INVESTIGATIONS ON FLEXURAL CAPACITY OF STEEL CONCRETE COMPOSITE DECK WITH DIVERSE BOND PATTERNS

A Thesis submitted to Gujarat Technological University

for the Award of

Doctor of Philosophy

in

Civil Engineering

by

Merool Devarsh Vakil

Enrollment No.119997106012

under supervision of

Dr. Harshvadan S. Patel

GUJARAT TECHNOLOGICAL UNIVERSITY

AHMEDABAD

February 2017

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ABSTRACT

The steel-concrete composite deck is as an effective flooring option all over the world since last five decades. Recently these systems have started gaining popularity in India as well. This research focuses on: 1.Evaluation of flexural strength of composite deck system analytically based on International standards and parametric variations. 2. Investigations on the flexural strength experimentally with different bond patterns and their comparison by strength prediction procedures.

The evaluation consists of code based studies on European, British and American standards for flexural capacity and limiting geometrical & material parameters. Subsequently, parametric analysis is performed to further investigate effect of material and geometric parameter variations such as - steel grade, concrete grade, profile sheet thickness and concrete depth on flexural capacity and neutral axes. The study proposes guidelines for flexural capacity of composite deck as per Indian scenario. It also suggests neutral axis factor, to ensure under reinforced section theoretically. The guidelines will be useful for users in India, in absence of Indian code of practice for a composite deck.

The investigation comprises of experimental work on three wavelengths and one wavelength composite deck specimen with different bond patterns. Five analytical strength prediction methods from no bond to full bond are compared with experimental flexural capacity. The research demonstrates that one wavelength test specimen with ductile failure show good agreement with three wavelengths tests. Among investigated bond patterns, specimens with bolts head at interface have significantly improved composite interaction. One wavelength test specimen is proposed to verify the composite action experimentally.The specimen represents the bending behaviour and minimize the cost of experiments.Incorporating the bond properties, analytical strength prediction models are prescribed to verify the test results.

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Merool Devarsh Vakil

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Notations

h_c	Height of concrete
h_s	Height of steel
S_{max}	Maximum slip
L	Span length
M_{conc}	Flexural capacity of concrete
M_{steel}/ M_{tn}	Theoretical flexure capacity of steel deck
S_{ult}	Slip capacity of connector
ε_c	Strain in concrete at interface
ε_s	Strain in steel at interface
M_{comp}/ M_{tf}	Theoretical flexural capacity under full interaction
η	Degree of connection
T	Tensile force
C	Compression force
M_{Rd}	Design flexural resistance(EN)
x	Depth of the neutral axis(EN)
b	Width of Deck(EN)
A_p	Area of profile deck(EN)
f_{yp}	Yield strength of profile deck(EN)
f_{cd}	Compressive strength of cylinder(EN)
γ_p	Factor of safety for profile deck(EN)
γ_c	Factor of safety for concrete(EN)
z	Lever arm of stress block(EN)
V_{Rd}	Shear resistance of deck(EN)
b_0	Effective width of profile deck(EN)

Notations

d_p	Depth of profile deck(EN)
v_{min}	Minimum shear resistance of concrete(EN)
$V_{l,Rd}$	Longitudinal resistance of composite deck(EN)
Ls	Shear span
m	Empirical Parameter derived from experiments(EN)
k	Empirical Parameter derived from experiments(EN)
γ_{Vs}	Factor of safety (EN)
f_{cu}	Characteristics compressive strength of concrete cube (BS)
d_s	Effective depth (BS)
V_s	Shear-bond capacity (BS)
D_p	Overall depth of profile sheet (BS)
D_s	Overall depth of composite slab(BS)
m_r	Empirical parameter derived from experiments (BS)
k_r	Empirical parameter derived from experiments (BS)
B_s	Width of composite slab (BS)
V_v	Vertical shear capacity (BS)
b_a	Mean width of trough (BS)
b_b	Minimum width of trough (BS)
V_c	Design concrete shear stress (BS)
A_s	Area of steel deck (ANSI)
c	Distance from extreme compression fiber to composite neutral axis (ANSI)
d	Distance from extreme compression fiber to centroid of steel deck (ANSI)
f_c'	Specified compressive strength of concrete (ANSI)
F_y	Specified yield strength of steel deck (ANSI)

Notations

h	Nominal out-to-out depth of slab (ANSI)
M_{ru}	Flexural resistance an under-reinforced composite slab (ANSI)
M_y	Yield moment for the composite deck-slab, considering a cracked cross section (ANSI)
I_{cr}	Cracked section moment of inertia (ANSI)
y_{cc}	Distance from top of slab to neutral axis of cracked section (ANSI)
n	Modular ratio (ANSI)
V_n	One way shear strength of the composite deck-slab (ANSI)
V_D	Shear strength of the steel deck section (ANSI)
A_C	Concrete area available to resist shear (ANSI)
f_{ck}	Characteristics compressive strength of concrete cube (IS)
x_u	Depth of neutral axis (IS)
M_{tn}	Theoretical flexural capacity of steel deck (No bond)
I_{Deck}	Moment of Inertia of steel deck
y_{Deck}	Centroidal depth of steel deck
f_{Deck}	Strength of steel of deck
M_{ty}	Theoretical flexural capacity using first yield approach
T_1	Tensile force of the top flange
T_2	Tensile force of the web
T_3	Tensile force of the bottom flange
e_1	Moment arms for T_1
e_2	Moment arms for T_2
e_3	Moment arms for T_3
M_{tl}	Theoretical flexural capacity using Lutrell's approach
n	Length of embossment /bond

Notations

m	Spacing of embossment /bond
p_h	Height of embossment /bond
t	Thickness of profile deck
k_1, k_2,	Bond factors
k_3/ k_w	No. of wavelength factor
M_{tm}	Theoretical flexural capacity using Modified beam analogy approach
N_p	No. of connectors provided
N	No. of connectors required for full bond
M_{tf}	Theoretical flexural capacity with full bond

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CHAPTER-1

Introduction

1.1 General

Composite decks are efficient and economical floor systems to the building units. Since last five decades, the system is one of the prevalent construction practices in many parts of the world. Composite deck refers to a structural slab system created by compositely combining the structural properties of concrete and cold formed light gauge metal steel decking. This type of deck acts as a one-way slab in which steel sheet and concrete are so interconnected that the deck and concrete act together to resist bending in longitudinal direction as shown in Fig.1.1. The system is also referred as 'composite deck', 'composite slab' or 'composite floors'.

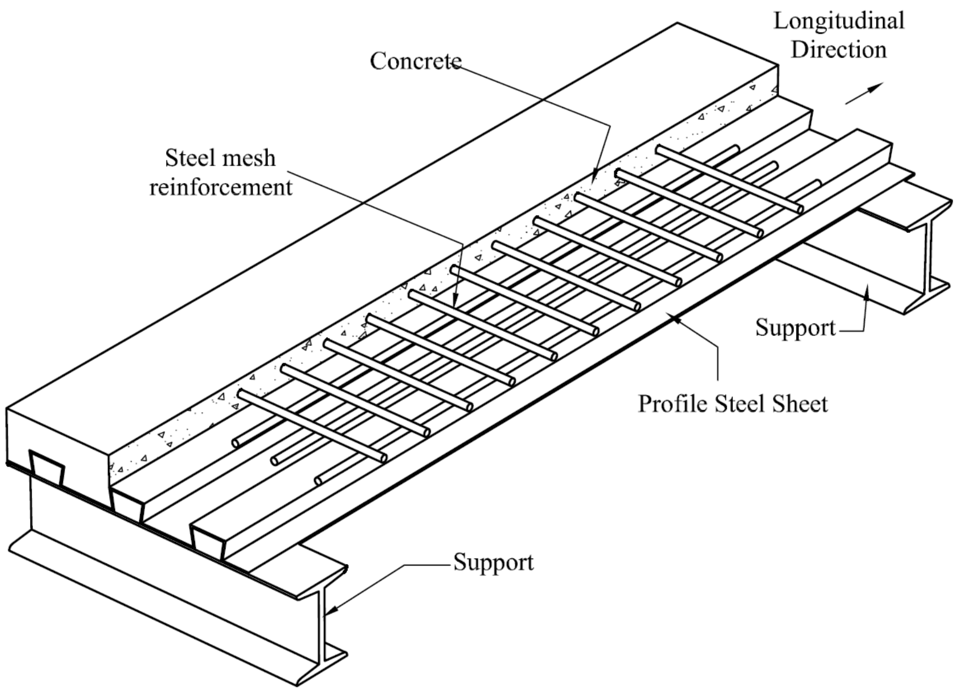


FIGURE 1.1 Steel Concrete Composite Deck

1. Introduction

Cold formed profile decks are used as a permanent form work for a composite deck system. These profile decks also act as tensile reinforcement, if the strength of profile steel sheet is utilized. Earlier, the composite deck system was considered as an optimum solution to the building floors for high-rise steel framed structures only but in recent years, it is becoming more popular for low - medium rise steel and R.C.C. buildings [1]. The reinforced concrete frame composite deck provides same benefits as composite decks for steel framing. Composite deck is a popular construction practice all over the world. But in India, the floor system is still at a nascent stage.

1.2 History

The first forms of steel decking were used to support concrete floors in 1920.In that particular system, the deck was a load-carrying structural element and concrete was used only to provide a level surface. After the World War II, metal deck systems were introduced that could function as stay-in-place forms used without shoring.[2] In a patent filed in 1926, Loucks and Giller had proposed a steel-deck system with further improvements. In this early growth, the metal deck was used to provide the structural resistance and concrete was poured to provide a walking surface and fire resistance. The use of steel deck provided a striking alternative to conventional R.C.C. slab as it served as a platform for workers and permanent formwork. By 1938, engineers were using a non-composite cellular floor system produced by the H.H. Robertson Company.[3]

In America, application of profiled steel sheeting as permanent formwork as well as reinforcement to the composite slabs was first developed commercially by Granco Steel Products Corporation in 1950[3]. In order to achieve composite action between the concrete and steel deck, the Inland-Ryerson company produced a trapezoidal steel deck with embossments on the profile in 1961.In 1968, the American Iron and Steel Institute (AISI) commenced a program for development of a general design procedure for composite deck systems. The composite deck system was then introduced in the United Kingdom in the year 1970 and has become the most common form of floor system. In today's practice, composite slab systems use metal deck rolled to form channels running in one direction. Mechanical interlock in form of Indentations, embossments, protrusion, transverse wires etc. are used to provide a better transfer of interface forces between deck and concrete.

1.3 Components of Profile Deck

The metal deck, also known as profile deck is one of the important structural parts of composite deck system. The section describes the stages of development of profile sheet from plain coil of steel.

1.3.1 Process of Cold Forming

Structural cold-formed profile shapes are produced from thin steel strips of commonly specified grades such as Grade S280 and S350. To improve ductility of the strip and to achieve higher strength to weight ratio, steel strips are cold formed from hot rolled steel coil by means of annealing process. By the process of cold forming and strain hardening, the yield strength of the steel is increased. In case of stiffened section as much as 10% to 30%, increase in yield strength is achieved by cold working.

The thickness of pre-galvanized cold-formed steel sheet for decking typically ranges from 0.7 to 1.5 mm and the width varies between 1 to 1.25 m. The plain sheets are then fed into a series of roll formers. The set of rollers moves in an opposite direction to form a desired deck shape (trapezoidal or reentrant) along the line of rolls. The number of rolls needed to form the finished shapes depends on the geometry of profile sheet. As an alternative to roll forming method, press-braking method is also popular for relatively simple shapes. The press braking method is generally used for small production up to short length of 0.6 m, in which press machines are used to bend the steel sheet to produce desired shape. In the case of roll forming, setting-up costs are very high as compared to press braking.[1]

1.3.2 Profiled Decking Types

The bottom surface of the composite deck is made up of corrugated cold formed steel sheets, which is popularly known as profile deck. There are two basic deck profile types: Trapezoidal and Re-entrant, as shown in Fig.1.2 (a) and Fig.1.2 (b). Trapezoidal deck sheet performs a composite action by means of indentations, embossments or mechanical interlock in the deck sheet, whereas Re-entrant profile interacts with concrete by means of frictional interlock.

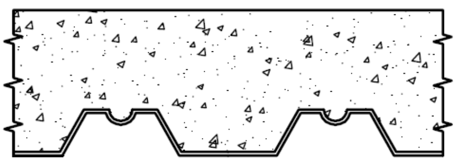


FIGURE 1.2 (a) Trapezoidal Profile

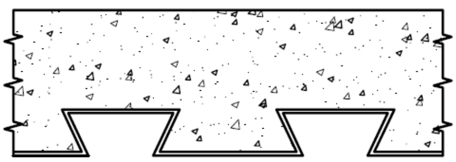


FIGURE 1.2 (b) Re-entrant Profile

1.3.3 Steel to Concrete Connection

To develop the required composite action between the concrete and steel deck, the steel deck must be able to resist longitudinal slip and vertical separation between the concrete and steel deck. Only adhesion between the steel sheet and concrete is not sufficient to produce proper composite action in the deck. An efficient connection [5] can be achieved by following means as depicted in Fig.1.3 (a) to Fig.1.3 (d).

a) Frictional interlock in Re-entrant trough profile
b) Mechanical interlock by indentations, embossments, protrusion, holes
c) End anchorage by welded studs
d) End anchorage by deformation of the ribs

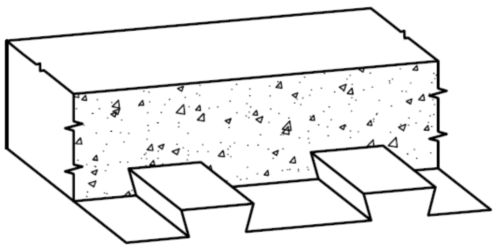


FIGURE 1.3 (a) Friction Interlock

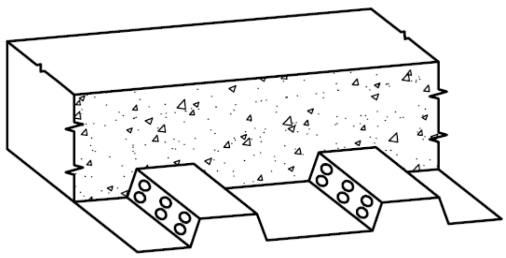


FIGURE 1.3 (b) Mechanical Interlock

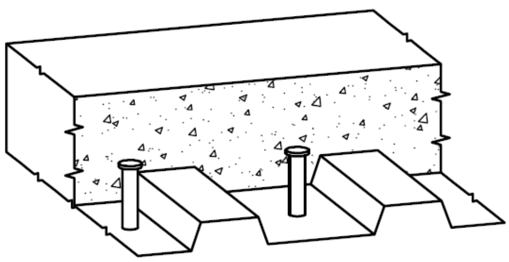


FIGURE 1.3 (c) Welded Stud

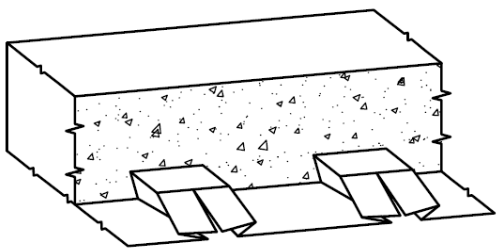


FIGURE 1.3 (d) Deformed Ribs

1.3.4 Embossment in Composite Deck

Among various forms of mechanical interlocks, embossment is one of the ways to provide composite action between steel and concrete. Different profiled sheeting product has different geometry of embossments which may be pressed or rolled. As depicted in Fig.1.4, these embossments have shapes such as horizontal, inclined, chevrons, staggered, rectangles and circles. The location of the embossments generally depends on the available areas to be pressed and quality of steel sheet material of the profiled sheeting. The depth or height of embossments is restricted from the point of view of energy requirement for the pressing process and to avoid tearing of the sheet.

Proper depth of embossment is only ensured by strict checking and quality control. Several problems during manufacturing can lead to a non-uniform embossing depth such as: poor roller setup, roller wear and inelastic behavior of the material. These can result in several problems such as 'uneven' or 'no' embossing depths. Excessively deep embossing can weaken the deck surface and lead to premature aging. Shallow embossing can trigger to the loss of composite action after construction, which can lead to serious issue from safety point of view.[6] Current products in the market do not provide the details regarding embossment dimension, spacing, depth etc.

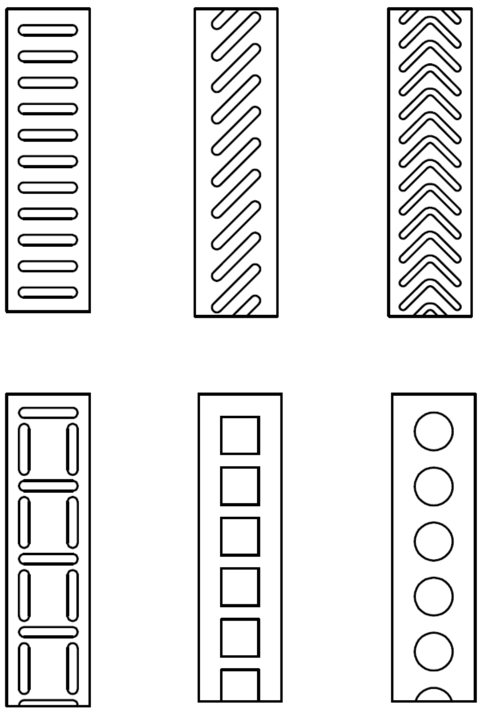


FIGURE 1.4 Patterns of Embossment in Composite Deck

1.4 Composite Deck Construction

The construction process of composite deck is different as compared to ordinary R.C.C construction. It is divided into several stages from the placing of steel deck to pouring of concrete. The sequence of the deck construction is discussed herein.

1.4.1 Installation of Profile Deck

Steel concrete composite deck works together with the concrete to make a firm, light-weight and cost-effective floor system. These decks are available in various profiles and thicknesses, out of which trapezoidal shape is most common. Profile metal decking is placed on the structural steel or on R.C.C. beam at predetermined points in the erection sequence. Metal decking is attached to structural steel, either by welding or by powder actuated tools and then a nail-like fastener is driven through the metal deck into the steel beam. Depending upon the available sizes, materials and grades, headed studs connectors are installed to create a strong bond between the steel beam and the metal deck. Welded wire fabric or rebar grid is laid on metal deck to control crack due to temperature and shrinkage. The process from, laying of the deck to the installation of reinforcement is shown in Fig.1.5 (a) to 1.5(c).

1.4.2 Installation of Concrete

Once the decking is installed at its place, concrete is poured on top of the composite metal decking. Generally pumping methods is used to pour concrete. If the span of the deck is large, propping should be used to reduce deflection due to wet concrete. An experienced concrete contractor should be employed for concrete work. Concrete should be deposited over supporting members first and then it should be spread towards the deck midspan. To avoid the effect of ponding, accumulation of concrete at a particular portion (generally in center) must be avoided. As the concrete hardens, it forms a composite connection with the metal decking. The concreting process is described by the photographs shown in Fig 1.5 (d) to Fig.1.5 (f).

FIGURE 1.5 (a) Decking Installation [54]

FIGURE 1.5 (b) Installation of Studs [54]

FIGURE 1.5 (c) Secondary Bars

FIGURE 1.5 (d) Concreting[54]

FIGURE 1.5 (e) Bottom view of Decking

FIGURE 1.5 (f) Finishing of Concrete

1.5 Advantages and Disadvantages of Composite Deck

The structural features and advantages of composite deck over conventional systems of reinforced concrete slabs make them very attractive to structural designers. Steel concrete composite system include following advantages:

1. Considerable speed and simplicity of construction.
2. Acts as stay-in-place formwork and offers an immediate working platform.
3. Reduction in construction time due to the elimination of formwork.
4. A sustainable system as 94% steel construction can be re-used or recycled [4].
5. Reduction in dead load compared to conventional concrete building.
6. Strict tolerances, as profile decks production is under controlled factory conditions.
7. Approximately 30% reduction in concrete because of use of corrugated steel deck, results into compact structural section and reduces dead loads to foundations.
8. Elimination of excessive amount of reinforcing steel.
9. Reduction in labour costs.
10. Ease of transportation and installation.

Among all, significant reduction of tensile reinforcement, sustainability and elimination of form work for concrete casting are three most important advantages. This is in contrast to the earlier practice of the steel deck-concrete floor, where the deck was used only as a form work.

1.6 Motivation of the Research

The steel-concrete composite deck is an effective flooring option to structural designers all over the world. Owing to booming economy, infrastructural development and fast track construction trend, recently these systems have started gaining popularity in India as well. The cost of steel per ton fell from $2000 (2007) to $600 (2016), which makes the composite deck system more affordable to the users/structural engineers in India.

