# A COMPOSITE FLOOR TRUSS TOP CHORD USING CONCRETE-FILLED STEEL TUBE (CFST)

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## Abstract

Steel and concrete composite systems are generally used as major structural components in multi-storey buildings. Steel decking is a more common composite system in buildings since it serves as a working platform to support the construction loads and also acts as a permanent formwork for concrete. To achieve large column-free spans (in the range of 8m-12m), as often demanded in multi-storey office buildings, steel and concrete composite floor trusses may form economical solutions since they are able to accommodate various service ducts within the structural zone. The concept of introducing a concrete-filled steel tube (CFST) instead of the conventional open-flanged steel section, as the top chord of these floor trusses has been discussed. However, the viability of this new concept should be ensured with experimental evidence on the longitudinal shear transfer capacity at the composite stage. This paper discusses the experimental results of a series of push-off tests conducted on CFST embedded composite slab panels. The effect of providing different concrete top covers and the effect of different concrete strengths were investigated and the results compared with existing guidelines for headed shear studs.

**Keywords:** composite slab, concrete-filled steel tube

# 1. Introduction

Composite construction using steel and concrete has been used since the early 1920s. It gained widespread use around the world in bridges in the 1950s and in buildings in the 1960s. This is due to the advantages of composite construction, including the reduction of steel use, a reduction of overall structural depth, and the increase in floor stiffness and load capacity (Perera 2007).

Profiled steel sheeting is most commonly used in composite construction in buildings since it serves as a working platform to support the construction loads and acts as a permanent formwork for the concrete. This eliminates the need for traditional, temporary forms and falsework. Also, not only the sheets are suitably shaped to ensure a proper bond with the concrete, but the sheeting can provide all or part of the main tension reinforcement in the slab as well.

Steel and concrete composite systems are generally used as major structural components in multi-storey buildings. Therefore, the structural arrangement of floors is particularly important. Several different configurations of composite floor systems namely, composite stub-girders, slim floor systems, composite trusses, and composite beams with web openings in the steel beam are in use worldwide for long spans.

A composite truss is a steel truss, the top chord of which is designed to act compositely with a concrete slab right above it. To achieve large column-free spans (in the range of 8m ~ 12m), as often demanded for multi-storey office buildings, steel concrete composite trusses may form an economical solution since they provide the ability to accommodate various service ducts within the structural zone (i.e. these could be passed through the openings in the truss) which would otherwise have to be placed underneath it.

In this type of construction, the bare steel truss is generally expected to withstand the construction stage loads until the composite action develops in the top chord when the concrete is hardened. Consequently, the size of the steel top chord member is governed by the construction stage loading (non-composite action). This means the composite truss contains a more than adequate amount of structural steel fixed to its top chord for the serviceability and ultimate design states. Hence, an economical design may be achieved by introducing alternative means for the truss top chord, which is to reduce structural steel. Instead of the conventional open flanged steel section, as in the top chord of these floor trusses, one such alternative is to use a concrete filled steel tube (CFST), as described in this study.

The use of hollow steel tubes filled with concrete has become widespread in the past few decades. This is mainly due to their high strength, high ductility, and large energy absorption capacity. In this type of composite truss the uncertainty is with the shear connection rather than its compressive strength capacity as a top chord of the truss. The viability of this concept could be ensured by experimental evidence on the longitudinal shear carrying capacity at the composite stage.

In conventional composite beams, shear carrying capacity is gained by mechanical connectors (to resist specially longitudinal shear), the most popular form being welded headed studs. The shear studs are welded to the flange of the steel beam, generally through a composite steel deck. A composite slab is cast on top of the deck with the stud, functioning to tie the slab and beam together as a unit. A composite beam has greater strength and stiffness than if the beam and slab were behaving independently. In CFST embedded composite trusses which are covered in this study, the shear carrying capacity is created by the surface area of the concrete-steel contact.

To design composite trusses with CFSTs in top chords, further experimental evidence is required on the shear carrying capacity at the composite stage. Two configurations using 114mm diameter pipes as the truss top chord were proposed for this study.

a) In configuration 1, the top chord was an embedded CFST in a composite slab where it acted as a continuous circular shear connector.

b) In configuration 2, headed shear studs were used for shear connection and the top chord of the composite slab was a CFST. However, it was not embedded into the slab.

