CHAPTER 1

INTRODUCTION

1.1 Background

Composite construction using steel and concrete has been used since the early 1920s. It gained widespread use in bridges in the 1950s and in building in the 1960s in the world. The reason for that is structural designers have been aware of the advantages of composite construction, such as saving of steel, reduction of overall structural depth, and the increase in floor stiffness and load capacity.

Composite construction in buildings has become more popular with the profiled steel sheeting since it serves as a working platform to support the construction loads and permanent formwork for the concrete. This eliminates the need for traditional, temporary forms and falsework. Also the sheets are suitably shaped to ensure proper bond with the concrete, the sheeting can provide all or part of the main tension reinforcement in the slab.

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 and several different configurations of composite floor systems are in use worldwide for long spans, namely, composite stub-girders, slim floor systems, composite trusses, composite beams with web openings in the steel beam, etc.

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 span (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 facility 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 more than adequate amount of structural steel fixed to its top chord for the serviceability and ultimate design states. An economical design may hence 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 the top chord of these floor trusses, one such alternative is to use a concrete filled steel tube, as described in this study.

The use of hollow steel tubes filled with concrete has become wide spread in the past few decades. This is mainly due to their properties such as high strength, high ductility, and large energy absorption capacity. In this type of composite trusses the uncertainty is with the shear connection 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 unit. A composite beam, which is shown in Figure 1.1b, has greater strength and stiffness than if the beam and slab were behaving independently. In concrete filled steel tube (CFST) embedded composite trusses which is suited in this study, the shear carrying capacity is gained by surface area of concrete-steel contact.

When studs or any other shear connectors are used in a design, one must be able to predict their ability to resist the longitudinal forces that arise between the steel and concrete. Hawkins and Mitchell (1984) expressed very well how difficult it is to predict the strength of studs by stating, “The analysis of the actions of an embedded stud shear connector near failure is very complex due to the inelastic deformations in the stud under the combined effects of shear, bending and tension and due to the inelastic deformations in the concrete surrounding the stud.”

Strength prediction equations have predominantly been derived from empirical studies. Both push-out tests (push-off test, push 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-out tests are usually used to evaluate a wide array of parameters. A push-out test specimen is shown in Figure 1.2. A photo of a push-out test is shown in Figure1.3. Beam tests are often used to verify the results of methods developed from push-out tests. It has been found that push-out test results can be used to accurately predict beam test results if the push-out tests are detailed similar to the beam test (Easterling et al 1993).

Early shear stud strength prediction equations, developed in the 1960s and 1970s, were for solid slab construction. The equations were developed based on the results of push-out tests. The equations were modified for the use of steel deck in the late1970s and were based on full-scale beam tests (Grant et al 1977).