Slabs are basically flexural members and full flexural strength can be achieved if there is proper composite action between steel deck and concrete. Composite interaction of the slab can be analyzed by performing large-scale experiments on slab specimen. The degree of composite action depends on quality of mechanical interlocking. Poor roller setup, roller wear and use of low ductility steel can lead to non-uniform depth, size and shape of the interlock as discussed in section 1.3.4. The accuracy of embossments also depends upon steel grade and its ductility, if the steel sheet used is not of standard quality it may raise questions about the effect of embossment to act as composites.

Furthermore, composite deck construction is at growing stage in India and there is no Indian code of practice for composite floor deck. Users are not aware of the role of bond and the details about bond patterns are not mentioned in product information. At some constructions projects, sheets without any mechanical interlocks/bond are used. Uncertainty about the quality of bond raise questions about the effect of bond to act as composite, which can lead to placement of more reinforcement than the required as shown in Fig.1.5 (c).Considering the above facts, motivations for the present study are as follows:

1. To provide a detailed study on International standards.
2. To understand the effect of different mechanical interlocks.
3. To provide analytical studies about the role of bond and its effect on strength.
4. To reduce the reliance on expensive and time-consuming large scale tests.

1.7 Objectives and Scope of Work

The research work presented here has manifold objectives. The objectives of this research are: To evaluate flexural strength of composite deck system analytically based on International standards and parametric variations. To investigate the flexural strength of composite deck experimentally with different bond patterns and their comparison by strength prediction procedures.

The research work includes: Studies on code based analysis for flexural capacity and limiting geometrical & material parameters under full bond. Estimation and comparison of flexural resistance as per Euro, British and American design codes of practices. Studies on parametric variations such as different materials, profiled sheet thickness and slab thickness. Experimental studies on three and one wavelength test specimens, considering series of line loads with different bond patterns. Analytical approaches considering the bond properties, from 'no bond' to 'full bond' cases.

The study proposes guidelines for flexural capacity, neutral axis factor and limiting parameters for composite deck as per Indian scenario. The experimental study recommends the bond pattern for better composite action and proposes one wavelength test specimen to verify the composite action. Analytical strength prediction models are prescribed to verify the test results.

1.8 Original Contribution by the Thesis

Most of the earlier investigations indicate the development of composite deck design in different parts of the world. But the composite deck design is not much explored in the Indian context. For geometrical parameters and design of composite slab with profile deck, no guidelines are available in Bureau of Indian Standards (BIS).No specific study has been performed for code based comparison and parametric variation. Moreover, most of the research work focus on the behavior of composite deck considering a full-scale test of the deck. However, variation in bond pattern with small scale bending test has not been attempted before. It is essential to acquire in-depth understanding about the composite deck before a design procedure is set up. Hence, a code based analysis as per relevant standards, parametric study, experimental program with varying bond patterns and analytical studies for flexural capacity is identified as a research gap and it is expected that the present study will contribute towards bridging this gap.

1.9 Limitations

The composite action at steel-concrete interface is questionable in most of the cases and details about the bond are not found in product information. One of the purposes of this research is to study effect of various bond patterns on strength and behavior of deck so as to suggest a bond pattern which can be simply implemented by Indian small scale industry and/or the local user without much cost escalation. However, its feasibility at large scale implementation requires further investigation and advanced construction technologies.

1.10 Organization of Thesis

The preceding sections outlined the introduction, motivation and objectives of the research work. This section presents the outline of the thesis. The remainder of the thesis is categorized into six chapters. Chapter 2 reviews analytical and experimental research on composite deck by previous researchers and critical review of literature. The behavior and design philosophy of composite deck are presented in Chapter 3. Study of International standards, parametric study and design software for flexural capacity considering full bond is explained in Chapter 4.Chapter 5 explores feasibility of new bond patterns as first phase of experimentation on three wavelength specimen. Further, experimental investigation on small-scale one wavelength tests and results are discussed in this chapter. An analytical study considering the composite action and comparison of results are summarized in chapter 6. Next chapter states summary of work and conclusions regarding the study. Major contributions and recommendations for composite deck construction as per Indian scenario and the scope of future research is also discussed.

CHAPTER-2

Review of Literature

2.1 Studies on Composite Deck

Steel concrete composite decks have started becoming popular in 1950's. Many researchers have investigated the behavior of composite deck, since the inception. Many aspects of the composite interaction between steel deck and concrete have been studied and reported in different parts of the world since 1964[3].Most of the studies relate to the behaviour of composite deck by large scale experimentation. Few studies are reported on analytical formulations and numerical modeling of composite deck.

The next sections review various research work reported on composite deck. The review is classified into two different components of research on steel concrete composite deck:
(i) Analytical and Numerical Studies (ii) Experimental Studies

2.2 Review of Analytical and Numerical Studies

Many researchers have performed an exhaustive mathematical analysis on the results of the composite slab-derived from numerical analysis or experiments. Different design and strength prediction methods were evaluated in many parts of the world. Review of analytical and numerical studies of composite deck are discussed below.

1. Johnson [7] (1975)

An extensive research on composite construction started by Chapman and Johnson led to publishing a book. The book describes the analysis, design and the standard guidelines as per European practice for the composite structures with a review of behaviour of composite structure of steel and concrete.

2. Luttrell and Prasannan [8] (1984)

Authors argued for the assumption that, in the flexure mode, the slab behaves as a reinforced concrete section and tensile force of steel deck section acts at its centroid. They pointed out that the behaviour of steel deck is different in profile sheet than in R.C.C. sections because the deck is only bonded on one surface and is free to deflect on the other surface. Therefore, the geometry of the deck has a great effect on the resistance. They developed equations for the moment capacity based on performance factors. However, the performance factors presented were cumbersome and authors stated that efforts are continued to simplify the factors.

3. Heagler [9] (1992)

Heagler devised flexural capacity equation based on transformed area. He considered separate tensile forces for profile sheet at the top and bottom flanges (T_1, T_3) and the web (T_2) separately. This procedure provides three tensile forces with their respective moment arms (y_1, y_3, y_2), This development was particularly advantageous for predicting the performance of a newly created deck. But the effect of interface topology was not considered in this method.

4. Daniels and Crisinel [10] (1993)

Daniels and Crisinel developed a special purpose finite element procedure using plane beam elements for analyzing single and continuous span composite slabs. The procedure incorporated nonlinear behavior of material properties, additional positive moment reinforcement, the load-slip property for shear studs (obtained from push out tests) and the shear interaction property between the concrete and the steel deck (obtained from pull out test).A nonlinear partial interaction finite element analysis was used to predict the behavior of the slab under load.

They gave some recommendations for the further research in composite deck as per following: Numerical analysis by finite element modeling, require number of assumptions and the use of a very sophisticated nonlinear, large-displacement finite-element analysis. Further, Pull-out and push-off tests should be standardized. More research should be concentrated on similar numerical models to calculate composite slab behaviors and maximum load-carrying capacities to increase the confidence with which modeling is used.

5. Wright, Veljkovic [11] (1996)

Authors studied composite slab design and discussed important observations regarding the mechanism of failure by finite element analysis. They concluded that at the beginning of the load history the mechanical interlocking in the shear span is the main contributor to the interaction resistance. Then, the effect of mechanical interlocking was reduced as tensile strains increase in the sheeting and the friction at support became more important in anchoring the sheeting. In the last phase, local buckling occurred causing a sudden drop of the slab resistance.

6. Tenhovuori and Leskela [12] (1998)

They studied the effects of bond failure on the behaviour of composite slabs with the profiled steel sheeting. The effect of different important parameters was considered and critical factors were revised using numerical data obtained from a non-linear calculation based on the finite-element method. A comprehensive study was carried out to compare current analytical methods for longitudinal failure. In extension to this research, Sebastian and McConnel (2000) used method of layered beams and layer approaches for modeling composite deck. The authors discussed that these methods are still under development but useful for demonstrating the behavior.

7. Michel and Frederic [13] (2004)

Authors proposed a new design approach for the prediction of composite slab behavior, which combines result from standard materials test and small-scale tests .The approach uses a simple calculation model to obtain the moment-curvature relationship at critical section of composite slab. Unlike other proposed methods, the calculation method described by the authors was not depended upon numerical simulations. The result obtained using the new design approach was verified by its comparison with large-scale tests.

8. Roger [14] (2006)

Design data for the longitudinal slip resistance of the composite slab are provided by manufacturers of profile sheeting. The data are based on tests which may not be in accordance with standards of the country. Roger stated that the safe load tables provided by the manufacturer for the particular type of configuration may not be strict as per code specifications. All the details about profile decks are not provided by manufacturers and use of sheeting outside the country of origin may require verification.

9. Ferrera, Marimon and Crisinel [15] (2006)

To understand the steel–concrete slip mechanisms and its dependence on geometrical and physical parameters, authors developed 3D non-linear FEM models to simulate the longitudinal slip mechanics of composite slabs. On the other hand, several pull-out tests, as well as reduced m–k tests, have been carried out in order to verify the results. Parameters such as the embossing slope, the retention angle, the surface friction conditions were found as significant parameters in slip resistance. Authors studied profile deck behaviour with embossment pattern as inwards embossment, outwards embossment, alternate inwards/outwards. The authors found that the sliding movement of steel sheet and concrete produces interaction forces located just on embossment ends because steel sheet bends while concrete remains straight.

10. Abdullah, Cole and Easterling [16] (2007)

The research was conducted to analyze steel deck-concrete composite using non-linear dynamic finite element analysis ABAQUS/Explicit module software by the authors. A three-dimensional finite element modeling was performed which incorporated, tensile brittle cracking of concrete, horizontal shear bond behavior between the concrete and the steel deck. Some of the input data were deduced from bending test. They developed the quasi-static analysis method which was capable of predicting the load-deflection behavior and the ultimate load of composite slabs.

11. Abdullah and Easterling [17] (2011)

Authors discussed the new experimental bending test and shear bond modeling of composite slabs. The analytical study was conducted to improve the existing partial shear connection design procedure. Experiments on elemental bending test were performed. The results of the study demonstrated that the elemental bending test was feasible as a replacement for the full-size test. They proposed improved PSC design procedure which was found to be comparable with the m-k method.

2.3 Review of Experimental Studies

Several researchers have investigated the behaviour of composite deck experimentally. Most of the research work has been focused on full scale test. Few studies have been done on small scale test on various configurations on steel concrete composite deck. A review of experimental findings is as follows:

1. Luttrell and Davison[18] (1973)

The investigation reported by the authors involved tests on 1.5" and 3" deep galvanized, painted and bare metal steel panels. The concrete slab depths were also varying from 3.5" to 6" inches. The tests were conducted under simple spans loaded with symmetric loading near the third points in span. The analysis was based on the limitation of stresses at the extreme fibers and on the consideration of flexure and bond failure.

Authors concluded that the vertical embossments are 50% more effective in slip resistance than the horizontal, when comparing vertical and horizontal embossments. Horizontal embossments resisted vertical separation but did not sustain much load after the chemical bond was destroyed. These horizontal bond patterns contributed little to composite interaction as compared to vertical bond patterns.

2. Porter and Ekberg[19] (1976)

Explanation about the use of cold-formed steel deck sections in composite floor slabs, loading at construction phase and full-scale test on deck was discussed by Poter et al. They proposed full-scale tests to determine the experimental strength. Authors discovered that, the prevailing failure mode is frequently bond failure if proper composite interaction is not achieved. From the compatibility of strains and equilibrium of internal forces, flexural capacities equations were also developed. The other necessary design considerations such as casting and shoring requirements, span-depth ratio and deflections relations were described in the research. For the deflection of deck, effective moment of inertia was considered as the average of the standard cracked and uncracked sections. Porter et al. also recommended factor "m" and "k" from the full-scale test on at least six composite decks, for predictions of the strength of slabs.

3. Schuster[20] (1976)

Schuster has reviewed different research work on composite steel deck reinforced slabs, which were presented by many authors. Schuster explained and classified various commercially available steel decks based on their means of developing slip resistance and on the pattern of mechanical interlock devices. The classification was into three categories: profiles that provide horizontal slip resistance capacity by virtue of mechanical interlock devices, profiles with a variable spacing of mechanical connectors and profile without mechanical connectors.

4. Stark[21] (1978)

Stark discussed the types of interlocking in profile sheet. He further stated that when the sheet is trapezoidal profiled, the indentations must be able to prevent separation. Composite action is dependent on the type of sheet, depth and number of indentations and the span of the slab. Tolerances in form and depth of the indentations may have a considerable influence on the composite action. Stark has experimentally investigated and classified ductile or brittle failure. As per his findings, brittle behavior occurs when the maximum flexural strength is reached soon after failure or slip initiates. The sustained load drops suddenly. A ductile slab, however, continues to sustain load even after slip initiates. The curvature of the slab increases and the steel and concrete components no longer have a common neutral axis.

5. Seleim and Schuster[22] (1985)

Authors evaluated the previously conducted 196 experiments and discussed the results of three different sets of experiments on composite slab which were: same deck thickness samples, varying deck thickness sample and samples with varying shear span length. Seleim and Schuster showed that no noteworthy influence is found on bond resistance due to compressive strength of concrete. They proposed an equation based on experimental results of composite deck- which includes the steel deck thickness as a parameter. Authors claimed that the presence of the steel deck thickness parameter can result in a reduction of up to 75% of the presently required number of laboratory performance tests. They also described the sequence of bond failure:

- Prior to cracking, the load is carried by both steel deck and concrete and bond is completely effective.
- With the increase in applied load, cracking initiates at the critical section. These increases the difference in concrete stress and steel stress and also in the bond stress.
- With further growth of cracks, the profile deck and concrete tend to detach, lessening the efficacy of the bond.
- Concrete critical span portion begins to slide with respect to the steel deck, since the disengaged bond devices are no longer active, resulting in end-slip.
- The degree of cracking is unacceptable and the shear span is completely separated from the deck at ultimate load.

6. Luttrell [23] (1986)

Luttrell compiled the experimental data of research work at West Virginia University and added 75 new tests to eliminate or minimize extensive testing. A set of strength formulas were presented for two commonly used embossing categories. The formulas depend on precise details of the deck panels, particularly on the lug dimensions. The author found that, as compared to unembossed profile, embossed profile decks failed more gradually. The embossed slab continued to sustain load after initiation of slip and addition of embossment increases strength and stiffness of profile deck. With the increased slip, higher web stiffness increases the overriding resistance.

Further, he stated that after concrete cracking, slabs with smaller depths and thinner steel panels begin the transfer the forces more gently. Furthermore he identified response types depending on the embossing types. In one of the types, the mechanical strength was found not much greater than the adhesive strength. So role of embossments were limited and that too, for prevention of vertical separation.

7. Jolly and Zubair[24] (1987)

Jolly and Zubair pointed out that, effective composite action is achieved by increasing the depths of bonds. Depth is the most prominent factor but it should be noted that higher depths of bond may cause tearing of steel deck during production. The location of bond should be carefully decided as the bond pattern at the junction of the web and flange is very difficult to produce. Moreover, such corner patterns do not show any improvement in terms of composite action. Authors showed that placement of bond pattern at the middle of web are optimum. They also found that upon loading, effectiveness of bond decreased in the tension flange zone as they continued to flatten. Whereas in compression flange, they behaved as initial deformations, which promote buckling.

8. Young, Easterling[25] (1990)

The research described by authors was an outcome of research program at Virginia Polytechnic Institute and State University. In this research, experiments were performed on continuous spanned composite decks. The authors evaluated the strength of the composite full scale slabs which were constructed to simulate actual field conditions. All the three supports- intermediate supports and end spans were constructed considering the actual field condition. The study was done with the focus on influence of adjacent spans and pours stop details by conducting six full-scale tests on composite decks. Comparisons between the test results and predicted strengths based on conventional reinforced concrete theory were made.

9. Porter[26] (1992)

Porter investigated composite steel deck slab by conducting number of experiments. Based on maximum strength concepts, he proposed design criteria for composite slabs and recommended design procedures. He set some parameters and constants for the slabs failing in bond failure mode. As per his investigations, a plot was made using the parameters, $V_u / bd\sqrt{f_c'}$, on y-axis and $\rho d/L'\sqrt{f_c'}$ on x-axis. To provide an equation for longitudinal slip resistance, he performed a linear line regression analysis to determine the slope (m) and the intercept (k) of the line. Based on these constants ' m' and 'k' and equation, longitudinal capacity of the composite deck can be found out. Furthermore, for deflection average of the composite moments of inertia of cracked and uncracked sections is considered as per Porter's research.

10. Daniels: Pull out test[27] (1993)

Authors described the detailed procedure of small scale "Pull-out test" to study behaviour of composite deck. In specimen preparation , decking is cut one rib width wide, with an additional 50 mm of extra material at each longitudinal edge. Specimens are normally 400 mm long, including 100 mm of decking that remains unconcreted. A steel plate is then placed between the profiles and bolted together along both longitudinal edges. Pull-out test specimens consist of two halves placed back to back. Concrete is poured and compacted in the formwork after the profiles and steel plates have been assembled. The placement accounts for the initial erratic behavior immediately following reductions in shear resistance. Shear resistance that remains after chemical bonds have been broken on both sides of the specimen are assumed to be due to mechanical and frictional interactions. If the mechanical and frictional shear resistances are substantially lower than the initial chemical bonding shear resistance, brittle behavior is observed.

Transverse loads are applied constantly at the top and bottom of each side of the specimen. The total transverse load represents a minimum value for the real dead weight of the concrete slab on the decking. This force is resisted by support reactions placed at the top of each concrete block. The axial load is slowly increased using a displacement control. Measurements are made periodically of the axial load and the corresponding behavior.

11. Daniels and Crisinel [28] (1993)

Researchers reported about several important factors in designing a pull-out test procedure. The concreted length of the specimen must be large enough to contain a representative quantity of embossments, but not so long as to induce plasticization of the decking or nonlinear shear-stress distributions. The decking should be placed in tension to eliminate local instabilities, as decking is normally used as tension reinforcement for composite slabs in positive moment regions and ignored in negative moment regions. Transverse load, which represents the weight of the concrete slab and part of the applied load on the composite slab, should be applied to the specimen and controlled during the test to assure that it remains constant. Lateral movements at the longitudinal edges of the specimen must be eliminated.

12. Wright and Essawy[29] (1996), Makelainen and Sun [30] (1999)

Authors worked on different types of embossment on reentrant and trapezoidal profiles. They stated that for the investigated steel sheeting profile, the depth of embossments has more effect on resistance compared with the length and the shape of embossments. It was also found that re-entrant profiles improved performance by 63%-88%, and also unembossed deck of re-entrant type still provided 50% the strength of embossed deck. Increasing embossment length increased performance, but there seemed to be a length limit when improvement ceased. Great improvement in slip resistance is found in case of Penetrant embossments by the concrete that entered the holes.

13. Marimuthu, Seetharaman, Jayachandran, Chellappan, Bandyopadhyay, Dutta [31] (2007)

V. Marimuthu et al. have carried out an experimental study to investigate the bond behaviour of the rectangular embossed composite deck slab to evaluate the m–k values by conducting a two point load testing on eighteen composite slabs. They tested the slabs with different shear spans and found that for the shorter shear spans, the behaviour of the slab is governed by bond failure and if the shear span is large enough the behaviour of the slab is governed by flexural failure.

14. Lopes, Rui[32] (2008)

Authors studied the provisions of European standards for composite deck- Eurocode 4 and its drawback. They found that in m-k method or the partial connection method, chemical adherence is not accounted. Moreover, the application of these methods requires some fitting parameters that must be determined by full-scale tests. The author developed a method based on pull out test which does not rely on full-scale tests or on numerical modeling. This method is based on the determination of the moment-curvature relation of all composite slabs at critical sections.

15. Kurz [33] (2008)

They have experimentally investigated the behaviour of adhesively bonded joints between steel and concrete under shear, tension stressed and combined effects. To study the typical characteristics of adhesives in combination with steel and concrete, Kurz used different high-strength polyaddition curing adhesives such as polyurethane epoxy. The results showed a high load capacity for tensile stressed, shear stressed and combined stressed connections with adhesive.

16. Chen [34] (2011)

To study the behaviour of composite deck slabs, Chen tested seven simply supported one-span composite deck slabs, and two continuous composite slabs, using various end restraints in the simply supported slabs. The slabs that have end anchorage using stud connectors were found to afford higher bond strength compared to the case of slabs without stud connectors. The slip between the concrete and steel also reduced.