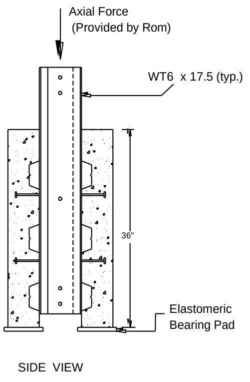
The proposal was to test specimens of the both configurations by varying the clear-cover thickness in the concrete and the concrete grade since the effects of those were not well known. The objectives of this study are: 1) to experimentally verify feasibility of the configuration suitable for top chord members of composite trusses; 2) to determine the effect of concrete top cover on shear transfer capacity of new deck slab configurations; 3) to determine the effect of compressive strength of concrete on shear transfer capacity of new deck slab configurations; and, 4) to identify the pattern of shear failure planes for each new deck slab configuration.

# 2. Longitudinal shear capacity prediction

The longitudinal shear force in composite beams is transferred across the steel flange/concrete interface at a discrete number of points by the dowel action of the individual shear connectors. If the concrete slab fails to resist the longitudinal shear stresses produced by connectors, longitudinal cracking along the line of the beam may occur. This leads to a loss of interaction between the steel beam and the concrete compression flange as well as a drastic reduction in the moment capacity of the composite section (Hicks and Mconnel, 1995).

Strength prediction equations have predominantly been derived from empirical studies. Both push-off (push-out) tests, which were first used in Switzerland in the 1930s (Davies, 1967), and full-scale beam tests have been used to develop shear stud strength prediction expressions. Because of the large size and expense of beam tests, push-off tests are usually used to evaluate a wide array of parameters. A push-off test specimen is shown in Fig. 1. Beam tests are often used to verify the results of methods developed from push-off tests. It has been found that push-off test results can be used to accurately predict beam test results if the push-off tests are detailed similar to the beam test (Easterling et al. 1993).

The property of a shear connector with the most relevance to design is the relationship between the shear force transmitted and the corresponding slip at the interface. This load-slip curve should ideally be found from tests on full-scale composite beams, but in practice a simpler push-off specimen is used (Fig. 1). The failure mode of shear connectors in composite slabs can also be found from the push-off tests.



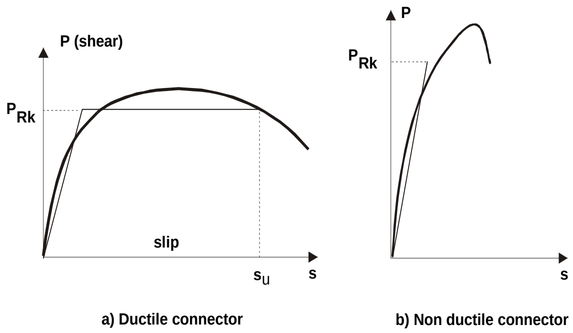




Figure 1: Typical details of push-off specimen    Figure 2: Standard connection behavior

According to past studies, surface cracks appeared along with separation of the concrete from the deck just before ultimate load was reached (Ollgaard et al. 1971, Elkelish and Robinson 1986, Oehlers and Coughlan 1986, and Hawkins and Mitchell 1984). After ultimate load was reached, the slabs were seen to ride over the sheeting and cause extensive profile distortion. Wedge-shaped failure cones, not pyramidal-shaped cones as suggested by Hawkins and Mitchell (1984), occurred around the studs in all of the tests. This mechanism has been found to occur in a composite beam test.

The longitudinal shear resistance ($Q_K$) can then be found as

$$Q_K = K\,\lambda\,(A_c\sqrt{f_{cu}}) \qquad (1)$$

where, $f_{cu}$ is concrete strength, $A_c$ is area of concrete, $K$ is shear friction factor, and $\lambda$ is factor for concrete type.

Shear connectors can be classified as ductile or non-ductile. Ductile connectors are those with sufficient deformation capacity to justify the simplifying assumption of plastic behaviour of the shear connection in the structure considered. Shear-slip curves are obtained by push-off tests. Fig. 2 shows examples of both ductile and non-ductile behaviour. A ductile connector has an elastic-plastic type of curve with a yield plateau corresponding to the connector characteristic resistance $P_{Rk}$ and to a high ultimate slip capacity $s_u$. Eurocode 4 (1994) considers that connectors having a characteristic slip capacity higher or equal to 6 mm can be assumed to be ductile, provided that the degree of shear connection is sufficient for the spans of the beam being considered.