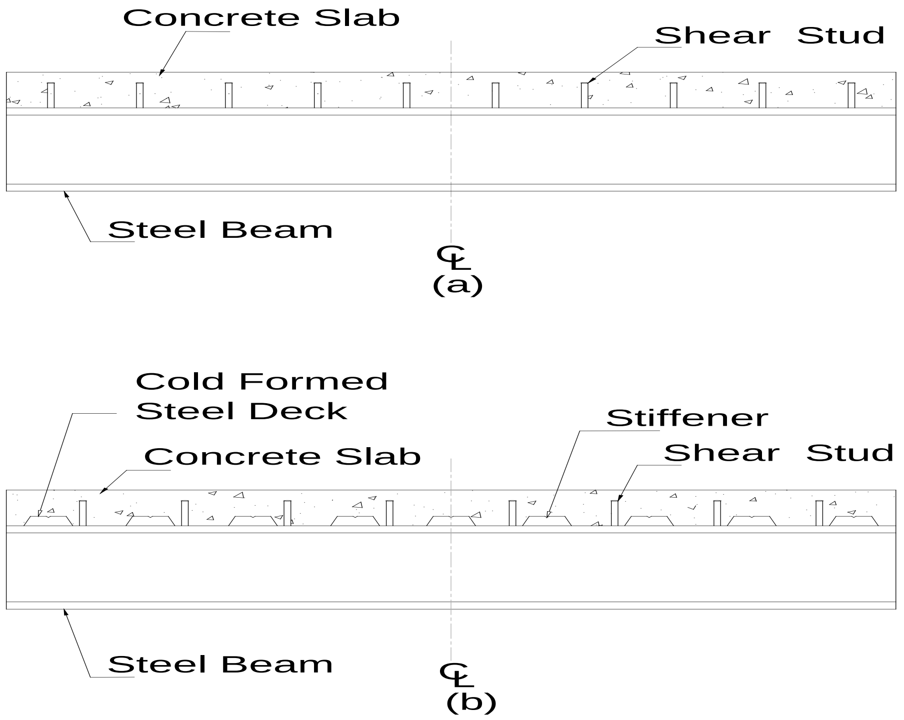


Fig. 1.1 Composite Beams Using Shear Stud Connectors:
(a) Non-Composite Solid Slab (b) Composite Slab Using Steel Deck

Typical steel deck profiles used in many countries have stiffeners (see Figure 1.4) in the center of the deck flanges. This results in the need for the studs to be placed off-center. The side of the stiffener that the stud is welded on (either toward the nearest end of the beam span or toward the ceterline of the beam span) affects the strength of the stud (see Figure 1.4). The stud is in the “strong position” if it is placed nearer to the end of the beam span, and in the “weak position” if it is placed nearer to the middle of the beam span (for more details see Chapter 2). The stud strength equations (Grant 1977) were developed from tests mostly using deck without stiffeners where the studs were welded in the center of the deck rib. The position of the stud in the rib is not considered in the present strength equations used in the Eurocode 4 and American Institute of Steel Construction (AISC) specifications.

Experimental research performed in many countries have shown that the existing shear prediction equations that are included in both Eurocode 4 and AISC specification are not conservative.

Figure 1.2: Typical Details of Push-Out Specimen

Figure 1.3: Typical Push-Out Test of a Composite Slab with Steel Deck (Rambo-Roddenberry, M. D. 2002)

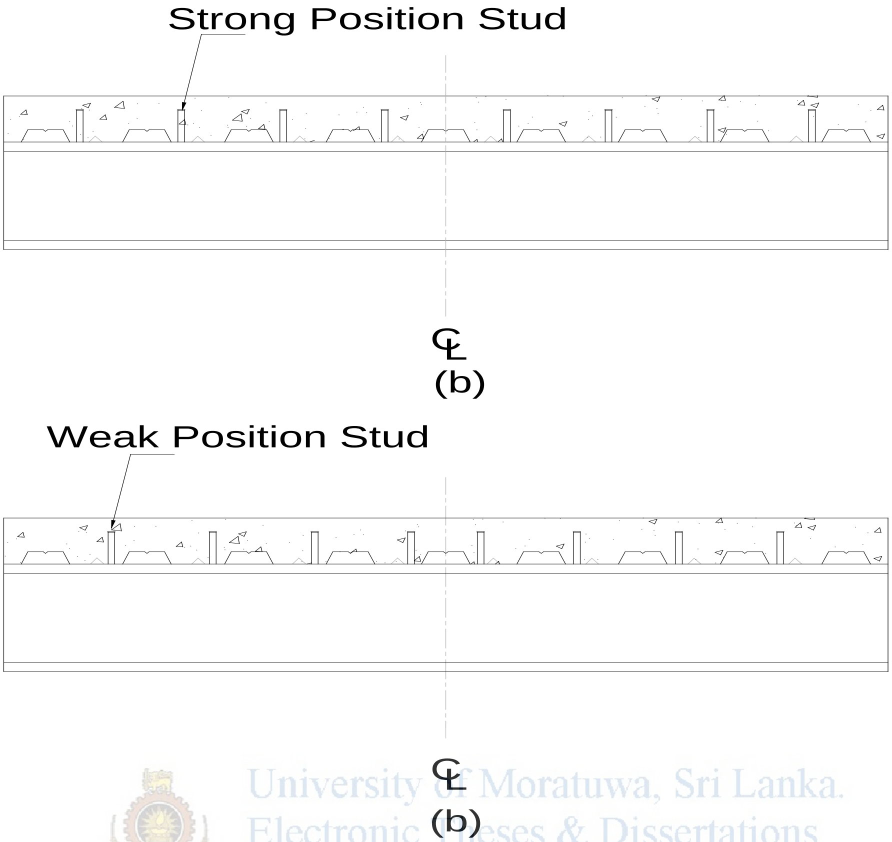


Figure 1.4: (a) Strong Position Stud in a Composite Beam, (b) Weak Position Stud in a Composite Beam

1.2. Longitudinal Shear Failure

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, and a drastic reduction in the moment capacity of the composite section (Hicks, McConnel 1995).