17. Holomek, Bajer [35] (2012)

They conducted four-point bending tests and vacuum tests on the whole span slab. The additional small-scale test was also carried out to understand the behaviour of thin-walled steel sheet in ultimate limit state. The data from the tests make possible to set up and calibrate numerical models. They concluded that small-scale tests represent an interesting alternative to expensive and time-consuming four-point bending tests, which are required in current standardized design methods. Its disadvantage is that they cannot include all the properties influencing the longitudinal slip resistance of composite slabs.

18. Lakshmikandhan, Sivakumar, Ravichandran, Jayachandran [36] (2013)

Authors investigated different connector assemblies to arrive at a better, simple interface mechanism. They developed three types of mechanical connector by means of reinforcement bars and concluded that, the composite slab without connectors, slips and fails at the earlier load level. By incorporating the connector, brittle behaviour of the composite slab can be modified into ductile. They stated that three external mechanical connector schemes which use reinforcement bars develop full interaction and do not show any visible delamination and slip.

2.4 Summary of Literature

Over the past five decades, several studies were carried out to understand the behaviour of composite deck by performing experimental investigations and numerical modeling on a number of full-scale and small-scale specimens.

Most of the earlier investigations were focused on six to eight full scale tests on composite deck specimen. The researchers that worked on large scale experimental tests were: Poter (1976), Seleim and Schuster (1985), Young, Easterling (1990), Marimuthu et al. (2007), Chen (2011), Lakshmikandhan et al.(2013).Their research was concentrated on different aspects of deck behaviour such as: development of empirical method for strength prediction, behaviour of simply supported and continuous composite deck, testing on new bond patterns and various end restraints of deck. Few studies demonstrated the behaviour of composite deck by small scale push off or pull out tests. The studies on small scale test were reported by Daniels and Crisinel (1993), Abdullah and Easterling (2007).

As per reported literature, the procedure for strength analysis of composite deck is based on m-k method or partial shear connection method which is then based on data from large scale testing. Some semi-empirical formulations were developed by Schuster and Ling (1980), Luttrell and Prasanan (1984) for strength predictions based on performance factors. Researchers Ferrera, Marimon and Crisinel (2006) , Abdullah, Cole and Easterling (2007) have also performed Numerical modeling using ABAQUS, DYNA and other FE packages but approaches are not satisfactory till date.

Jolly and Zubair (1987) stated that increased depth of interlock beyond certain point, can lead to tearing of sheet during. Wright and Essawy (1996) and Makelainen and Sun (1999) investigated that depth of interlock is very significant as compared to length and shape. Roger (2006) showed concern about quality and details of deck provided by the manufacturer and mentioned that use of profile deck outside the country of origin requires verification.

2.5 Critical Review

In order to review the composite deck design holistically, review of literature has been made covering analytical methods, numerical approaches, full scale as well as small scale testing procedures.

2.5.1 Critical Review: Analytical and Numerical Studies

Analysis of composite deck by numerical modelling is not at satisfactory stage till date. For numerical modelling also, input parameters of steel-concrete interface interaction are required, which in turn requires experimentation. Most methods of analysis rely on performance coefficients that are derived from large scale testing on steel concrete composite deck. Analytical studies on parametric variations and its effect on strength of composite deck are not reported in the literature. Guidelines about limiting geometrical parameters and material parameters are available in most of the International standards. However, specifications and limiting parameters of composite deck with profile sheet are not available as per Indian standards.

2.5.2 Critical Review: Full-Scale Test

Earlier investigations on the composite deck demonstrate that the major focus of previous research was onto development of composite deck sections and its behavior considering the full-scale test. The development of any new configuration of profile deck by the manufacturer may not be strictly in accordance with standards and/or its use outside the country of origin require further confirmation [14]. The experimental investigations reported on various research works were based on the large numbers of specimen having width variations ranges from 12" to 36".The bond between the steel deck and concrete depends upon parameters such as protrusion size and depth, deck profile, steel sheet thickness and concrete grade and type. Poor roller setup and roller wear lead to non-uniform protrusion depth, size and shape. As the protrusion details depend upon steel grade and its ductility, non-standard quality of steel may raise questions about the effect of protrusion to act as composite. Due to large scale testing with a number of tests, new pattern of embossments is difficult to generate. Moreover, effect of protrusion on strength is a complex phenomenon and cost of deck [36] increases by about 25% with embossment. Increased depth of the embossment leads to tearing [24]. In some cases, breakdown of bond will trigger failure in some of the profiles. The actual failure mode of the slab is complex involving a bond failure in the concrete, local yield or buckling in the steel deck and excessive amounts of slip displacement between the concrete and steel deck.

2.5.3 Critical Review: Small Scale Test

Few studies were reported on small-scale test on composite deck such as push off or pull out test. Earlier investigations on most of the small scale test methods were theoretically same as composite slabs test with shear loading acting on it. In actual slab the loading condition is complex and slab behaviour is interactive of bending and shear. This combined behaviour study is not addressed by the elemental test and thereby small scale test have limitations that the test condition is not the same as in actual slabs. The effect of one way bending, deflections, actual support conditions of the slab and other phenomenon coupled with bending cannot be simulated in the direct shear loading tests on small scale specimen. Furthermore, the fixing of steel sheeting to the test bed or to the opposite deck can exert constraint to the movement of steel sheeting and hindered the tendency of the sheeting to separate from the concrete naturally. The specimen is relatively complicated and difficult to construct. It was also found that slab slenderness and loading arrangement greatly influenced the behavior and strength of composite slabs. These factors are also not included in the existing elemental tests.

A small scale test procedure does not describe the actual loading condition of slab and the issued associated with the actual behavior are still not addressed thoroughly. The accuracy of the test depends on the slenderness of the slab, which cannot be replicated in the elemental push off or pull out test.

2.6 Concept in the Present Work

Review on analytical studies and experimental studies recapitulate that: Provisions for analysis and design are available in different International standards but no specific guidance is available on design of composite slab with profile deck in Bureau of Indian Standards (BIS). Most of the experimental studies on composite deck are performed on the full scale test. Experimental studies based on variation in interface topologies and elemental test are scant. Analytical consideration of composite action on behaviour of slab and flexural strength is also far and few.

2. Review of Literature

Looking to the above details on the composite deck, the present work has manifold intentions. The concept illustrated in the work, consist of analytical studies and experimental work on different bond patterns to achieve better composite action. The research includes code based analysis and comparative studies on flexural capacity of steel concrete composite deck under full bond. It comprises of development of design charts for parametric variations. Further, the studies include experimental work for bending tests on different wavelength composite slabs with different bond patterns. The work discusses various analytical theories for flexural capacity of composite deck considering effect of bond.

CHAPTER-3

Behaviour of Composite Deck

3.1 General

In design procedures of composite deck, concrete in the slab is assumed to resist only compressive stresses, while the metal deck resists tensile stresses depending on span and strength of the metal deck. If the deck is continuous across two or more supports, negative reinforcement may be required at the interior supports. To control shrinkage and temperature cracks, welded wire fabric or reinforcements are used. A simply-supported composite deck may fail by three principal criteria: Flexural, vertical shear and bond or longitudinal slip failure. A bond failure results in slippage between the concrete and the metal deck, which can result in cancellation of the composite action at interface. The chapter discusses the behaviour, failure modes and design criteria of steel concrete composite deck.

3.2 Assumptions in Composite Analysis

The behaviour and strength of composite deck depend on interlocking mechanisms, profile geometry and loading on the slab. The interlocking mechanisms are considered as combined contribution of profile geometry and interface bond or anchorages at the end. The assumptions made in composite analysis for simple-plastic rectangular stress block theory are as follows:

1. The concrete has zero tensile strength.
2. A uniform compressive stress develops in the top of the concrete slab.
3. Resultant tensile force in the sheeting equals the compressive force in the concrete.
4. The portion of the sheeting in tension is stressed uniformly to the yield stress.
5. The effects of vertical shear on the stress distribution can be ignored.

3.3 Behavior of Composite Deck

From the previous studies, it has been identified that three major modes of failure [19] can occur in simply-supported composite decks under bending as shown in Fig.3.1. i.e. Flexure failure at section A-A, vertical shear failure at section B-B and longitudinal slip failure at section C-C.

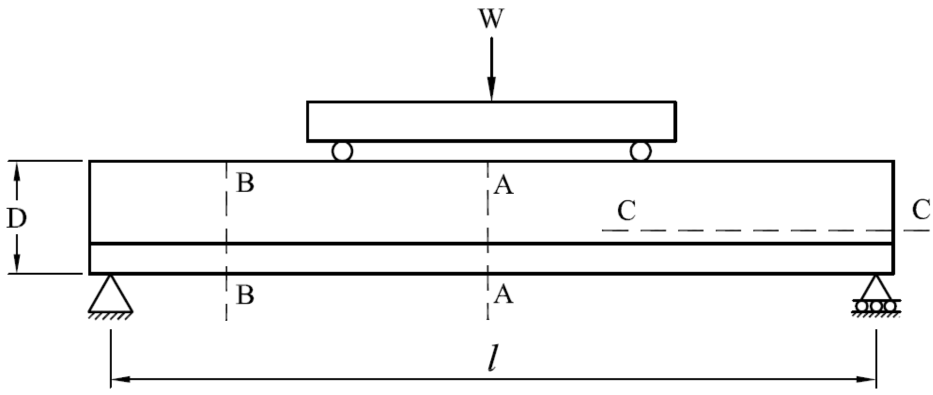


FIGURE 3.1 Behavior of Composite Deck

1. Section A-A: Flexure Failure

In this case, the slab will fail in a flexural failure mode. The maximum load is attained when at the critical section the optimum stress situation is reached. The full steel section can yield in tension before the crushing of concrete in the upper fibers. From equilibrium conditions, the situation can be reached when complete interaction occurs at the interface between concrete and steel. The flexural failure depends upon the steel and concrete interaction, which can be either complete or incomplete.

2. Section B-B: Vertical Shear Failure

Vertical shear failure is normally not critical in composite decks because decks are relatively slender elements. The characteristic of the vertical shear failure mode has been studied by Patrick and Bridge [37] (1994). This type of failure will occur near the supports in case of short and thick slab with a high concentrated load, which is not common in construction practice. Therefore, the effect is typically ignored in design.

3. Section C-C: Longitudinal Slip Failure

Horizontal slip failure or bond failure mode is likely to occur for composite slab if proper bond is not present. As depicted in Fig.3.1., this failure mode at 'C' occurs, depending upon the connection between steel and concrete when the slab is subjected to vertical loads. Longitudinal slip failure is recognized by the development of an approximate diagonal crack under or near one of the concentrated loads, followed by an observable end-slip between the steel deck and the concrete [38].In this case, the slab will fail in a longitudinal slip failure mode. The maximum moment depends on the degree or connection present at the interface between the concrete and the steel.

3.4 Degree of Interaction

Consider a simply supported composite deck of span *'l'* loaded at midspan. For the deck, height of concrete is considered as 'h_c' and height of steel is considered as 'h_s'. It can be seen from Fig. 3.2 that the maximum slip S_{max} occur at the supports, S_{max} increases as the degree of interaction reduces, and the span length '*l*' increases. The position of neutral axis and strain diagram varies as per no, full and partial interaction of the deck.

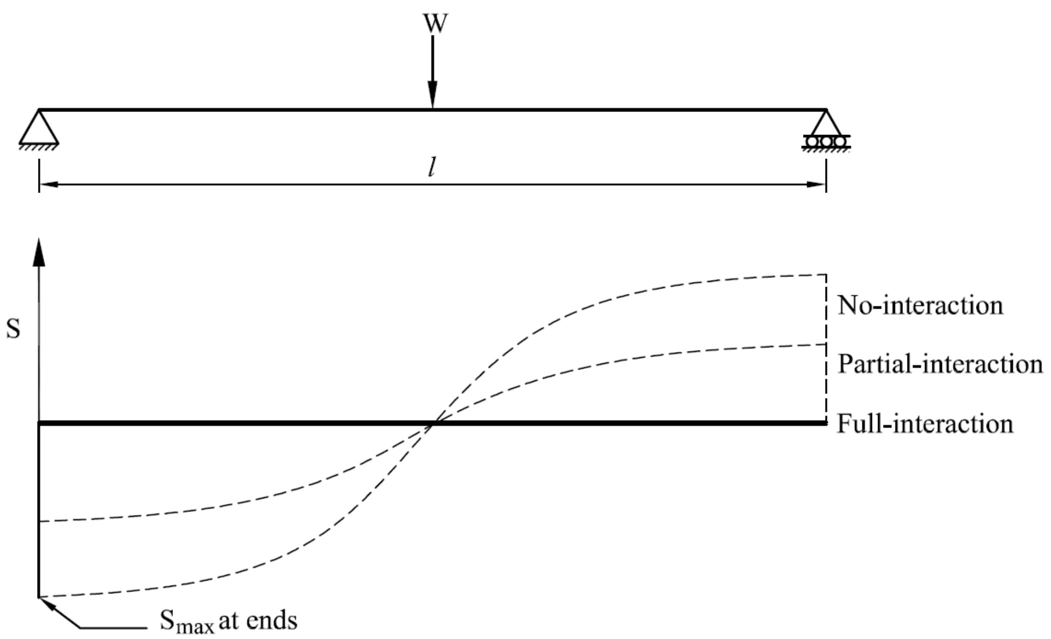


FIGURE 3.2 Slip in Composite Deck [39]

3.4.1 No Interaction: Case-A

If steel and concrete interface in the composite is oiled or made very smooth, there are no interface forces to restrict the interface slip. However, it is assumed that curvature between two elements remains same. There are two flexural members which have the same curvature and the external moments. These moments are resisted by pure flexure in the concrete M_{conc}, and by pure flexure in the steel slab M_{steel}. In no interaction, the strain profile is as shown in Fig.3.3 (a) with two neutral axes for steel and concrete.

3.4.2 Full Interaction: Case-B

If steel and concrete interface in the composite slab is glued then interface slip is totally prevented in composite slab on the application of external loads. The whole section will behave as monolithic section and there will be no slip and section will have one neutral axis. When there is full-interaction the strain profile as per Fig.3.3 (b) applies throughout the length of the slab, and hence, the slip strain is zero throughout.

3.4.3 Partial Interaction: Case-C

Partial interaction of a composite deck is a case between no interaction and full interaction. The degree of interaction is governed by the type and stiffness of the bond in composite deck and leads to failure when bond do not have sufficient ductility. The distribution for partial-interaction simply lies between these two extremes of no interaction and full interaction as depicted in Fig.3.3(c).

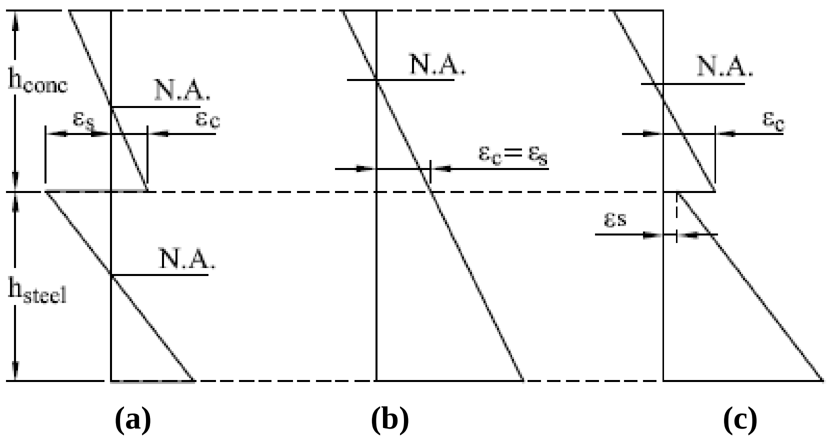


FIGURE 3.3 Strain Diagrams of Composite Deck [39]

3.5 Flexural Capacity of Composite Deck

The flexural capacity of a composite deck is limited by the finite slip capacity of the bond connection [39]. The slip in a composite deck depends on the degree of interaction, and degree of interaction is proportional to the degree of connection. Therefore for a composite slab of fixed cross-section, span and connector slip capacity S_{ult}, the moment at which fracture occurs M_{fr} varies with the degree of connection. The variation of the flexural capacity of the composite deck with the degree of connection is shown in Fig. 3.4. When there are no connectors, that is when $\eta = 0$, the strength of the composite deck is simply the flexural strength of the steel element M_{steel} or M_{tn}. The flexural capacity for full connection is given by full composite action M_{comp} or M_{tf} . Between these two extreme values, at moment capacity of the composite deck is given by the flexural capacity for partial connection M_i.

Consider a slab with a degree of connection η=i. As per Fig. 3.4.The slab will fail at M_i due to a lack of strength in the connectors before they have the chance to fracture at the moment M_{tf} due to the limited slip of the connectors. If the degree of connection is very limited then the deck can fail prematurely. At $\eta = 0$ the connectors will fracture at a moment less than M_{steel} (Moment capacity of steel deck), but this will not prevent the composite slab acting as a steel slab and achieving a moment capacity of M_{steel}.

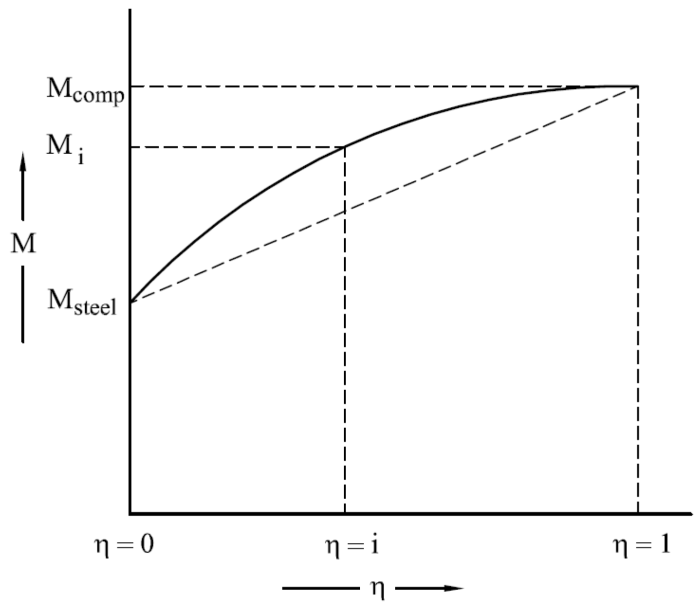


FIGURE 3.4 Degree of Interaction

3.6 Composite Deck Failure

Failure of the simply supported composite deck under vertical loads is discussed herein. The sequence of failure and regions of failure before crack and after crack are explained in this section.

3.6.1 Sequence of Failure

The sequence of failure [22] of composite slab with increasing loads occurs as follows:

1. Prior to cracking, load is carried by both steel deck and concrete and bond is completely effective.
2. Cracking initiates at the critical section, increasing the difference in stress of the concrete and the deck, which increases the bond stress and further increases cracking. The deck and slab begin to separate, lessening the effectiveness of the bond/protrusions.
3. Load transfer devices fail completely, resulting in end slip.
4. The degree of cracking is unacceptable and the shear span is completely separated from the deck.
5. A ductile slab continues to sustain load even after slip initiates, whereas brittle behavior occurs when the maximum flexural strength is reached soon after slip initiates and the sustained load drops suddenly.
6. The curvature of the slab increases and the steel and concrete components no longer have a common neutral axis.
7. Ductile specimens have significant post-slip capacity, while the brittle specimens show a large initial slip but then could not sustain additional load.

3.6.2 Regions of Failure

The regions of failure are divided into four categories as pointed out by Schuster and Ling[38], (1980) in their research. The mechanical interlocking capacity of composite slabs with the failure progress is as shown in Fig. 3.5.

