung open web girder bridge system.