It is found that when transverse reinforcement is provided in a solid concrete slab, the cracking resistance of the slab is improved, such that longitudinal cracks develop when the yield stress of the reinforcement is reached. Therefore, a certain minimum amount of transverse reinforcement has to be used, to achieve the maximum load-carrying capacity of a composite beam.

As in B.S 5950: Part 3, when the profile steel sheeting is positioned transversely to the beam and, is continuous or adequately anchored, it is assumed that it effectively contributes as additional transverse reinforcement for the prevention of longitudinal shear failure of the slab. Conversely, in cases where the sheeting is discontinuous and not sufficiently anchored or, when the decking is positioned longitudinally to the steel beam, the code assumes that the sheeting does not act as transverse reinforcement and additional reinforcement bars are required (Brett 2001).

1.3. Push-Out Test

The most common way used to evaluate the shear connector strength and the behaviour is the push-out test. Push-out tests have been used as early as the 1930s and were used to predict the strength of studs in solid slabs as described above.

The property of a shear connector of 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-out (push-off)" specimen is used (see Figure 1.2). Failure mode of shear connectors in composite slabs could also be found from the push-out tests.

1.4 Project Objectives

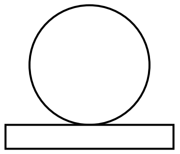
The project objectives are,

- To experimentally verify feasibility of different structural configurations suitable for top chord members of composite trusses
- To determine the effect of concrete top cover on shear transfer capacity of deck slab configurations.
- To determine the effect of compressive strength of concrete, on shear transfer capacity of deck slab configurations.
- To identify the pattern of shear failure planes for each of deck slab configurations.
- To determine the relevance of existing design methods given in standard codes of practice, for example, Eurocode 4 and BS 5950: Part 3, for the particular type of composite arrangement and to suggest new methods where appropriate.

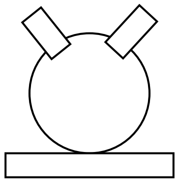
1.5 Methodology and Scope of Research

To design composite trusses with CFSTs in top chords, an extensive literature review was done. As a result, it is found that experimental evidence is required on the shear carrying capacity at the composite stage. Three configurations using 114mm diameter steel pipes, as the truss top chord were proposed for this study.

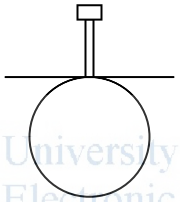
a) In configuration 1, the top chord was an embedded concrete filled steel tube (CFST) in composite slab and in composite slab it acts as a continuous circular shear connector.



b) In configuration 2, the top chord was a CFST with two steel strips welded to steel tube in composite slab and it acts as a different type shear connector to configuration 1.



c) In configuration 3 headed shear studs were used as shear connector and top chord of composite slab was a CFST but it was not embedded within the slab.



The objectives were to be achieved by testing specimens of three configurations by varying the clear-cover thickness in the concrete and the concrete grade.

1.6 Outline of Report

A historical overview of theories for the longitudinal shear strength capacity of composite slabs is discussed in Chapter 2. Past research on concrete filled steel tubes is presented in this chapter.

Chapter 3 describes the layout and materials for three configurations test specimens (specimens as a floor truss top chord) investigated.

Chapter 4 describes the results and includes comparison and discussion of push-out test specimens investigated (to check shear carrying capacity of floor truss top chord) in present study. The experimental results of those specimens are then compared with the existing theoretical models and modified models in Chapter 5.

The summary and conclusions of the present research programme (including most suitable configuration for floor truss top chord) are presented in Chapter 6. Possible areas for future work are also suggested in this chapter.