3. Behaviour of Composite deck

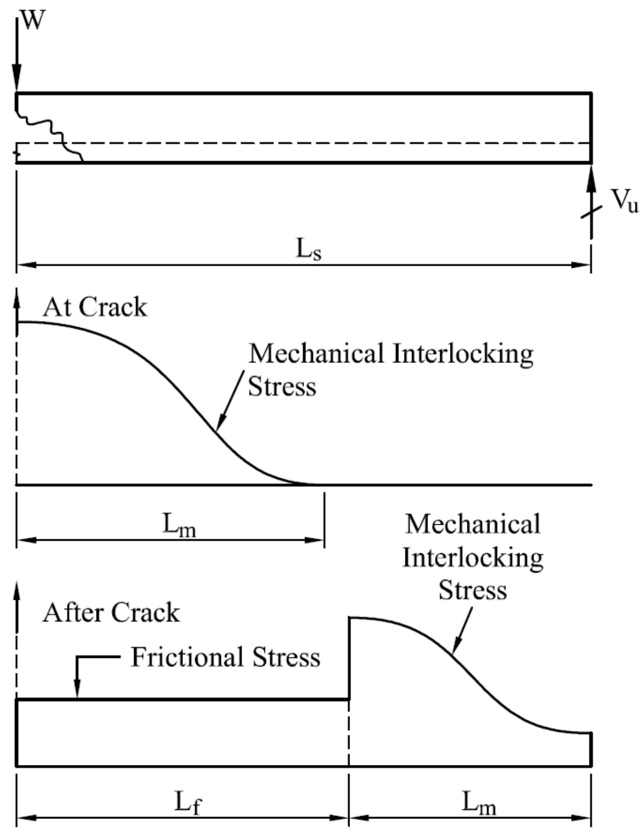


FIGURE 3.5 Regions of Failure

1. Before Crack

The concrete and steel deck act as a fully effective composite section, where the tensile bending stress is carried proportionally by both the concrete and steel deck. Hence, the resisting interlocking force between the concrete and steel deck is not active during this stage.

2. At Crack

In the immediate region of the crack, the mechanical interlocking devices begin to transfer load in the horizontal direction, causing the resisting mechanical interlocking force between the concrete and steel deck to become active in that region. If there are no interlocking devices in the steel deck (smooth deck), sudden failure of the system will result. However, with interlocking devices, the composite slab will only experience initial end-slip in the affected span and continue to carry additional load.

3. Behaviour of Composite deck

Resisting frictional forces are inherent with all interlocking-type composite slab systems after initial cracking has taken place. These frictional forces play particularly important role when early end-slip is being experienced. The magnitude of both the resisting mechanical interlocking and the frictional forces are dependent greatly upon the type of interlocking device and on the geometric profile of the steel deck.

3. After Crack

Immediately after the potential failure crack has occurred, the resisting mechanical interlocking capacity of the system in the vicinity of the crack has been exceeded. At the region of crack where the resisting mechanical interlocking capacity has been exceeded, resisting frictional forces starts acting, permitting the composite slab to carry additional load.

4. At Failure

The load carrying capacity of a composite slab is said to reach its maximum load when the combined resisting mechanical interlocking and frictiona1 forces reach their ultimate capacities within the failure critical span. Any additional load after this stage will cause the composite slab to fail, resulting in loss of composite action and large end-slip. However, ductile specimens had significant post-slip capacity and ability to carry much higher additional load.

3.7 Design as Per Euro Standard

The section describes analysis of steel-concrete composite deck as per Euro standard EN 1994-1-1,2004 [40]. Composite deck analysis for flexural resistance, vertical shear resistance, longitudinal slip resistance and deflections are mentioned herein.

3.7.1 Resistance to Flexure

The analysis of flexural resistance is carried out considering full interaction, which may be provided by protrusion or by re-entrant shape. The neutral axis normally lies in the concrete zone in case of full interaction.

3. Behaviour of Composite deck

The positive flexural resistance of a cross-section with the neutral axis above the sheeting as per the stress distribution is shown in Fig 3.6.The flexural resistance of width 'b' of composite deck is calculated by simple plastic theory.

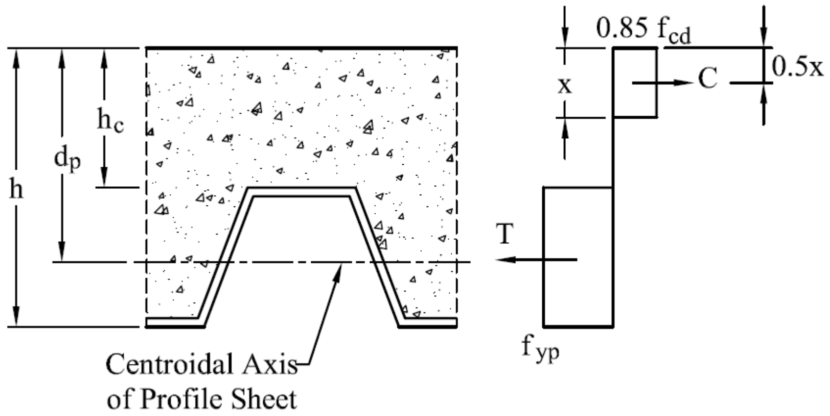


FIGURE 3.6 Stress Block of Composite Deck as per Euro Standard

In case of full connection, the design compressive force in the concrete, 'C' is equal to the yield force in the steel 'T'. Eurocode assumes the equivalent ultimate stress of concrete in compression as $0.85(f_{cd})/\gamma_c$ .Tensile force of steel deck and compressive force of concrete is calculated as per Eq.3.1 and Eq.3.2

$$T = A_p \frac{f_{yp}}{\gamma_p} \qquad \text{(Eq.3.1)}$$

$$C = b * x * \frac{0.85 * f_{cd}}{\gamma_c} \qquad \text{(Eq.3.2)}$$

By equating tensile force 'T' and compression force 'C', depth of the neutral axis and design flexural resistance is calculated as shown in Eq. 3.3 and Eq. 3.4.

$$x = \frac{\frac{Ap * f_{yp}}{\gamma_p}}{\frac{0.85 * b * f_{cd}}{\gamma_c}} \qquad \text{(Eq.3.3)}$$

$$M_{Rd} = T * z \quad \text{where } z = d_p - 0.5\, x \qquad \text{(Eq.3.4)}$$

Where, M_{Rd} = Flexural Resistance in kN. m/m

A_p = Area of profile deck in mm^2

f_{cd} = Compressive strength of concrete cylinder in MPa

f_{yp} = Yield strength of steel deck in MPa

x = Depth of neutral axis under full interaction in mm

b = Width of deck in mm

d_p = Effective depth of profile deck in mm

γ_c , γ_p = Material resistance safety factors of concrete and profile deck resp.

3.7.2. Resistance to Vertical Shear

Vertical shear failure is rarely governing in composite floor systems. Tests show that, resistance to vertical shear is provided mainly by the concrete ribs. For trapezoidal profiles, effective width 'b_0' should be taken as the mean width, at the centroidal axis as per Fig.3.6. For re-entrant profiles, the minimum width should be used. This shear resistance per unit width is given in Eq. 3.5 and Eq. 3.6, as per Euro standards EN 1994-1-1. Reinforcement contributes to the resistance only where it is fully anchored beyond the cross-section considered. The resistance of a composite slab with ribs of effective width b_0 at spacing 'b' is given by following equations.

$$V_{Rd} = \left(\frac{b_0}{b}\right) d_p v_{min} \qquad \text{(Eq.3.5)}$$

The recommended value for v_{min} is

$$v_{min} = 0.035 \left[1 + \left(\frac{200}{d_p}\right)^{1/2}\right]^{3/2} f_{cd}{}^{1/2} \qquad \text{(Eq.3.6)}$$

The expression $[1 + (200/d_p)^{1/2}]^{3/2}$ allows approximately for the reduction in shear strength of concrete that occurs, as the effective depth increases.

Where, V_{Rd} = Vertical shear resistance in MPa

b_o = Mean width at centroidal axis in mm

v_{min} = Minimum shear strength in MPa

3.7.3. Resistance to Longitudinal Slip

For profiled sheeting that relies on mechanical or frictional interlock to transmit longitudinal forces, there is no satisfactory conceptual model. Eurocode provides empirical ‘m–k’ method, which is based on the experimental parameters from six full scale test with varying shear span. The following Eq.3.7 is used to compute longitudinal resistance of slab with the safety factor added.

$$v_{lRd} = bd_p\left[\frac{mA_p}{bL_s} + k\right] / \Upsilon_{vs} \qquad \text{(Eq.3.7)}$$

Here 'm' and 'k' are constants with dimensions of stress, determined from shear-bond tests on six composite slabs with varying shear span 'L_S'. The values should exceed the vertical shear at an end support at which longitudinal failure could occur.

3.7.4 Deflection

Deflection is determined using the average value of the cracked and uncracked second moment of area. The deflection of the composite slab, excluding the self-weight is calculated using serviceability loads. The deflection of the profiled steel sheeting due to its own weight and the weight of wet concrete should not be included. The deflection of the composite slab should not normally exceed the following:

1. Deflection due to the imposed load: L/350 or 20 mm, whichever is the lesser.
2. Deflection due to the total load less the deflection due to the self-weight of the slab plus, when props are used, the deflection due to prop removal: L/250.

The limiting values for deflection criteria can be increased only if it is proved that the strength or efficiency of the slab is not affected or damaged by higher deflections. It should also not influence the finishes.

CHAPTER-4

Studies on International Standards and Parametric Variations

4.1 General

Most of the earlier investigations indicate the development of composite deck sections in various parts of the world. As far as the Indian scenario is concerned, composite deck construction is at growing stage. The current Indian standard for composite construction IS 11384-1985 (reaffirmed in 2003) does not cover the composite slab design with profile deck and it is not updated since long. No specific work has been done so far based on the comparative studies of existing standards and parametric variations. The purpose of the investigation is to provide the detailed studies on International standards and parametric variations for composite deck.

4.2 Objective of Code Based and Parametric Studies

The objective of this chapter is to discuss the highlights of three International standards for geometrical parameters and flexural capacity analysis considering the full interaction between steel and concrete. The objective is also to perform parametric analysis with variation in geometric and material properties.

The chapter deals with standard specifications and design procedure for flexural capacity considering British standard BS-5950: Part-IV:1994, Steel Deck Institute-ANSI: 2011 and Euro standard EN 1994-1-1:2004.The programs are developed to estimate flexural capacity as per various International codes & Indian standard stress block and parametric variations. Design charts for variations in geometric and material parameters are developed. The results of code based analysis and parametric analysis are presented in this chapter.

4.3 Composite Deck: Materials

Details pertaining to different materials used in steel concrete composite deck are provided herein. The properties and grades of various materials should be in accordance with the relevant standard adopted by that particular country.

4.3.1 Concrete

A typical strength class for concrete in Eurocodes and British standards is denoted by C25/30. The compressive concrete strengths used in the design as per Eurocode 4 are based on cylinder strengths. Strength classes are defined as Cx/y for normal weight concrete and LCx/y for lightweight concrete, where x and y are the characteristic cylinder and cube compressive strengths respectively. For example, C25/30 denotes a normal weight concrete with characteristic cylinder strength of 25 MPa and corresponding cube strength of 30 MPa. In the USA, concrete cube strength is used for design purpose. In India and UK also, concrete cube strengths are generally preferred. Normal weight concrete and lightweight aggregate concrete are having density of 2400 kg/m^3 and 1900 kg/m^3 respectively.

4.3.2 Structural Steel as Profiled Steel Sheeting

Profiled steel sheets used in composite decks are made of cold-formed steel sheeting, which exhibits highly nonlinear stress–strain characteristics. Ramberg–Osgood model [41] is often used to represent the stress–strain characteristics of cold-formed steel. The decks are designed to span in the longitudinal direction only. The specified or nominal yield strength of profile is that of the flat sheet from which the sheeting is made. In the finished product, the yield strength is higher at every bend and corner, due to strain hardening. Structural strength grades are specified as nominal yield strength and ultimate tensile strength. The grade of steel S 355, refers to yield strength of 355 MPa. Generally, the profile sheets are available with yield strengths (f_{yp}) ranging from 230 MPa to 460 MPa. The density of structural steel is assumed to be 7850 kg/m^3. Its coefficient of linear thermal expansion is 12×10^{-6} per ^{0} C. Designation S 280 GD + Z 275 means 280 MPa yield strength and 275 g/m^2 of zinc coating.

The sheeting is very thin, for economic reasons; usually between 0.8mm and 1.2mm. The design is based on the nominal thickness of the steel and the sheet must have at least 95% of that thickness. Dimples are pressed into the surface of the sheeting, to act as connectors. These dimpled areas may not be fully effective in resisting longitudinal slip so manufacturers are required to conduct tests on prototype sheets. Safe load tables are provided to designers with test-based values of resistance and stiffness.

The exposed surface on the underside of the profiled steel sheets should be adequately protected to resist the relevant environmental conditions, including those arising during site storage and erection. It adds approximately 0.04 mm to the bare metal thickness, 0.02 mm on each side. All zinc coatings should be chemically passivated with a chromate treatment to minimize wet storage stains (white rusting) and reduce chemical reaction at the concrete/zinc interface.

4.3.3 Reinforcing Steel

Standard strength grades for reinforcing steel is generally 460 MPa for ribbed bars, and 500 MPa, for welded steel fabric or mesh. These both types of reinforcement provide good bond action and ductility. The modulus of elasticity for reinforcement E_s is 200 kN/mm^2. Steel reinforcement, in the form of either bars or steel mesh fabric should be provided. The purpose of the reinforcement is to provide, nominal continuity reinforcement over intermediate supports in case of simple supports, full continuity reinforcement over intermediate supports in case of continuous spans, distribution steel in case of concentrated loads, secondary transverse reinforcement to resist shrinkage and temperature stresses and to increase the fire resistance.

4.4 International Standards: Overview

In the UK, standards are published by BSI under the designations BS EN 1990 to BS EN 1999; each of these ten Eurocodes is published in several parts and each part is accompanied by a National Annex that implements the CEN documents. Some provisions are added as per certain UK-specific provisions. British standard BS-5950 deals with the composite deck with profile sheeting.

4. Studies on International Standards and Parametric Variations

In America, the first Steel Deck Institute (SDI) was published in 1991, with a revised edition in 1997. Recognizing changes in technology, the SDI began activities to develop new standards for composite decks by initially publishing the ANSI/SDI C1.0 'Standard for Composite Steel Floor Deck' in 2006. 2006 Standard was revised and expanded in 2011 with the ANSI/SDI C-2011 'Standard for Composite Steel Floor Deck Slabs'. Over the past 80 years, the design of composite steel floor deck has evolved from empirical design based on testing into a product with well-understood behavior and mature design standards.

The Eurocodes are a set of structural design standards, developed by European Committee for Standardization (CEN) over the last 30 years, to cover the design of all types of structures in steel, concrete, timber, masonry and aluminum including code for composite construction EN 1994-1-1:2004.The Eurocode contain two distinct types of statement - 'Principles' and 'Application Rules'. The former must be followed, to achieve compliance; the later are rules that will achieve compliance with the Principles but it is permissible to use alternative design rules, provided that they accord with the Principles. All the material properties should be as per respective material standard unless specified.

Indian Standard IS: 3935 – 1966 Indian Standard 'Code Of Practice For Composite Construction' was adopted by the Indian Standards Institution in 1966 after the draft finalized by the Composite Construction Sectional Committee had been approved by the Civil Engineering Division Council. This standard deals with the design and construction of composite structures made up of prefabricated structural units and cast-in-situ concrete. The prefabricated units may consist of steel members or prestressed or reinforced concrete precast members. The another standard dealing with steel and concrete construction is IS 11384-1985, 'Code of Practice for Composite Construction in Structural Steel and Concrete' is published in 1985 and later reaffirmed on 2003. It stipulates that the steel-concrete composite structures may be designed by the limit state method. This standard deals with the design and construction of composite beams (simply supported) made up of structural steel units and cast in-situ concrete. Use of profile sheet as a structural member with concrete is not covered under this standard. Specification related to geometrical criteria of profile sheet and design details are not mentioned.

4.5 General Criteria: Geometrical and Material Parameters

The section discusses parameters related to material strength and geometry of composite deck as per British standard, American standard and European standard.

4.5.1 British Standard [42] (BS-5950-Part-IV-1994)

1. As per British standard, concrete grades C25/30 to C40/50 and LC20/25 to LC32/40 shall be used for normal weight and light weight concrete respectively.
2. The thickness usually ranges between 0.9 mm to 1.2 mm for commercially available profile deck. However, as per codal provisions, nominal bare metal thickness of the sheets shall not be less than 0.75 mm except where the profiled steel sheets are used only as permanent shuttering.
3. Common grades for steel strip in UK have yield strengths of 280 MPa and 350 MPa. Grades of steel for profiled steel sheeting are specified in BS EN 10326. The minimum yield strength of sheet shall not be less than 220 MPa.
4. For the design of the composite slab, the design strength of the profiled steel sheets shall be taken as 0.93 times the specified yield strength.
5. The modulus of elasticity 'E' of profiled steel sheets shall be taken as 210 kN/mm^2.
6. The zinc coating of 275 g/m^2 total, including both sides shall be normally specified for internal floors in a non-aggressive environment.
7. The total depth shall be not less than 90 mm and depth of concrete on top of profile shall be not less than 50 mm.
8. Transverse and longitudinal reinforcement shall be provided within the depth top of the slab with 25 mm nominal cover.

4.5.2 American Standard [43] (SDI-ANSI-2011)

1. The specified concrete compressive strength shall not be less than 3000 psi (21 MPa). The maximum compressive strength used to calculate the strength of the composite deck-slab shall not exceed 6000 psi (42 MPa). Minimum compressive strength (f'c) shall be 3 ksi (20 MPa) or as required for fire ratings or durability. Admixtures containing chloride salts shall not be used.

2. Composite steel floor deck shall be fabricated from steel conforming to the American Iron and Steel Institute, (AISI Specifications) with a minimum yield point of 33 ksi (230 MPa).
3. A most steel deck is manufactured in US conforming to ASTM A1008 /A1008M, Structural sheet for uncoated or uncoated top/painted bottom deck shall be as per ASTM A653 / A653M.
4. The thickness of the deck shall not be less than 95% of the design thickness. Thickness shall range between 22 gauge (0.75 mm) to 16 gauge (1.52 mm).
5. The overall deck section depth shall not be less than or equal to 3 in (75 mm).The minimum concrete above the top of the floor deck shall be 2 inches (50 mm).
6. The web angle measured from the horizontal plane, 'θ', shall be limited to values between 55^0 and 90^0 and the webs shall have no reentrant bends in their flat width.
7. In the case of additional steel to resist negative bending, the minimum cover of concrete above the reinforcing shall be 3/4 inch (20 mm).
8. Temperature and shrinkage reinforcement, consisting of welded wire fabric or reinforcing bars, shall have a minimum area of 0.00075 times the area of concrete above the deck (per foot or per meter of width).
9. When the ductility of the steel measured over a two-inch (50 mm) gage length, is less than 10%, the ability of the steel to be formed without cracking or splitting shall be demonstrated.
10. Embossments shall not be less than 90% of the design embossment depth.

4.5.3 Euro Standard [40] (Euro code EN1994-1-1-2004)

1. The range of concrete grades that are permitted in designs conforming to Eurocode 4 is much wider as C20/25 to C60/75 and LC20/22 to LC60/66 respectively.
2. The yield strength of the profile deck shall range from 235 MPa to 460 MPa .The thickness of deck shall be between 0.75 mm and 1.5 mm. Generally, preferred profile deck thickness ranges between 0.9 mm to 1.2 mm.
3. The depth of profile sheet shall be between 40 mm and 80 mm.
4. The overall depth of the composite slab shall not be less than 80 mm. The thickness of concrete above the main flat surface of the top of the ribs of the sheeting shall not be less than 40 mm.

5. If the slab is acting compositely with the beam or is used as a diaphragm, the total depth shall not be less than 90 mm and clear concrete depth shall be not less than 50 mm.
6. Transverse and longitudinal reinforcement shall be provided within the depth of clear concrete. The amount of reinforcement in both directions should be not less than 80 mm^2/m. The spacing of the reinforcement bars should not exceed '2h' and 350 mm, whichever is the lesser.
7. The reinforcement to prevent crack (Anti-crack mesh) shall be provided as A142, A193, A252 & A393 mesh, which represents area of mesh in 1 m^2.
8. The ‘simply-supported’ slabs require top longitudinal reinforcement of 0.2% of the cross-sectional area of concrete above the steel ribs for unpropped construction and 0.4% for propped construction at their supports, to control the widths of cracks.

4.6 General Recommendations for Construction of Composite Deck

This section is intended to be an aid and general guide for the safe and proper erection of steel deck, as it is emerging construction practice in India. The concerns for composite deck are quality assurance and safety, which is always paramount in construction industry. The guidelines presented in this section has been prepared in accordance with generally recognized engineering principles and accepted construction practice [46] in the other parts of the world. Design professional and builder may review the same for its applicability to specific job.

1. Steel Deck

The floor deck is made up of cold-formed steel sheet. Some mean of ensuring mechanical interlock must be provided to ensure composite action when the concrete is hardened. Deck indentations or protrusion to facilitate composite action must be properly checked and must not damage or improper in depth and geometry.

2. Packaging

The weight of profile deck which is packed into bundles shall be limited to around 1800 kg. This weight limitation is in regard to its structural weight over structural steel frames and for safe hoisting, spreading and installation procedures.