# 3. Test program

## 3.1 General

In practice, in the Sri Lankan construction industry concrete grades with slabs are either grade 20 or 30. The most economical concrete top covers are 20mm and 25mm. Therefore, for each configuration, C20, C30, and C45 concrete was used with a concrete top cover of 25mm to check the effect of concrete grade on shear transfer capacity. Similarly, concrete top covers 20mm, 25mm, and 30mm were used with C30 concrete to check the effect of concrete top cover on shear transfer capacity. In each case three replicates were tested to verify the results. All together, thirty samples were tested for both configurations.

The proposed push-off test (push test) has a single span arrangement with only one deck slab specimen, as opposed to standard push-off tests described in Standard Codes of Practice (BS 5950-1994 and Eurocode 4-1994) for conventional composite arrangements where two identical deck slab specimens are fixed on either side of the main steel beam. The major reason for this is that the proposed configurations do not contain open flange steel beam sections. A considerable amount of material saving is also expected as a result of the proposed arrangement. A similar arrangement has been successfully used for push off tests in the past (Perera 2008 and Hicks and Mconnel 1995). The testing arrangement is illustrated in Fig. 3. The proposed single span push off test rig provides additional means of maintaining verticality of the deck slab specimens during loading, as shown in Fig. 3(b).

## 3.2 Test specimens

### 3.2.1 Configuration 1

In this configuration, shear transfer capacity of composite slabs relies on contact between the steel tube and concrete. In other words, friction force between the steel and concrete enhances the shear transfer capacity of composite slabs in configuration 1 which is different from conventional headed shear studs in configuration 2.

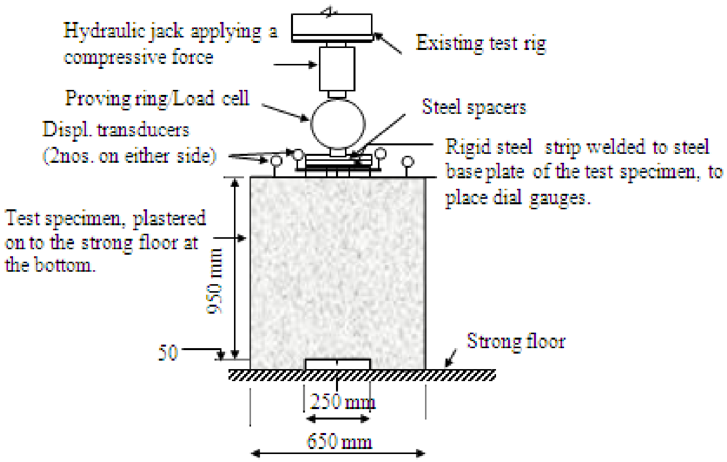
All push-off slab specimens were constructed using wooden forms. Each specimen consisted of CFSTs, each of which was welded to a 200mm wide x 950mm long x 9mm thick steel plate. Two “S” shape flashings, (25mm x 50mm x 20mm) riveted to two steel profile sheets, were riveted to a steel plate as shown in Fig. 4(a).

The CFST was welded to the steel plate prior to being filled with concrete. Then, it was concrete test cubes were cast at the same time as it was filled by concrete. Concrete test cubes were placed without curing (to check behaviour of the compressive strength of concrete inside the steel tube), but with proper covering.

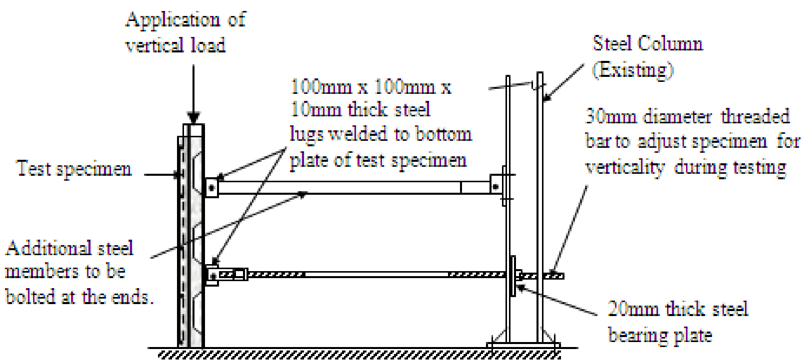
Two layers of steel reinforcement (R6@200c/c) were placed on top of the steel tube. Mortar cover blocks were used to support the reinforcement and silicon was used to fill the openings. All slabs were cast horizontally. A mechanical vibrator was used to vibrate the concrete after it had been placed in the forms.