3. Storage and Protection

The deck must be protected against damage during all phases of construction. The deck bundles should be stored off the ground in such a way that one end remains elevated as compares to another end to provide drainage in case of storage on the ground. Bundles should be protected with a ventilated waterproof covering.

4. Lifting

Steel deck bundles must be rigged for lifting so that shifting and excessive tipping will not occur. Rigging should be adjusted to keep hoisted loads well-balanced. The capacity of structure (lifting Device) must be verified before the operation.

5. Installation

A steel deck is generally furnished in panels to cover widths of 0.6 to 1 m and in lengths up to 12.8 m in simple spans and continuous spans. The construction is generally done on an elevated structure. The procedure should be carefully monitored with all the safety precautions as per country's guideline. Deck shall be installed in accordance with the 'Approved for Construction' drawings. The deck must be installed by qualified and experienced workers. The beginning point should be carefully selected for proper deck orientation and edge of floor slab location. While placing the sheets, bottom of the sheet should extend to the support at both ends.

6. Shoring

Construction of composite floors can be done either by shored approach or by unshored approach. In case of shored system, a temporary supports are positioned beneath the beams to reduce deflection when concrete is placed. Extensive testing has shown that the flexural strength of a composite beam is the same regardless of whether it is shored or unshored when concrete is placed [47]. In U.S. practice, unshored construction is favorable approach as it is economic and speedy. However, for higher span, shoring shall be provided to the deck.

7. Working

To prevent deck damage during construction, the deck panels should be attached to the frame and side laps should be connected as soon as possible. A working area should be at least 3.5 m wide. Workers should also maintain a safe distance of around 2 m from the end of the deck unit. Deck should be properly fastened in accordance with the drawing and should have adequate bearing before concrete is poured. Deck fastening to the structure must be strictly followed and must not be changed without the approval of the designer. It is important to inspect damaged deck (if any) which may require shoring before the concrete pour.

8. Concreting

The deck must be cleaned of all foreign matter such as dirt, loose ferrules, and excess oil before the concrete is placed. Concrete must be placed first over the supporting beams and then outward toward the center of the deck spans. Concrete should be poured from a low level to avoid impacting the deck. Areas that buckle during the pour are usually caused by previous damage, over spanning the deck, or concrete to pile up must be given proper attention. Concrete admixtures containing chloride ions shall not be permitted to limit/prevent the corrosion of steel deck.

9. Opening

Small openings in a floor slab should be formed with the deck intact, and the openings in the steel deck should be cut only after the concrete reaches sufficient strength and stiffness during construction.

10. Precautions

Ladders should be securely tied to the structural frame or the scaffolding and access areas should be patrolled to keep them free of equipment, material, and debris. Workers should take precautions to protect themselves from sharp edges steel decks. Proper eye protection is necessary at the time of welding and installing galvanized deck on sunny days.

4.7 Design Criteria: British Standard

This section deals with the code based studies on design of steel concrete composite deck assuming full bond at interface as per British standard BS-5950-Part-IV-1994, Structural use of steelwork in building, Part 4: Code of practice for design of composite slabs with profiled sheeting.

4.7.1 Flexural Capacity

As per British standard, flexural capacity for full connection should be treated as an upper bound to the capacity of a composite slab. The flexural capacity in positive flexural regions is calculated assuming rectangular stress blocks for both concrete and profiled steel sheets as per Fig.4.1. The design strengths should be taken as $0.45f_{cu}$ for the concrete and p_{yp} for the profiled steel sheeting. The lever arm' z' should not exceed '$0.95d_p$' and the depth of the stress block for the concrete should not exceed '$0.45d_p$'.The resulting flexural capacity can be found out as per Eq.4.1.

$$T = 0.93A_p p_{yp} \quad , \ C = b * x * 0.45 * f_{cu} \quad , M_{Rd} = T * (d_p - 0.5\,x) \qquad \text{(Eq.4.1)}$$

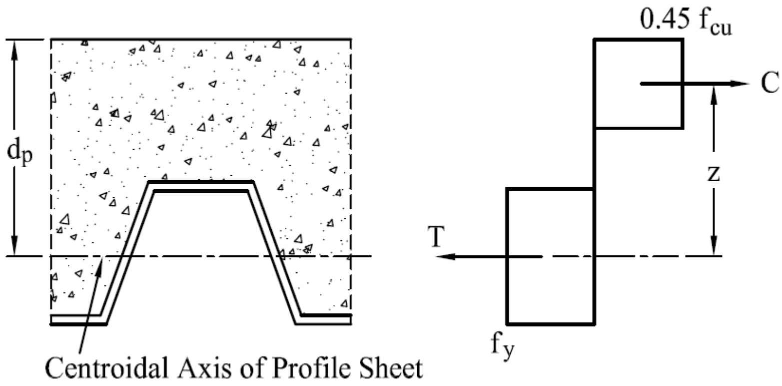


FIGURE 4.1 Composite Profile Deck as per BS-5950-1994

d_s	=	Effective depth of composite deck in mm
z	=	Lever arm in mm
p_{yp}	=	Design strength of profile steel sheets in MPa
f_{cu}	=	Characteristics concrete cube strength

4.7.2 Vertical Shear Capacity

The vertical shear capacity 'V_v' of a composite slab over a width equal to the distance between centers of ribs should be determined from the Eq.4.2 for open trough and reentrant profile sheet.

Open trough profile: $V_v = b_a d_s v_c$, Re-entrant profile: $V_v = b_b d_s v_c$ (Eq.4.2)

b_a	=	Mean width of trough of trapezoidal profile in mm
b_b	=	Minimum width of trough of reentrant profile in mm
v_c	=	Design concrete shear stress based on BS 8110: Part-I in MPa

4.7.3 Longitudinal bond capacity

When the capacity of a composite slab is governed by longitudinal bond, it should be expressed in terms of the vertical shear capacity at the supports. The longitudinal bond capacity 'V_s' should be calculated using Eq.4.3.

$$Vs = \frac{B_s d_s}{1.25}\left(\left(\frac{m_r A_p}{B_s L_v}\right) + k_r\sqrt{f_{cu}}\right) \qquad \text{(Eq.4.3)}$$

A_p = Cross sectional are of profile sheeting in mm^2

B_s = Width of composite deck in mm

L_v = Shear span in mm

The empirical parameters m_r and k_r should be obtained from parametric tests for the particular profiled sheet.

4.7.4. Deflection

The deflections of composite slab should not normally exceed the following at construction stage:

1. L/180 (but # 20 mm) when the effects of ponding are not taken into account.
2. L/130 (but # 30mm) when the effects of ponding (additional concrete due to the deflection of the sheeting) are taken into account.

The deflection of the composite slab should not normally exceed the following:

1. Deflection due to the imposed load: L/350 or 20 mm, whichever is the lesser.
2. Deflection due to the total load less the deflection due to the self-weight of the slab plus, when props are used, the deflection due to prop removal: L/250.

4.8 Design Criteria: American Standard

American code permits methods such as prequalified section method, shear bond method, full-scale performance testing as per SDI-T-C and other methods approved by the building official. The prequalified section method is discussed herein**.** This method for the calculation of strength of composite steel deck-slabs shall be used, when headed stud anchors (studs) are not present on the beam flange supporting the composite steel deck, or if steel headed stud anchors are present in any quantity.

4.8.1 Flexural capacity

Composite deck-slabs shall be classified as under-reinforced if slabs with (c/d) less than the balanced condition ratio (c/d) or over-reinforced if slabs with (c/d) greater than or equal to (c/d).Where (c/d) ratio can be calculated as per Eq.4.4.

$$c/d = \frac{A_s\,F_y}{0.85\,f'_c\,db\,\beta_i} \qquad \text{(Eq.4.4)}$$

The compression depth ratio for the balanced condition shall be calculated as per Eq.4.5.

$$\left(c/d\right)_b = \frac{0.003\,(h-d_d)}{d\left\{\left(\frac{F_y}{E_s}\right)+0.003\right\}} \qquad \text{(Eq.4.5)}$$

The factored flexural resistance, in positive bending, of an under-reinforced composite slab (c/d)< $(c/d)_b$ shall be taken as per following Eq.4.6 to Eq.4.9.Composite section and nomenclature as per American standard are depicted in Fig.4.2.

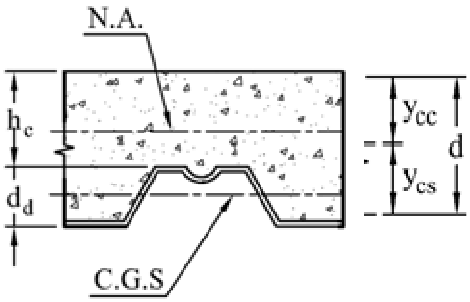


FIGURE 4.2 Composite Profile Deck as Per SDI-ANSI-2011

$$M_{ru} = \phi_s M_y \qquad \text{(Eq.4.6)}$$

$$M_y = \frac{F_y I_{cr}}{(h-y_{cc})} \qquad \text{(Eq.4.7)}$$

$$y_{cc} = d\left\{\sqrt{2\rho n + (\rho n)^2} - \rho n\right\} \leq h_c\ ,\ I_{cr} = \frac{b}{3n}\,{y_{cc}}^3 + A_s {y_{cs}}^2 + I_{sf} \qquad \text{(Eq.4.8)}$$

$$y_{cs} = d - y_{cc}\ ,\ \rho = \frac{A_s}{b\,d} \quad \text{and } n = \frac{E_s}{E_c} \qquad \text{(Eq.4.9)}$$

In Eq.4.4 βi shall be considered as: β_i=0.85 for $f_c' \leq 27.58$ MPa

β_i= 1.09-0.008 $f_c' \geq 0.65$ if $f_c' > 27.58$ MPa

ϕ	=	0.85 - Resistance factor
A_s	=	Area of steel deck in $mm^2/inch^2$
b	=	Unit width of compression face of composite slab in mm/inch
c	=	Distance from extreme compression fiber to composite N.A. in mm/inch
d	=	Distance from extreme compression fiber to centroid of steel in mm/inch

d_d = Overall depth of steel deck profile in mm/inch

E = Modulus of elasticity of steel deck in MPa/psi

f_c' = Specified compressive strength of concrete in MPa/psi

f_y = Yield strength of deck in MPa/psi

I_{cr} = Cracked section moment of Inertia in $mm^4/inch^4$

M_y = Yield moment for the composite deck-slab, with cracked cross section kN.m/m

4.8.2 One-way Shear Strength

The one-way shear strength of the composite deck in SI unit shall be calculated by Eq.4.10. and Eq.4.11.

$$\emptyset V_n = \emptyset_v V_c + \emptyset_s V_D \leq \emptyset_v \ 0.172\sqrt{f_c'}\,A_c \qquad \text{(Eq.4.10)}$$

where $V_c = 0.086\lambda\sqrt{f_c'}\,A_c$ (Eq.4.11)

V_D = Shear strength of the steel deck section in accordance with AISI S100 in MPa/ psi

A_c = Concrete area available to resist shear in $mm^2/inch^2$

ϕ_v = 0.75- Resistance factor

ϕ_s = 0.85 - Resistance factor

f_c' = Specified compressive strength of concrete in MPa/psi

In this equation value of 'λ' should be taken as '1' , if density of concrete exceeds 2100 kg/m^3 and 0.'75' if concrete density is less than 2100 kg/m^3.

4.8.3 Deck Deflection

Calculated deflections of the deck as a form or platform shall be based on the load of the concrete. This load shall be determined by the design slab thickness and the self-weight of the steel deck. For the uniformly loaded on all spans, deflection shall be limited to the lesser of 1/180 of the clear span or 3/4 inch (19 mm). Calculated deflections shall be relative to supporting members.

Live load deflections are seldom a controlling design factor. A superimposed live load deflection of span/360 is typically considered to be acceptable. The deflection of the slab/deck combination can be predicted by using the average of the cracked and uncracked moments of inertia as determined by the transformed section method of analysis.

4.9 Design Criteria: Euro Standard

This section deals with the study of Euro code for steel-concrete composite deck assuming full bond at interface as per EN 1994-1-1:2004, Eurocode-4, Design of composite steel and concrete structures-Part 1.1, General rules and rules for buildings, 2004. The vertical shear failure of the slab is rarely governing except in case of deep slab or short span slab. Vertical shear, longitudinal slip resistance and deflection criteria are as per discussion in chapter 3.

4.9.1 Flexural capacity

Full interaction between concrete and steel is assumed. In practice, this can be achieved by providing mechanical interlock which prevents horizontal failure or longitudinal slip between concrete and steel. Analysis of flexural resistance shall be carried as per Fig 4.3.

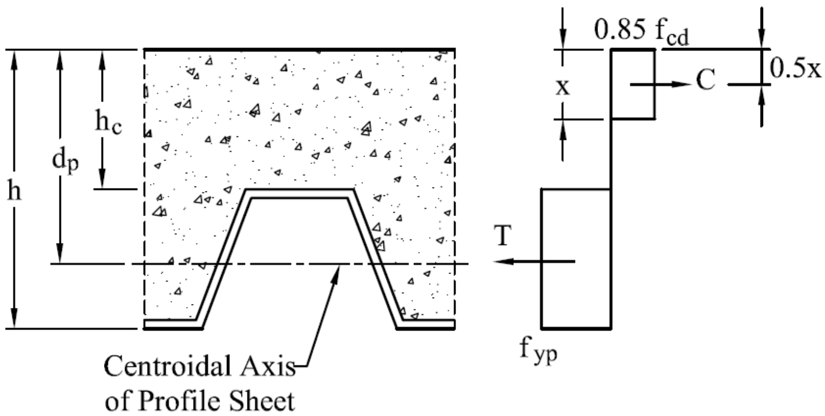


FIGURE 4.3 Composite Profile Deck as Per EN-1994-2004

Eurocode assumes the equivalent stress of concrete in compression as $0.85^*f_{cd}/\gamma_c$. The material factor of safety for concrete and steel are considered as 1.5 and 1.1 respectively.

$$T = A_p \frac{f_{yp}}{\Upsilon_p} \tag{Eq.4.12}$$

$$C = b * x * \frac{0.85 * f_{cd}}{\Upsilon_c} \tag{Eq.4.13}$$

$$x = \frac{\frac{Ap * f_{yp}}{\Upsilon_p}}{\frac{0.85 * b * f_{cd}}{\Upsilon_c}} \tag{Eq.4.14}$$

$$M_{Rd} = T * z \quad \text{where } z = d_p - 0.5\,x \qquad \text{(Eq.4.15)}$$

The neutral axis for bending lies in the concrete, where there is full connection; except where the sheeting is unusually deep. For sheeting in tension, the width of embossments should be neglected in calculating the effective area, unless tests have shown that a larger area is effective. Flexural capacity can be calculated using stress block theory as per Eq.4.12 to Eq.4.15.

4.10 Flexural Capacity: Indian Standard Stress Block

Design for the limit state of collapse in flexure is based on the following assumptions as per IS 11384-1985[44], which is same as described in IS 456-2000[45].

The maximum bending strain in concrete at the outermost compression fiber is taken as 0.0035. For characteristic compressive strength of concrete f_{ck} , maximum permissible bending compression in the structure is considered as 0.67 f_{ck}. Partial safety factor for concrete material is considered as 1.5. The stress-strain curve for the steel section is assumed to be bilinear and partial safety factor of the material is taken as 1.15. Steel deck material factor of safety is considered as 1.15 considering the fact that deck development is at growing stage in India.

The Flexural capacity for full connection can be determined using the partially rectangular, partially parabolic stress block as per Indian standard as depicted in Fig.4.4. The design strengths should be taken as $0.36f_{ck}$ for the concrete and f_y with factor of safety of 1.15 for the profiled steel sheeting. The lever arm z shall be considered as d_p-$0.42x_u$.

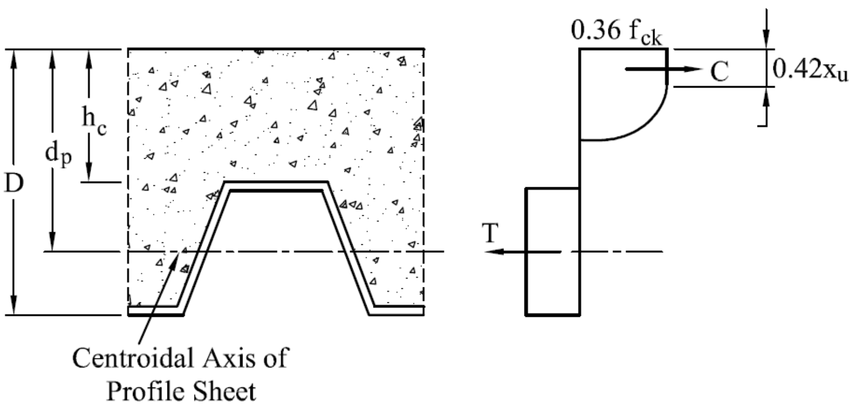


FIGURE 4.4 Composite Profile Deck as Per Indian Stress Block

4.11 Comparison of International Standards for Flexural Capacity

Comparison for flexural capacity considering full interaction between steel and concrete is made as per Euro standard EN 1994-1-1:2004, British standard BS-5950: Part-IV:1994, American Steel Deck Institute-ANSI: 2011 and Indian standard stress block. Euro and British standard assume rectangular stress block and for Indian standards partly parabolic and partly rectangular stress block is used. American national standard institute follows the cracked section moment of inertia and simple bending theory to calculate the flexural capacity. All countries have different factors of safety for profile deck. The comparison is done with different codes and thickness variation for profile configuration as shown in Fig. 4.5.Other properties such as overall depth of deck 110 mm, grade of concrete 25 MPa and grade of steel 230 MPa are considered. The results of flexural capacity using four International standards versus thickness of a profile are summarized as per Fig.4.6.

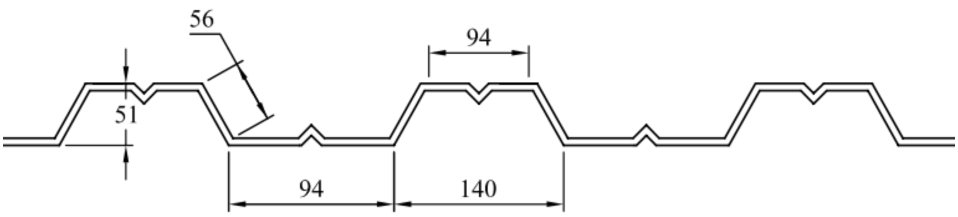


FIGURE 4.5 Profile Configuration '51'

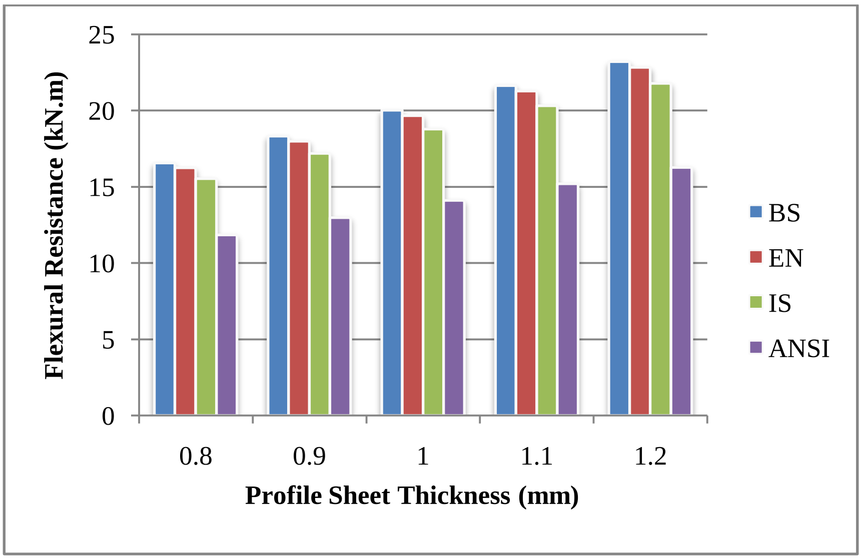


FIGURE 4.6 Variations in Profile Thickness: 0.8 mm to 1.2 mm

4.12 Problem Formulation: ComFlor Software and Euro Standards

From above discussed International standards, Euro Standard EN 1994-1-1:2004 is considered for further comparative studies due to the reason that, the process and approach as per Euro standards are close to Indian standard. In this section, analysis of composite deck is carried out by open source software 'ComFlor' [48] developed by Tata steel. The numerical problem is solved for flexural capacity using 'ComFlor' software for the Tata steel profile geometry. It is then compared with developed computer program as per Euro standards. In order to compare the results and design process of composite slab, analysis is carried out for a configuration shown in Fig. 4.7 and particular set of data. A solution of deck is carried out by developed program and ‘ComFlor' software as per Appendix A-1.The numerical data for the problem is also listed in Appendix A-1.

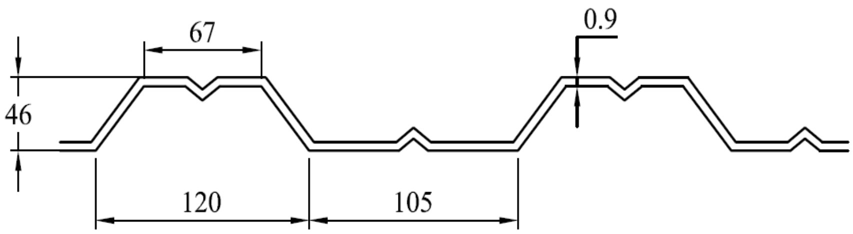


FIGURE 4.7 Profile Configuration '46'

4.12.1 ComFlor Software

'ComFlor' software developed by 'Tata steel' is used to analyze composite floor slabs in buildings. The deck may be single spanning or continuous over several bays. The analysis is limited to steel deck profiles from Corus Panels and Profiles. The software analyses composite deck as per British Standards or Euro Standards. Minimum yield strength analyzes by the software range between 280 to 450 Mpa.

The software has four basic features such as Structures tab, Loading tab, Design tab, and Results tab. Structures tab consist of input for given parameters: Deck profile types, Span, Span type, Propping, Beam or wall width, Slab depth, Concrete Class and Bar and mesh reinforcement.

After selecting a particular configuration, load data is to be input. It consists of Occupancy imposed loads, ceilings, and services, finishes and partition loads. Imposed loads, ceilings and services, finishes and partition loads are assumed to be uniformly distributed over the whole slab as is the slab self-weight, which is automatically calculated by the program.

Then analysis can be performed as per either Euro or British standard. It provides the definition of all parameters specific to the design code or process. To view the result, result tab is given, which provides comprehensive information regarding the outcome of the ComFlor analysis. The overall maximum unity factor is displayed, followed by detailed results of each check criteria. Graphs on normal stage bending, shear and deflection are provided.

For the geometry considered in Fig. 4.7 and other parameters as per Appendix-A-1, unity factor for flexure, i.e. ratio of flexural moment/flexural resistance is calculated. It is found that 'ComFlor' unity factor under normal slab bending resistance is '0.15' and developed computer program unity factor is '0.156'. The flexural capacity result of the developed program exactly matches with 'ComFlor' software.

4.13 Parametric Study

Profile sheets are made up of thin cold formed steel sheet and so it can be rolled into any desired shape. There can be a number of variations in configurations, geometrical as well as material parameters. As profile sheets are available in various size, steel grade, thickness and shape, a parametric study is carried out for analyzing the flexural capacity of a particular pattern of composite deck. In addition, neutral axis factors are derived to verify under reinforced section. A program is developed as per Euro standard to study the important geometrical-material parameters, which influence the flexural capacity.

4.13.1 Neutral Axis Factor

In the case of flexural failure under full interaction, it is also necessary to ensure the ductile behavior of steel concrete composite deck analytically. Ductility of composite slabs is a very important consideration, although it appears that many of the existent international codes throughout the world do not have an inherent ductility clause, which is reflected in the design of reinforced concrete slabs. The codes investigated include BS 5950, and EC4. Hence, the value of the balanced depth of neutral axis is developed for different steel grade. These values should be compared with the actual depth of neutral axis under full composite action. Neutral axis depth is calculated for commonly used steel grade of profile deck considering strain diagram of singly reinforced R.C.C. section. For deriving the neutral axis strain in concrete is taken as 0.0035 and corresponding strain in steel is calculated. Modulus of elasticity of steel is assumed as 20000.

It is suggested that, in the absence of current recommendations for ductility, following consideration for limiting depth of the neutral axis shall be taken into account. Table 4.1 depicts neutral axis factor for balanced section and commonly used steel grades in profile deck.

TABLE 4.1 Neutral Axis Factor

Sr. No	Steel Grade	Neutral Axis Factor
1	220 #	0.538
2	230#	0.535
3	280*	0.517
4	310	0.507
5	350*	0.494
6	365	0.489
7	450*	0.464

Minimum Grade, * Commonly used Grade

The trapezoidal shape of profile as per Fig.4.5 is considered for calculating flexural capacity and depth of neutral axis. Fig.4.8 (a) to Fig. 4.8 (c) shows values of estimated flexural resistance and depth of neutral axis for variation in different parameters keeping any one constant. Reference parameters are: concrete grade 25 MPa, steel grade 230 MPa , profile thickness 1 mm and overall depth 110 mm. The variations are chosen considering the limiting criteria for geometrical variation as per Euro standards. In that regard, variations are made in: (i) overall depth of deck from 90 mm to 120 mm (ii) Grade of steel from 230 MPa to 450 MPa (iii) Concrete grade from 25 MPa to 40 MPa.

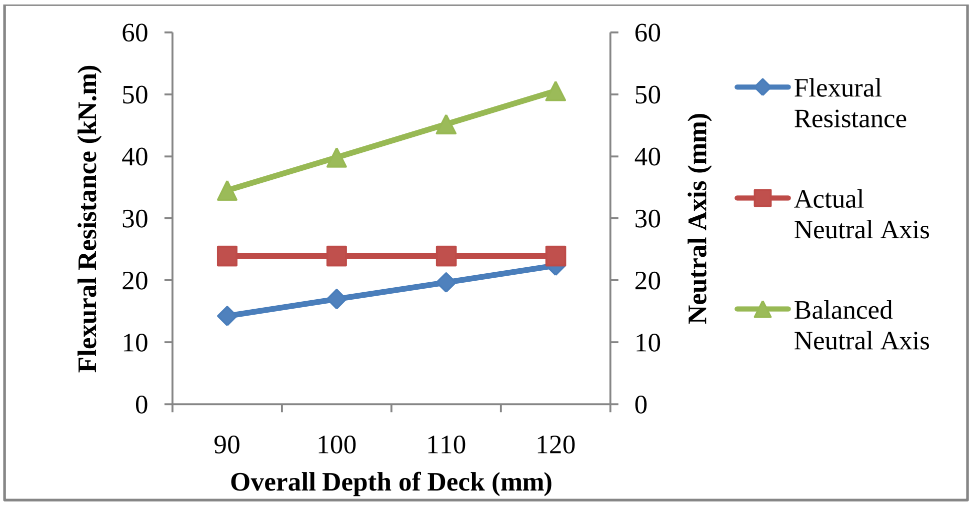


FIGURE 4.8(a) Variation in Overall Depth: 90 mm to 120 mm

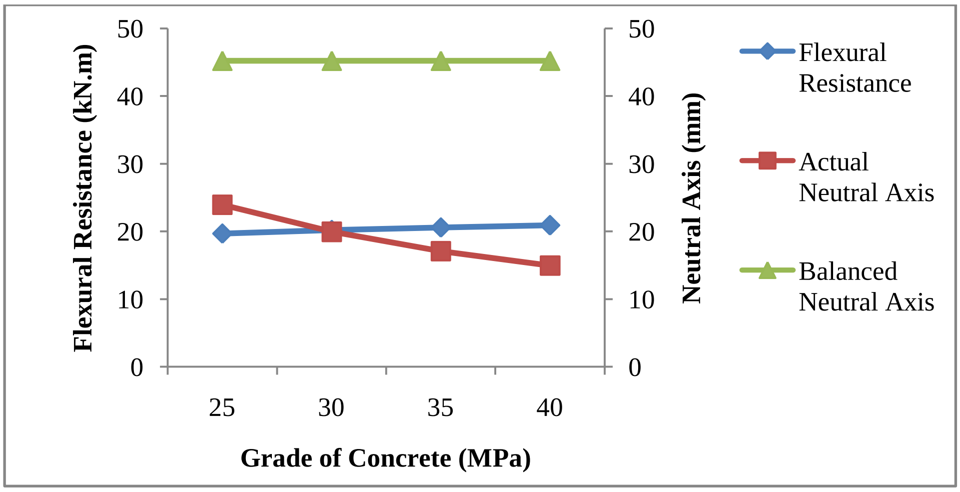


FIGURE 4.8(b) Variation in Concrete Grade: 25 MPa to 40 MPa

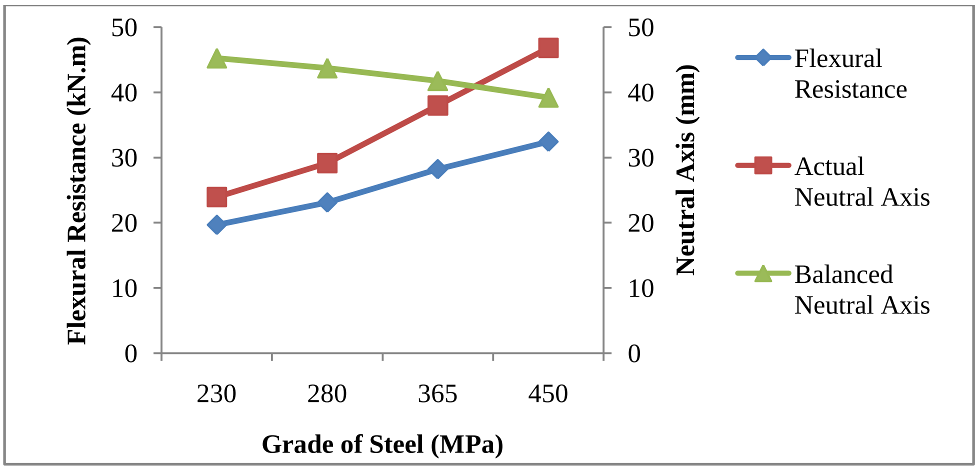


FIGURE 4.8(c) Variation in Steel Grade: 230 MPa to 450 MPa

4.14 Results and Discussions

The study herein involves code based analysis for flexural capacity with various International design codes, comparative studies and development of design chart for parametric variations such as different materials, profiled sheet thickness and slab depths for a particular geometry considering full bond. General guidelines for composite deck construction are presented as the type of construction is an emerging trend in India. The major results and discussions are as follow:

1. Comparison of International standards - British, Europe, America and India depicts that, Europe and British standard assume rectangular stress block, whereas Indian standard considers partly parabolic and partly rectangular stress block for design. American national standard institute follows the cracked section moment of inertia and simple bending principle to calculate the flexural capacity.
2. The factor of safety for profile deck considered as per Euro code is 1.1 whereas British and American code consider resistance factor of 0.93 and 0.85 respectively. Factor of safety for profile steel sheet as per Indian condition is suggested as '1.15'. considering the fact that production/use of profile deck is at developing stage in India. For small scale projects fabrication is done by press braking which might not have proper production control.
3. Flexural capacity of the deck is estimated as per various design standards, considering full bond. Highest estimated flexural resistance is observed as per British standard over other standards. For Indian design criteria, flexural resistance of composite deck is calculated considering partially parabolic and partially rectangular stress block, with the strain value '0.0035' and factor of safety for steel as '1.15'.
4. The estimated value of flexural resistance is 4% and 6% higher based on Euro standard and British standard respectively as compared to IS stress block. American standard SDI-ANSI gives conservative results of flexural capacity.
5. Limiting geometrical parameters such as depth of the deck, overall height of concrete, dimensions of web and flanges of profile depth and thickness of profile deck are suggested to consider same as Euro standards. Limiting material grade for steel and concrete shall also be considered similar to Euro code.

6. Generalized computer program is developed as per Appendix -I, to estimate flexural capacity and to perform parametric analysis of composite deck. The results of the program are compared with available software 'ComFlor'. The limitations of 'ComFlor' software is that , steel profiled sheet manufactured by Tata steel with steel grade of 280 MPa and 350 MPa can only be analysed . A concrete grade below C25 cannot be analyzed by this software.Whereas the developed program can be used for any geometric and material variations.
7. Parametric study shows that estimated value of flexural capacity varies with a variation in grade of concrete, grade of steel and overall depth of deck. For the deck analyzed, 57.10% increase in flexural resistance is found on increasing overall depth from 90 mm to 120 mm. If a grade of concrete is increased from 25 MPa to 40 MPa, 6.18 % increase in moment of resistance is observed. For a higher grade of steel varying from 230 MPa to 450 MPa, 64.78% increase in the moment is obtained.
8. The limiting value of neutral axis for a balanced section of a composite deck is developed as per Table 4.2, which should be analyzed for any composite deck design to ensure under reinforced section theoretically. Neutral axis clause is not described in EN 1994 and BS 5950. For the investigated geometry and material properties, steel grade of 450 MPa should be avoided as it makes the section over reinforced as per Fig.4.8(c).
9. The increase in grade of steel significantly increases the value of the actual depth of neutral axis. For the deck considered, the actual neutral axis is far below the balanced section neutral axis. For any developed configuration, best possible value of neutral axis may be analyzed to minimize the area of concrete in the tensile zone.

CHAPTER-5

Experimental Studies on Composite Deck

5.1 General

Composite decks rest onto supporting beams or walls along the direction of ribs. Total slab thickness generally ranges from 100 mm to 250 mm and span ranges from 3 to 4.5 m. When the concrete gains sufficient strength, it acts in combination with the tensile strength of the decking to form a ‘Composite’ slab. Experimental work on the composite deck and experimental results are presented in this chapter. Studies are made on three wavelength test and one wavelength test specimen with varying bond patterns by conducting two series of experimentation. The presentation is first focused on the observations of the behavior of three wavelength test (Series # 1) specimens and then on one wavelength (Series # 2) specimens with different bond patterns.

5.1.1 Objective of Experimental Studies

The concern about steel concrete composite deck in India is quality of interlocking. Poor roller setup, roller wear and use of low ductility steel sheet can lead to non-uniform protrusion depth, size and shape. Cost of deck [36] increases by about 25 % with embossment and increased depth of the embossment leads to tearing [24].Various experimental methods were developed in past, to estimate the capacity of composite deck. Most of the research work was focused on large scale testing. Few study demonstrated small scale test but in those cases, the test procedure does not represent the actual loading conditions of the deck. Moreover, steel concrete composite deck construction is at growing stage in India and there are no guidelines available as per Indian standards. The purpose of the experimental work is to study behaviour of composite slabs with different bond patterns under bending. In this regard, this study can be useful to understand the effect of different mechanical interlock, so that an efficient composite floor system can be developed, which optimally utilizes steel and concrete.

The experimental tests are conducted on three wavelengths and one wavelength test specimen. The three wavelength test specimen comprises of three deck corrugations of 700 mm width whereas one wavelength deck specimen comprises of one deck corrugation of 230 mm width. The purpose of the one wavelength elemental specimen is to narrow in size so that test is more economical and simple to conduct as compared to full-scale tests. Both the types of specimen are tested under vertical loads in such a way that one way bending takes place. The span of the specimen and other parameters for three and one wavelength specimen are within the range of European standard guidelines.

5.2 Experimental Investigation

The experimental program has been conducted in two series: One series for three wavelength tests and another series for one wavelength tests. The full-size series of three wavelengths consisted of eight specimens with different bond patterns (Test CS1 - CS4). All specimens were built using trapezoidal deck profiles. The full-size specimens were 700 mm wide and 1500 mm long and constructed in fully supported condition. The tests in Series # 2 consisted of twelve specimens (Test CB1 - CB12) of width 230 mm. The variables for Series # 2 tests were similar to the Series # 1 specimens, which included variation in bond patterns keeping the span length same.

Results of three wavelength series # 1, is used as the base for indication of the effect of different bond pattern as primary investigation. Then results of small-scale series # 2, are presented with improved and simple to apply bond patterns. Throughout the discussions, the specimens are identified with the test numbers as given in Table 5.1 for full-scale series and Table 5.2 for small scale series. All loads are presented as equivalent uniform loads to facilitate comparison between results.

5.3 Three Wavelengths Composite Deck Specimen

In three wavelength specimens, different bond patterns between steel and concrete for composite deck floor were considered. An experimental investigation was conducted on eight profile steel decks in sets of two. All eight decks were having same geometrical configuration as shown in Fig.5.1 (a).

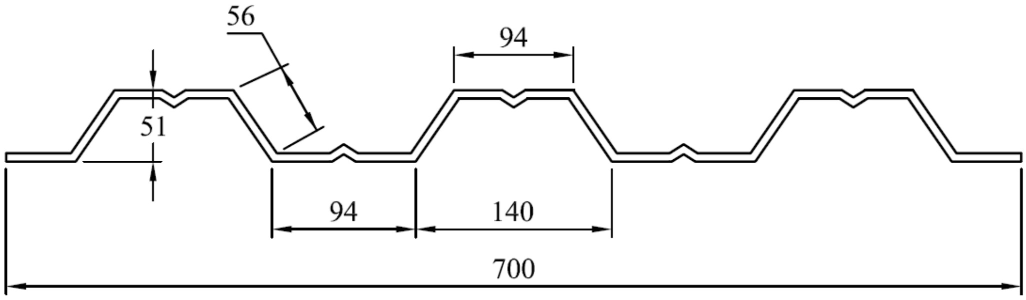
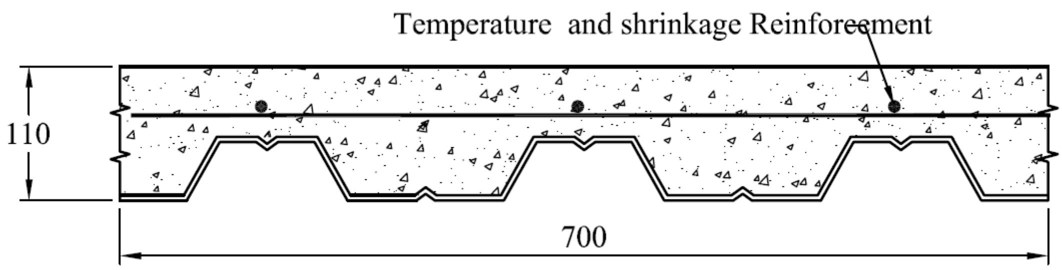


FIGURE 5.1(a) Profile Configuration: Series #1



All Dimensions are in mm

FIGURE 5.1 (b) Section of Composite Deck: Series #1

5.3.1 Material Properties and Interface properties

Out of eight decks, two decks were embossed and other six were unembossed. Embossed sheet was having embossment of oval shape. Unembossed profile decks were tested for three different bond patterns: Welded hemisphere on the surface of deck, chemical bond between profile and concrete and cross stiffening plates. Average section properties and dimension of the deck were as per Table 5.1. Yield strength steel sheeting was 250 MPa.