The specimens were covered and moist-cured for seven days, at which time the forms were removed. Concrete test cubes were cast along with the specimens and cured similarly.

The push-off specimens were tested 28 days after being cast. At that time both concrete test cube samples were tested.




(a). Elevation of the General Test Assembly.




(b). Side Elevation of Test

Figure 3: Test arrangement

### 3.2.2 Configuration 2

In this configuration, conventional headed shear studs (19mm dia., 80mm height) were used to improve the shear transfer capacity of composite slabs (Fig. 4(b)).

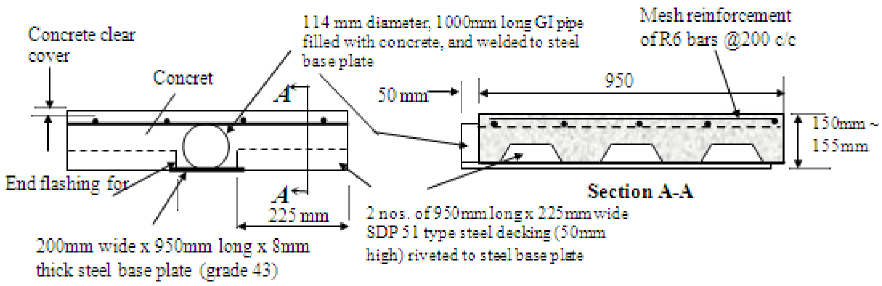
### 3.3 Testing procedure

The load was first applied in increments up to 40% of the expected failure load and then cycled 25 times between 5% and 40% of expected failure load. Subsequent load increments were then imposed such that failure did not occur in less than 15 minutes. The longitudinal slip between each concrete slab and CFST was measured continuously during loading or at each load increment. The transverse separation between the steel section and each slab was measured as close as possible to each group of connectors (Fig. 3).

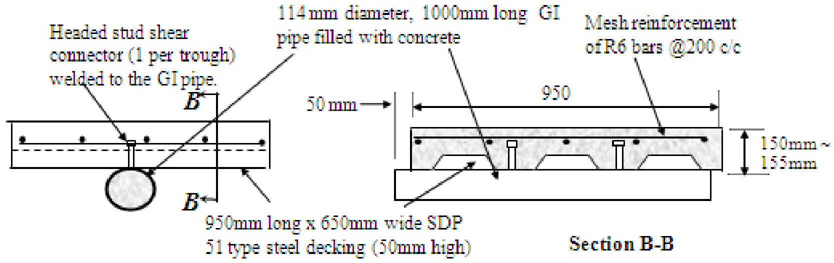
## 4. Experimental results and discussion

All push-off test results in each configuration are presented in Table 1 and Table 2 with average concrete-cube strength of four samples. In each configuration, concrete cube strength, and concrete top cover were used as variables.

During the analysis, the mean value of each variable was considered and outliers were neglected. For comparison, the averages of test results were used.