Table 5.1 Geometric and Material Properties: Series #1

Specimen	Tested comp. the strength of concrete (MPa)	Thickness of Profile Deck (mm)	Width of Profile Deck (mm)	Area of Profile Deck (mm^2)	Overall Concrete Depth (mm)	Effective Depth (mm)
CS1A-B	26.78	0.8	700	736	110	84.5
CS2 A-B	26.78	0.8	700	736	110	84.5
CS3 A-B	26.78	0.8	700	736	110	84.5
CS4 A-B	26.78	0.8	700	736	110	84.5

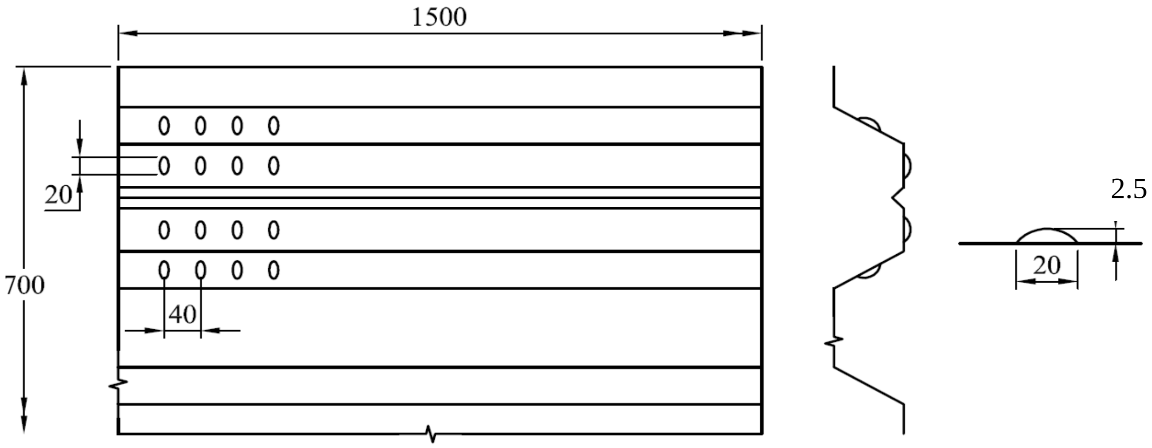


FIGURE 5.2 (a) Interface Topology CS-1

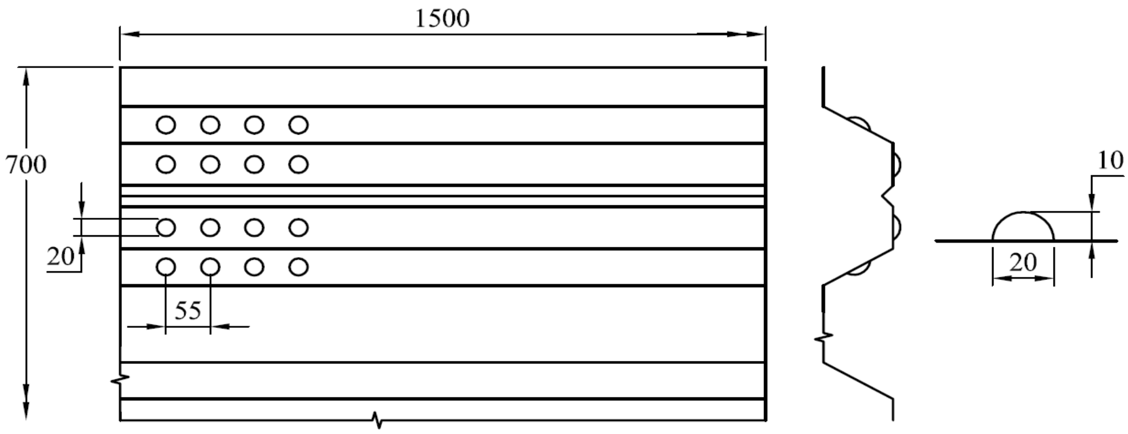


FIGURE 5.2 (b) Interface Topology CS-2

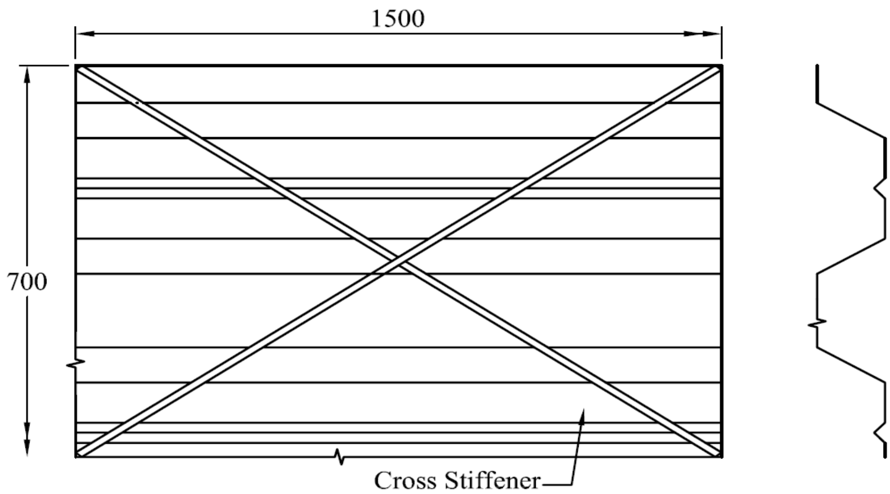


FIGURE 5.2 (c) Interface Topology CS-4

All Dimensions are in mm

Two steel decking sheets with normal embossment pattern were named as CS-1A and CS-1B.These sheets were having oval shape embossment pattern with approximate lateral dimension of 20 mm and approximate depth of 2.5 mm. In second slab set CS-2A and CS-2B, steel hemispheres were welded on plain corrugated steel sheets (without embossments). 20 mm diameter hemispheres were welded on the inclined side and top side along the length of the steel sheet at distance of 55 mm by argon welding. In third set, two coats of chemical binding agent epoxy 'Sikadur 31' was applied on CS-3A and CA-3B. Cross stiffeners were connected to unembossed sheet, CS-4A, and CS-4B. Cross diagonal stiffeners were made up, from plain steel sheet having length of 1625 mm, height of 25 mm and thickness of 2 mm. The yield strength of steel deck was 250 MPa. An exact geometrical drawing and laser cutting techniques were employed to prepare stiffeners. Details of composite specimen CS-1, CS-2 and CS-4 are as per Fig.5.2 (a) to 5.2 (c).The photograph of all different interface topologies are shown in Appendix-II.A.2.1.

5.3.2 Experimental Set up and Instrumentation

A total of eight numbers of composite decks were casted as per the parameters shown in Table 5.1. Set of two slabs in each type specimens were constructed. Steel decking surface was well cleaned before casting of the concrete. Concrete mix for M-25 grade was designed as per Indian standards. After 28 days, concrete compressive strength was determined from testing concrete cubes 150 mm × 150 mm × 150 mm size and was found as 26.78 MPa. 6 mm reinforcement bars at a spacing of 175 mm centre to centre were placed at a top cover of 30 mm from concrete top, to resist temperature and shrinkage. All composite slab specimens were casted with full support on the plain surface and included a mesh. The curing period of all specimens was 28 days.

The composite deck specimen were simply supported on two external steel I sections and were loaded symmetrically with the uniformly spaced I-girder as shown in Fig.5.3. In most of the test program conducted by other researchers[32], two- point loading arrangements are used, although the results are used to design slabs which, in practice, may have different loading patterns than these[49]. So in this research numbers of line loads are used to simulate as close as possible real life case.The tests were carried out on simple spans loaded with a symmetrical arrangement of a number of line loads. The line load arrangement approximated a uniformly distributed load, which is usually normal design situation for slabs.

A static load test to failure was then applied by the load cell of capacity 25 Ton. Proving ring with the least count of 2 Division (1.41 kN) was kept between the load cell and the top cross girder.The graduation on the proving ring was adjusted to zero and then load was applied gradually from the manually operated hydraulic jack at constant interval of 2 divisions. Three dial gauges were placed beneath the bottom edge of the deck, one at midspan and two at quarter span of the slab. Two dial gauges were placed on concrete and steel surface to measure relative slip. The least count of the dial gauges was 0.02 mm. No effort was taken to measure vertical separation.

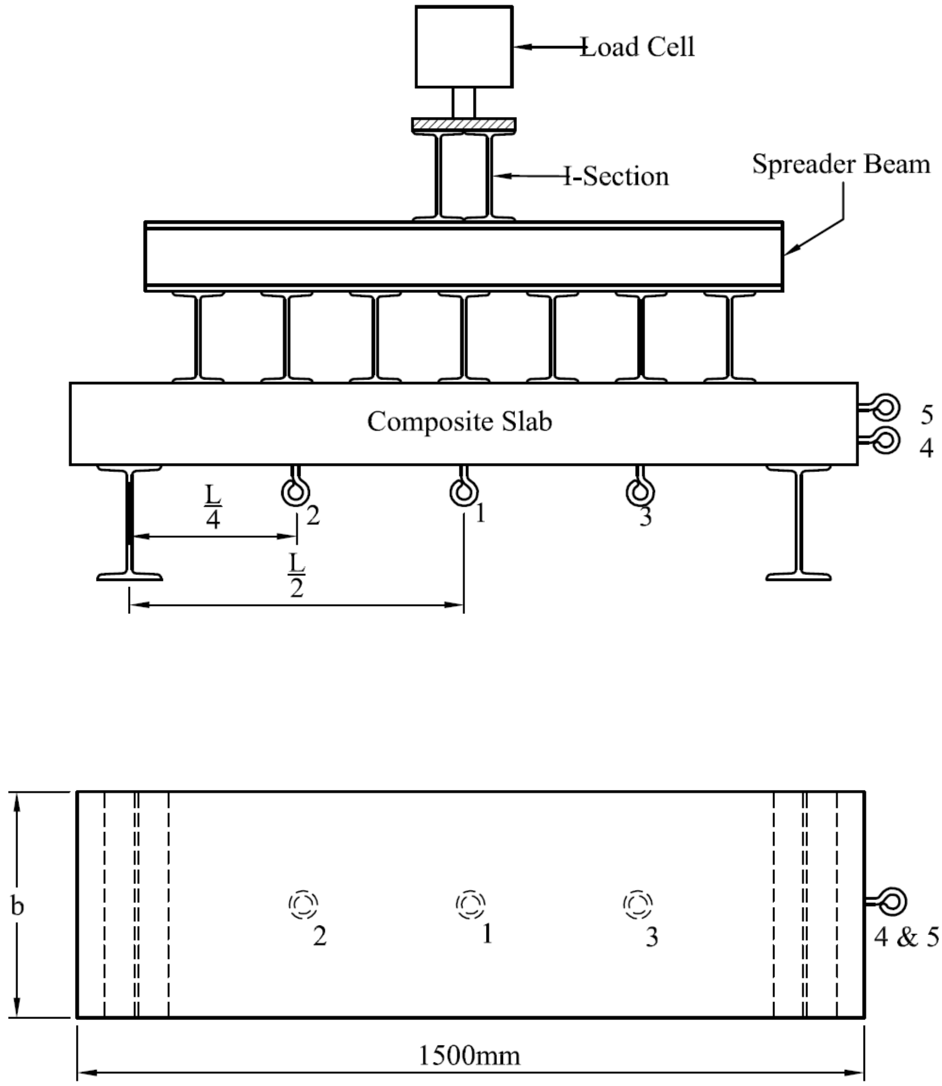


FIGURE 5.3 Test Set Up and Instrumentation

The test specimens were properly whitewashed to obtain a clear picture of cracks under different stages of loading. The actual test set up is as depicted in Fig.5.4.The weight of the I sections as spreader beams weight was added to the load cell reading. The weight of the slab was ignored owing to its negligible effect on behaviour and the composite action. The load at first crack, local damage, slip readings and crack patterns were observed in all specimens.

FIGURE 5.4 Actual Test Set Up: Series #1

Load-deflection curves and Load Slip curves are established for all eight specimens as shown in Fig 5.5 and Fig.5.8. For each specimen load at first crack, load at slip initiation, load at significant slip and maximum load are reported as per Table 5.2.Significant observation on behaviour and failure is shown as per Table 5.3.

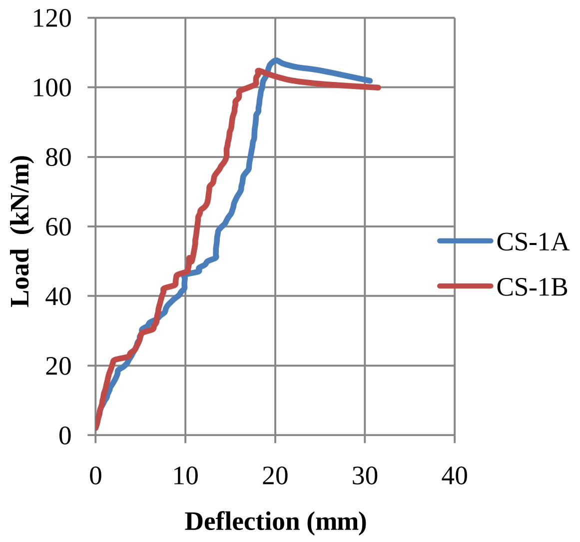


FIGURE 5.5 (a) Load Deflection Curve CS-1

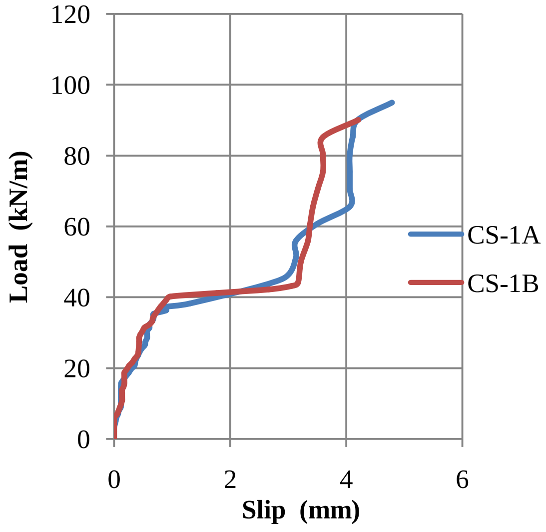


FIGURE 5.5 (b) Load Slip Curve CS-1

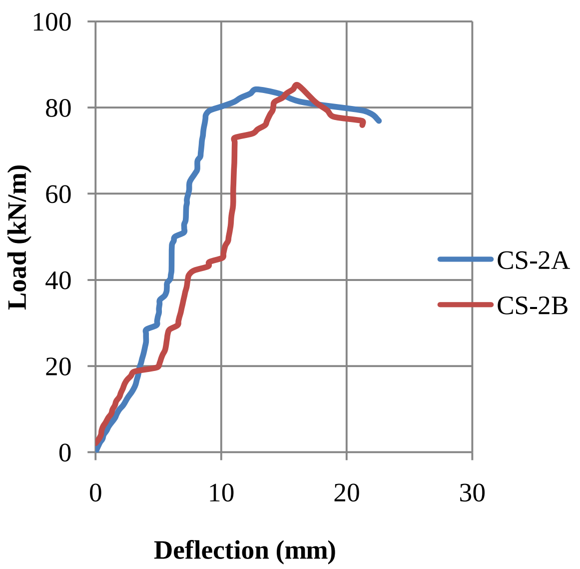


FIGURE 5.6(a) Load Deflection Curve CS-2

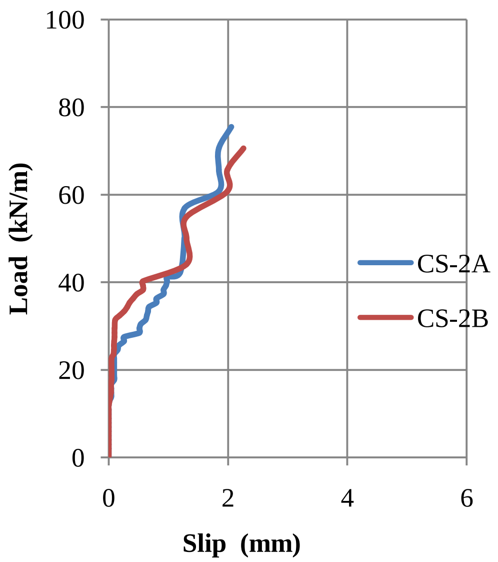


FIGURE 5.6(b) Load Slip Curve CS-2

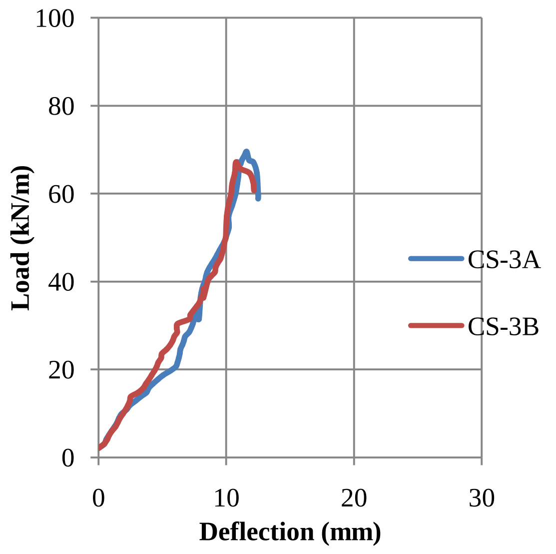


FIGURE 5.7(a) Load Deflection Curve CS-3

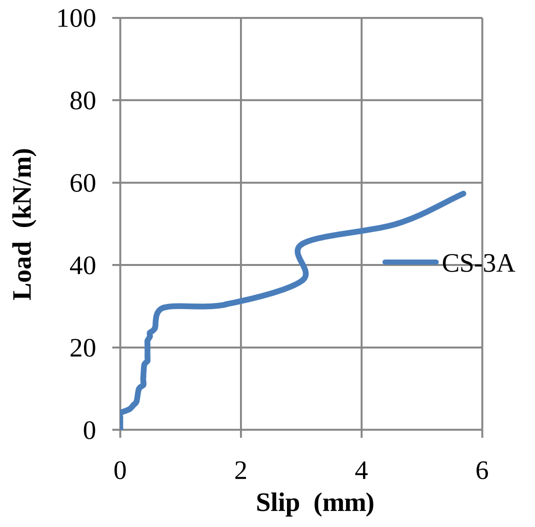


FIGURE 5.7 (b) Load Slip Curve CS-3

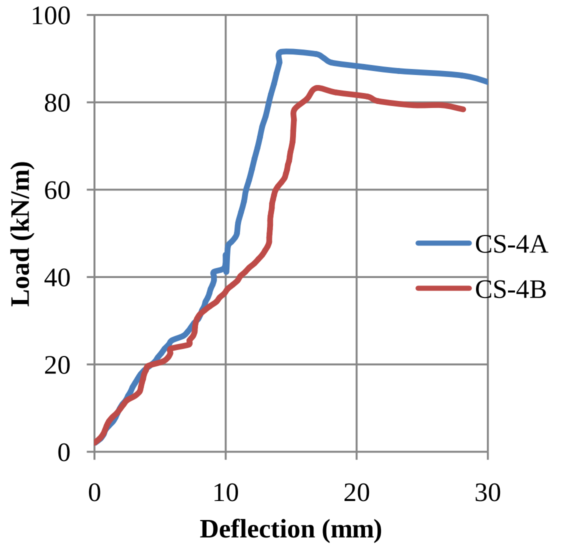


FIGURE 5.8 (a) Load Deflection Curve CS-4

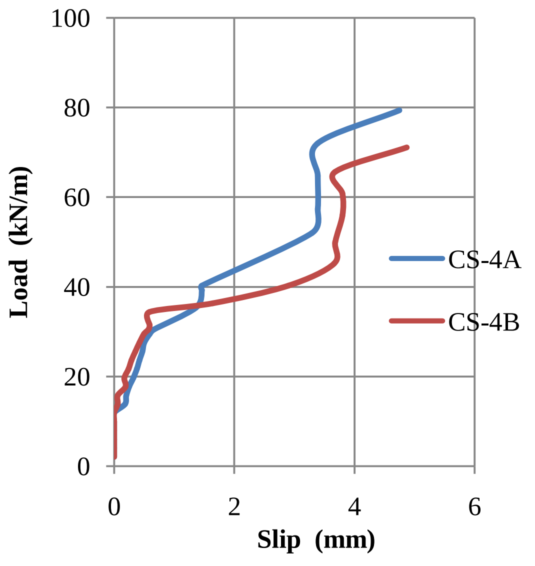


FIGURE 5.8 (b) Load Slip Curve CS-4

Table-5.2 Significant Load and Deflection Values: Series #1

Specimen	Topology at interface	Load at first crack (kN/m)	Load at slip Initiation (kN/m)	Load at Significant Slip (kN/m)	Max. Load (kN/m)	Midspan Deflection (mm)	Last Recorded Slip (mm)
CS1A-B	Oval embossment	25.07	39.74	47.56	106.26	31.39	4.50
CS2 A-B	Welded Hemisphere	17.24	43.16	60.78*	84.74	21.90	1.90
CS3 A-B	Chemical Agent	30.95	30.45	36.32	68.36	12.33	5.69
CS4 A-B	Cross Stiffener	23.60	35.83	48.54	87.43	29.80	4.81

*Potential slip < 3mm

Table 5.3 Significant Observations Series #1

Specimen	Topology at interface	Observation of slip	Observation of vertical separation	Failure
CS1A-B	Oval shape embossment	Minor	Major	Ductile
CS2 A-B	Welded Hemisphere	Negligible	Negligible	Ductile
CS3 A-B	Chemical Agent	Major	Major	Brittle
CS4 A-B	Cross Stiffener	Minor	Minor	Ductile

5.3.3 Significant Observations: Three Wavelength Specimen

The first set of experiments were carried out with eight specimens of three wavelengths , out of which one set was embossed and three sets were unembossed. Different bond patterns such as welded hemisphere on the surface of the deck, chemical agent and cross stiffening plates were provided for composite action to arrive at the best possible system. At the loading point that caused initial slip and potential slip, flat plateau in the load deflection curves can also be seen in most of the cases. Experimental values of load at initiation of (1 mm) slip [50] (Patric, Bridge1994) and load at a significant loss of composite action (3 mm) are tabulated as per Table 5.2. In all the specimens loading was continued after the maximum analytical flexural capacity was achieved to know that whether the slip between the deck and concrete takes place in the pre-ultimate range or post ultimate range.