(a). Configuration 1




(b). Configuration 2

Figure 4: Different configurations of deck-slab specimens

Table 1: Configuration 1 test results

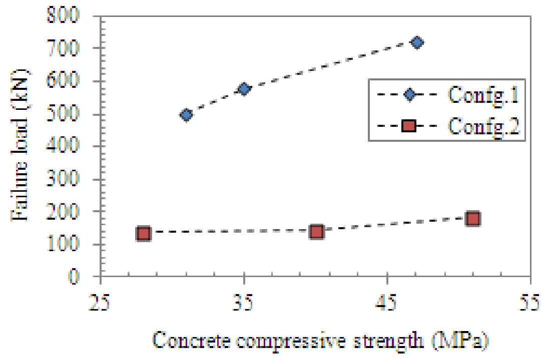
| Specimen | Design concrete strength (MPa) | Concrete top cover (mm) | Concrete cube strength (MPa) | Failure load (kN) |
|---|---|---|---|---|
| C30-20-1-i | 30 | 20 | 36.4 | 501 |
| C30-20-1-ii | | | 33.6 | 501 |
| C30-20-1-iii | | | 38.2 | 496 |
| C30-25-1-i | 30 | 25 | 34.5 | 569 |
| C30-25-1-ii | | | 35.3 | 589 |
| C30-25-1-iii | | | 26.8 | 517 |
| C30-30-1-i | 30 | 30 | 25.3 | 444 |
| C30-30-1-ii | | | 32.3 | 527 |
| C30-30-1-iii | | | 41.7 | 678 |
| C20-25-1-i | 20 | 25 | 32.0 | 501 |
| C20-25-1-ii | | | 32.2 | 428 |
| C20-25-1-iii | | | 30.8 | 574 |
| C45-25-1-i | 45 | 25 | 45.1 | 772 |
| C45-25-1-ii | | | 48.3 | 933 |
| C45-25-1-iii | | | 48.2 | 678 |

Table 2: Configuration 2 test results

| Specimen | Design concrete strength (MPa) | Concrete top cover (mm) | Concrete cube strength (MPa) | Failure load (kN) (one stud per rib) |
|---|---|---|---|---|
| C30-20-3-I | 30 | 20 | 40.0 | 121.2 |
| C30-20-3-ii | | | 39.8 | 121.2 |
| C30-20-3-iii | | | 36.9 | 128.3 |
| C30-25-3-i | 30 | 25 | 44.3 | 151.1 |
| C30-25-3-ii | | | 37.8 | 131.2 |
| C30-25-3-iii | | | 39.2 | 149.6 |
| C30-30-3-i | 30 | 30 | 32.8 | 135.4 |
| C30-30-3-ii | | | 38.6 | 156.7 |
| C30-30-3-iii | | | 36.4 | 155.3 |
| C20-25-3-i | 20 | 25 | 27.5 | 149.6 |
| C20-25-3-ii | | | 25.0 | 125.5 |
| C20-25-3-iii | | | 31.1 | 168.1 |
| C45-25-3-i | 45 | 25 | 51.7 | 192.2 |
| C45-25-3-ii | | | 51.1 | 168.1 |
| C45-25-3-iii | | | 49.8 | 192.2 |

## 4.1 Effect of concrete strength on shear connectors

The effect of concrete strength on the strength of shear connectors was examined in this study. Two configurations were used: configuration 1 and 2 as shown in Fig. 5. With an increase in concrete strength, the failure load for configurations 1 and 2 increased 16% to 45% and 4% to 33% respectively. Further, configuration 1’s shear carrying capacity was nearly four times the configuration 2 capacity.



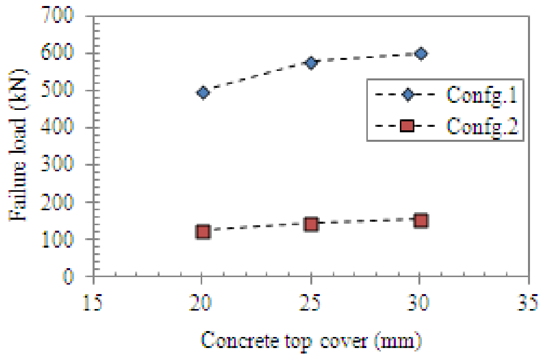




Figure 5: Effect of concrete strength on failure load on different configurations (with average test results)

Figure 6: Effect of concrete top cover on failure load on different configurations (with average test results)

### 4.2 Effect of concrete top cover on shear connectors

The effect of concrete top cover on the strength of shear connectors was checked in this study and it was checked for configuration 1 and 2 as shown in Fig. 6. The failure load increased 16% to 21% and 16% to 26% with increase of concrete top cover for configuration 1 and 2 respectively. The shear carrying capacity of configuration 1 was nearly four times that of configuration 2.

### 4.3 Behavior of shear connector

For configuration 2, which had a shear connection with headed shear studs, the plots against slip and failure load displayed behaviour like that shown in Fig. 2(a). However, configuration 1's behaviour was similar to that in Fig. 2(b) type behaviour. Therefore, it can be stated that configuration 2's shear connection is ductile and configuration 1 has a non-ductile shear connection (Fig. 7).

### 4.4 Shear failure pattern with each configuration

The shear failure pattern of configuration 1 is shown in Fig. 8; the longitudinal crack was propagated on top of the embedded steel tube. The failure of the composite slab was mainly due to loss of friction force at the steel concrete interface.

The shear failure pattern of configuration 2 is shown in Fig. 9. A longitudinal crack and transverse cracks propagated on top of the headed shear studs. The failure of the composite slab was mainly due to the pulling out of the studs, in a process is known as shear cone failure (Wedge cone failures were seen).

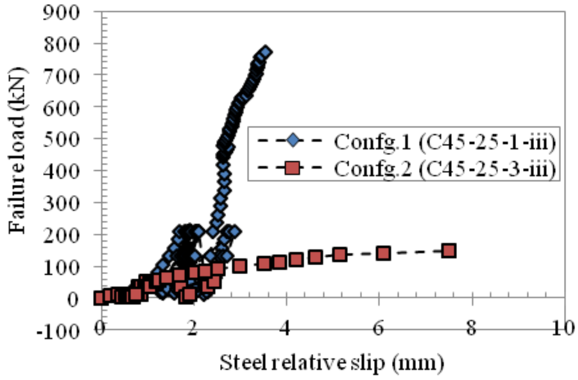



Figure 7: Experimental connection behavior

Figure 8: Configuration 1 Shear failure pattern

Figure 9: Configuration 2 Shear failure pattern

## 4.5 Prediction of shear failure load

For analysis, concrete top-cover 25mm test results were used since more results were available for cover 25mm compared with the two other covers.

Eq. (1) was modified to suit two configurations [see Eq. (2)]. Concrete area for configuration 1 was taken as the surface area of a steel tube (331,414mm$^2$) and for configuration 2 it was taken as wedge shear cone area (227,334mm$^2$) (Lloyd and Wright 1990 and Rambo-Roddenberry 2002).

$$Q_K = (A_c\sqrt{f_{cu}})^{n} \qquad (2)$$

where, n = 0.44 for Configuration 1 and n = 0.35 for Configuration 2

Predicted failure load was compared with experimental failure load as shown in Fig. 10. The average ratio of tested to predicted shear capacity of configuration 1 and 2 specimens were 1.00 and 1.05 with a standard deviation of 0.20 and 0.14 respectively.

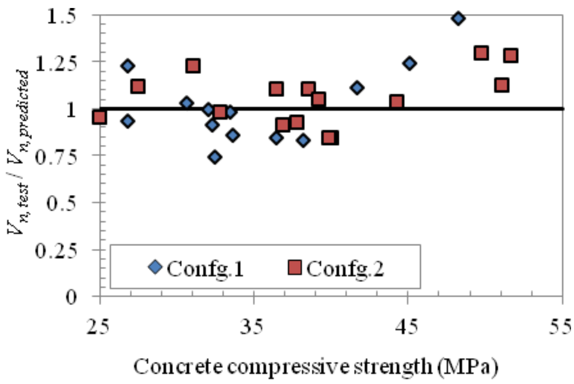



Figure 10: Comparison of experimental results with predicted values

## 5. Conclusion

The shear carrying capacity of configuration 1 and 2 increased with concrete cube strength and concrete top cover. The shear carrying capacity of configuration 1 was nearly four times the configuration 2 capacity. With configuration 1, a 16%- 45% increase in shear carrying capacity was achieved by increasing concrete cube strength from grade 20 to grade 45. Also, a 16%-21% increase in shear carrying capacity was achieved by increasing concrete top cover from 20mm to 30mm. With configuration 2, a 4%- 33% increase in shear carrying capacity was achieved by increasing concrete cube strength from grade 20 to grade 45. Also, a 16%- 26% increase in shear carrying capacity was achieved by increasing concrete top cover from 20mm to 30mm.

The connector type for configuration 1 was non-ductile whereas it was ductile in configuration 2. The shear failure pattern propagated at minimum top cover in configuration 1. One longitudinal crack was seen on top of the steel tube in configuration 1, and shear cone failure was seen in configuration 2. Therefore, Configuration 1 is more suitable for composite floor truss top chord compared with configuration2.

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