During the test, after recording the significant slip, precise data of the slip was not obtained at the end due to vibration occurring from set up and larger slip. Cracks and failure of specimen CS-1 to CS-4 are shown in Appendix-II - A.2.3.

5.3.3.1 CS-1-A-B

The concrete compressive strength for all specimens was 26.78 MPa. The loading program was proceeded by load increase of 2 Division (1.41 kN) increments. The loading was considered in kN/m and addition is made for load of I-girder and load cell. The first crack was observed at 25.07kN/m. After that deboning noise is produced which showed the breakdown of bond and slip is started occurring at a load of 39.74 kN/m. Significant slip is observed at a load of 47.56 kN/m. The concrete and profile deck were in contact with I sections and the supporting system. Maximum load applied was 106.26 kN/m but at that time major vertical separation was observed. CS-1A and CS-1B showed a bending crack and local buckling of profile configuration. The average slip of both the specimen was found as 4.5 mm.

5.3.3.2 CS-2-A-B

The loading sequence for this test was similar to CS-1. At a load of 17.24 kN/m, first crack appeared. Then at a load of 43.16 kN/m, minor separation between concrete steel sheet occurred. At some points during the loading process, the slab was unloaded and then reloaded. However, response due to unloading was not possible to measure. Along with the flexure cracks, diagonal crack on a side, under I section was observed. Then at load of 60.78 kN/m again significant slip was observed. But in the case with hemisphere, slip was only around 1.9 mm and not as significant as CS-1A-B.Maximum load applied to the specimen was 84.74 kN/m. No vertical separation was observed in these specimens.

5.3.3.3 CS-3-A-B

For specimen with chemical agent, loading sequence was similar to those of previous two sets of tests. Cracking over the supports occurred at a load of 30.95 kN/m. Along with the first crack, the specimen showed a slip initiation at interface. The slip is generated because of breaking of chemical bond. After that load is increased and at a load of 36.32 kN/m major slip is occurred in the slab. Further, the load is increased by the maximum value of

68.36 kN/m. CS-3 specimen showed a major visible slip of about 5.7 mm as well as vertical separation. The concrete of the slab had almost separated from the steel with the sudden failure. Load-slip reading of CS-3B could not be taken because of an instrumental error.

5.3.3.4 CS-4-A-B

The compressive strength of the concrete and loading sequence for CS-4 was similar to those of previous tests. The crack load was obtained at load of 23.60 kN/m. Once the crack initiated, mechanical interlock started working for composite action. The slip was observed at a load of 35.83 kN/m. At the load of 48.54 kN/m significant slip was observed in cross stiffener composite specimens. Maximum load applied to specimens was 87.43 kN/m. The slabs CS-4 failed in bending along with minor local buckling of members and end slip of 4.8 mm on each side. Because of the stiffner, the deck behaved as monolithic element but intermediate bucking is observed between steel and concrete.

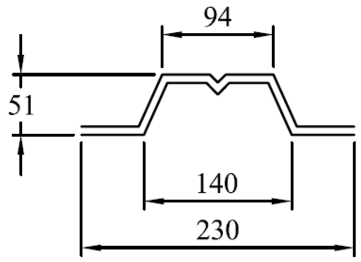
Above experimentations and bank of test information has led to several results and discussions, in regard to the behaviour of composite slabs. Series of tests conducted with different bond pattern shows different response to composite action. However, the maximum load in most of the cases were very high because of overall ductility of the system and reserved strength[49]. Results of the three wavelength specimen indicate that composite specimen with Hemisphere case - CS-2 show considerable slip resistance from initial slip to significant slip by 40% ,as compared to only 19 % for oval embossment case. The oval shape embossments pattern deck contributed minimally to composite action. These observations accord well with those of other researchers. It is proved that depth of the mechanical interlock is important parameter for the composite action. Though there was an issue with welding of hemisphere on steel plate, so in the next phase of one wavelength specimen, bolts are used as bond instead of hemisphere. The influencing parameters for one wavelength specimen are explained in next section. Chemical bond at the interface case was excluded in further analysis owing to its brittle behaviour. It was decided to increase load interval in Series #2 to 5 divisions instead of 2 divisions which was a case of Series #1.

5.4 One Wavelength Composite Deck Specimen

Composite slabs are essentially made from similar components as a composite beam namely a steel section (profiled sheet) and a concrete slab which are connected to resist longitudinal forces. Composite decks are designed to act as one way bending elements, which bend along the direction of rib. The small scale one wavelength specimen also behaves as one way bending element. Other than that, there are two critical factors which govern the behaviour of small width -one wavelength specimen.

The first factor is the flexibility of one wavelength deck.The overriding resistance between the concrete and steel sheet is decreased at the time of occurrence of horizontal slip in small-scale specimen. The effect may be less significant in full-scale test specimens because the webs are interconnected. But it should be noted that the so called full-scale test done by researchers were actually not too wide. Porter and Ekberg [19] have studied experiments on mostly one-panel deck composite slab specimen with 12" to 36" width. As reported by Prasannan [8] test conducted at West Virginia University for varying no. of ribs had shown that load carrying capacity increased as the number of ribs of the test specimen increased. However, the full-scale result may still be below the actual slab. The second factor is web curling which may occur in the edge webs. The presence of embossment in the webs can create a reaction force that pushes the webs away from the concrete once the slip occurs. Thinner steel sheets and deeper webs are more vulnerable to curling as recognized by Stark [21]. Therefore in Series #2 experiments, thickness of deck is chosen as 1 mm to avoid the effect of web curling and steel-concrete interface connections are provided by preparing a hole and then inserting a bolt .

This section deals with simple to apply bond techniques in profile sheets by relatively small scale test. It deals with different bond connections by considering one wavelength test specimens as shown in Fig.5.9 (a) and (b). In Series #2,total of twelve composite specimens (CB-1 to CB-12), each in set of two specimens was casted and tested under uniformly distributed load (same as three wavelength slab) which is the actual condition of slab loading [35] with varying interface connections. The interface topologies considered were: bolt with different orientation, Arc bent, stiffener, pressed in-in and pressed in-out embossment. Average section properties and dimension of the deck are listed as per Table5.4.



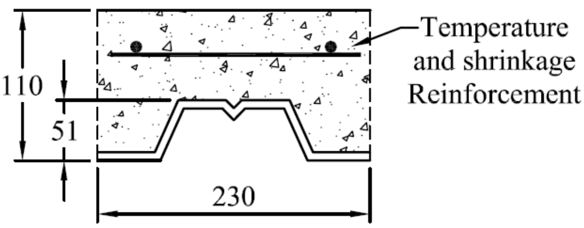


FIGURE 5.9 (a) Profile configuration: Series #2

FIGURE 5.9 (b) Section of Composite Deck: Series #2

All Dimensions are in mm

Table 5.4 Geometric and Material Properties : Series #2

Specimen	Tested comp. the strength of concrete (MPa)	Thickness of Profile Deck 't' (mm)	Width 'b' (mm)	Area of Profile 'Ap' (mm^2)	Overall Concrete Depth (mm)	Effective Depth 'dp' (mm)
CB1-2	32.88	1	230	298	110	84.5
CB3-4	32.58	1	230	298	110	84.5
CB5-6	33.48	1	230	298	110	84.5
CB7-8	30.07	1	230	298	110	84.5
CB9-10	29.92	1	230	298	110	84.5
CB11-12	30.21	1	230	298	110	84.5

5.4.1 Material Properties and Interface Topologies

An experimental investigation has been conducted on twelve profile steel decks in sets of two. All twelve decks were of same profile geometrical configuration. These decks were having different interface pattern as shown in Fig.5.10 (a) to (f).Out of twelve decks, four decks were embossed and other eight are unembossed. Profile decks were tested for six different topologies: **(**1) Bolt head at interface (2) Bolt shank at interface (3) Arc bend and bolts (4) Straight stiffeners (5) Pressed in-in embossments & bolts (6) Alternate Pressed in-out embossments & bolts. Yield strength of specimen CB-1 to CB-8 was 230 MPa and CB-9 to CB-12 was 365 MPa.

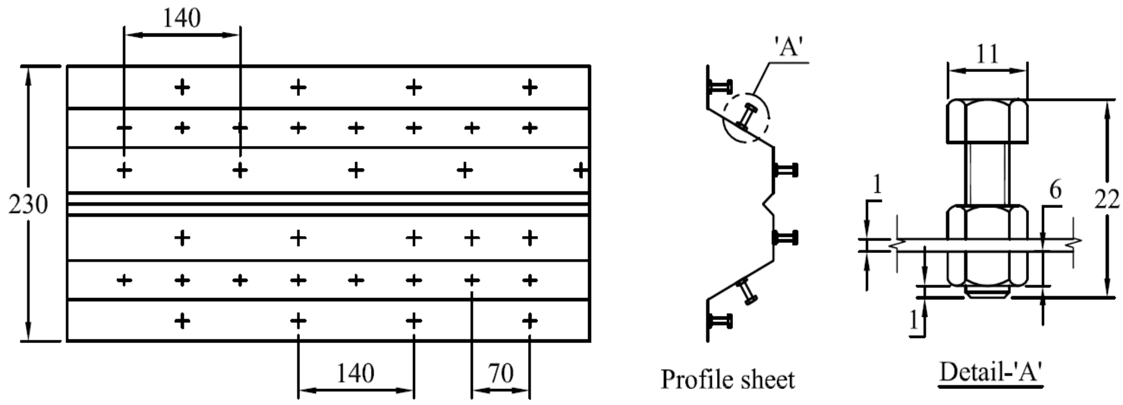


FIGURE 5.10 (a) Interface Topology CB-1-2

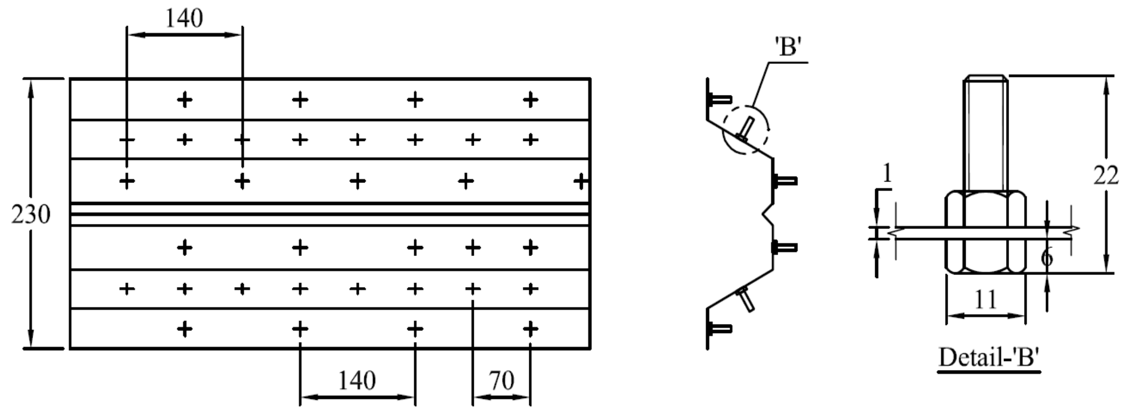


FIGURE 5. 10 (b) Interface Topology CB-3-4

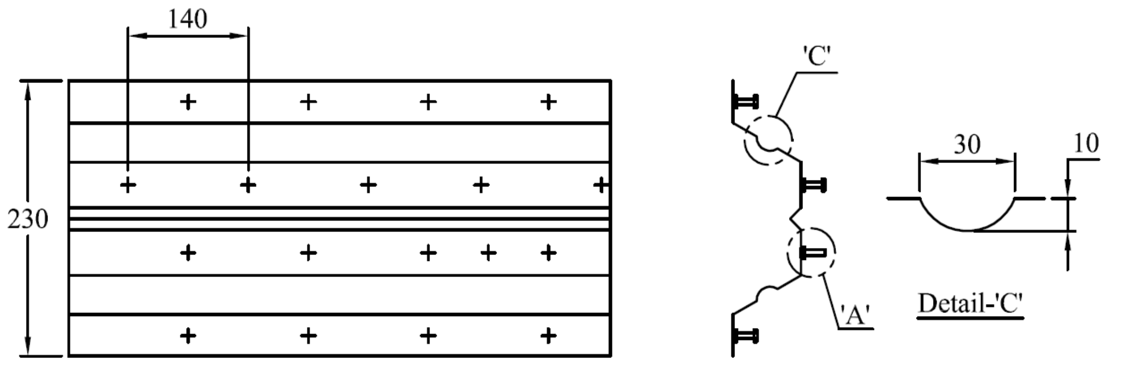


FIGURE 5. 10 (c) Interface Topology CB-5-6

All Dimensions are in mm

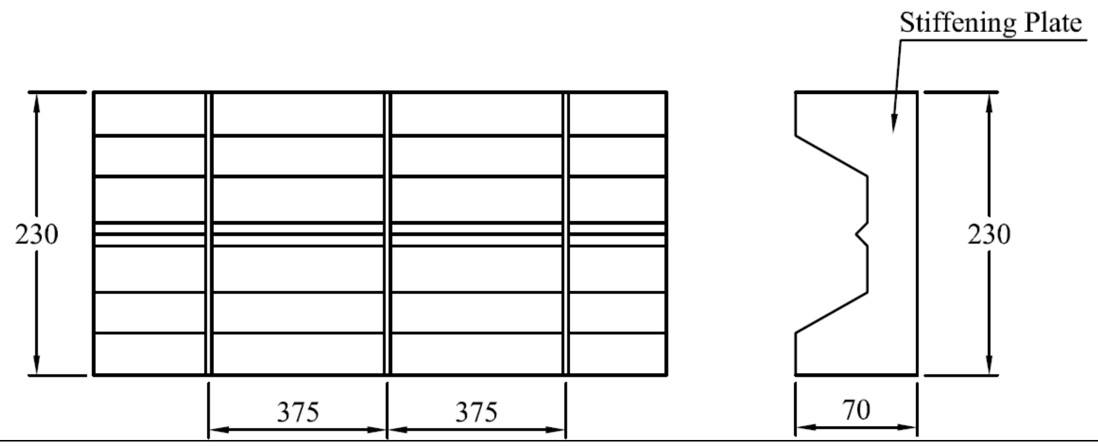


FIGURE 5. 10 (d) Interface Topology CB-7-8

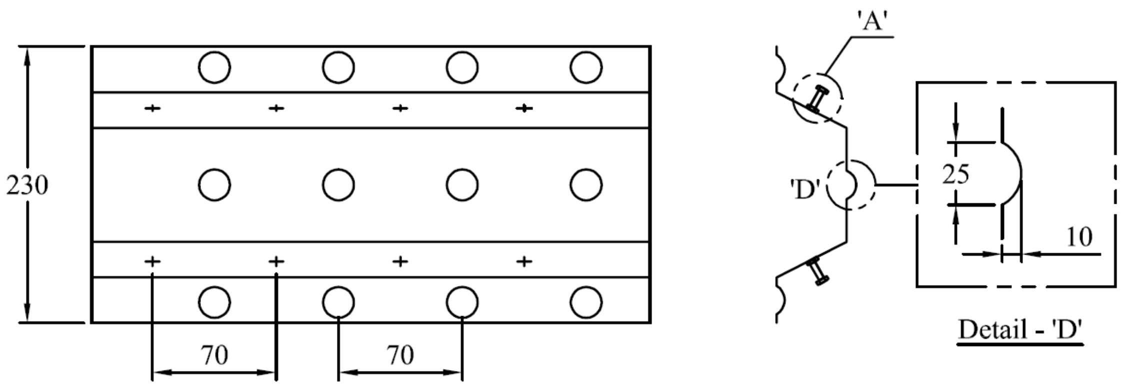


FIGURE 5. 10 (e) Interface Topology CB-9-10

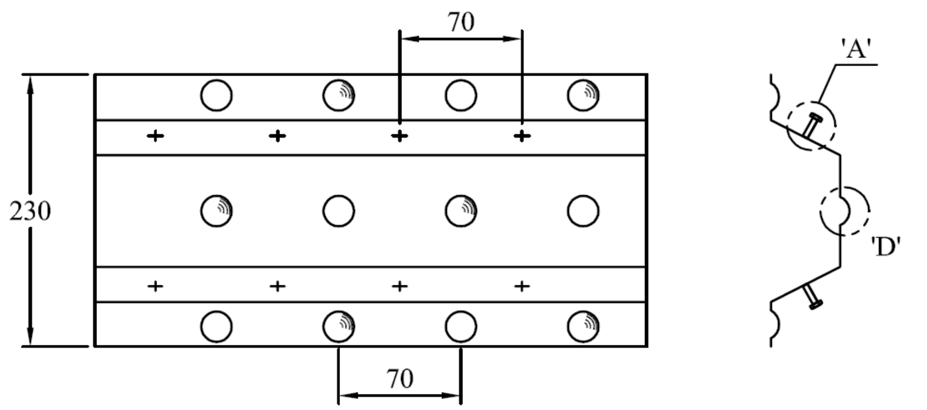


FIGURE 5. 10 (f) Interface Topology CB-11-12

All Dimensions are in mm

The specimens with the bolt head at interface were named as CB-1 and CB-2. The topology considered for these specimens were bolt head facing the top of the profile. Bolt connectors with 6 mm diameter and 22 mm height were used with 11 mm head diameter. The distance between two bolts on the webs was 70 mm, whereas on the flanges was 140 mm as zigzag pattern. Composite specimen CB-3 and CB-4, were similar to CB-1 and CB-2 in all manners except about the orientation of the bolts. In theses topologies, bolts were placed with the shank of the bolt facing profile deck. Case 3 specimens with circular arc bents in webs were named as CB-5 and CB-6. These specimens were having an arc bend on the web of the profile and bolt head on other faces. Specimens with straight stiffeners named as CB-7 and CB-8. For stiffeners, mild steel plates of size 230 mm x 70 mm x 2 mm were cut into a trapezoidal shape and welded at quarter distance from the centre span. No bolting has been done in this case. Case 5 is the topology with specimen denoted by CB-9 and CB-10. Pressed round embossment of 25 mm diameter and 10 mm depth were punched into the plain sheet at distance of 70 mm on flanges. Then the sheets were bent to trapezoidal shape and bolts were applied on webs of the specimen. CB-11 and CB-12 specimen were same as previous case, except the orientation of embossment. In this case alternate in and out embossment is done on profile deck. The yield strength of steel deck (CB-1 to CB-8) for bolting, bending and stiffening plate type bond patterns was 230 MPa. For the pressed embossment (CB-9 to CB-12), higher yield strength of 365 MPa was used to prevent tearing of the steel sheet due to embossment. The photographs of different bond patterns are shown in Appendix-II-A.2.2.

A total of twelve numbers of composite specimen on one wavelength width were prepared as per the parameters are shown in Table 5.4. Set of two specimen for each bond pattern were constructed. Steel decking surface was well cleaned before the casting of the concrete. Concrete mix for M-25 grade was designed as per Indian standards. After 28 days, concrete compressive strength was determined from testing concrete cubes 150 mm × 150 mm × 150 mm size. All composite slab specimens were casted with full support on the plain surface keeping geometrical parameters as per Euro standard guidelines. Composite specimens with 1.5 m span were tested under uniformly line loads in such a way that one-way bending takes place. Experimental set up of one wavelength specimen was kept same as, that of three wavelength specimen.

5.4.2 Experimental Set up and Instrumentation

The test set up and loading condition for Series # 2 experiments were same as series # 1. Composite specimen were simply supported on two external steel I sections and were loaded symmetrically with the uniformly spaced I-girders. Proving ring was kept between the hydraulic jack and the top cross girder. The graduation on the proving ring was adjusted to zero and then load was applied gradually from the manually operated hydraulic jack at constant interval of 5 divisions. Three deflectometers were placed beneath the bottom edge of the deck, one at mid-span and two at a quarter span of the slab. Two dial gauges were fixed on steel face and concrete to measure relative slip. The complete actual test set up is shown in the Fig.5.11.Total weight of I-girders and load cell was found between 1.16 to 1.5 kN. The center to center distance for the entire composite specimen was 1.42 m.

FIGURE 5.11 Actual Test Set Up: Series # 2

Load-deflection curves and Load-slip curves have been established for all twelve specimens as shown in Fig.5.12 to 5.17. For each specimen load at first crack, load at slip initiation, load at significant slip and maximum load are reported as per Table 5.5. For all the specimens, failure mechanism, composite action and separation between the steel deck and concrete have been observed. Significant observation on behaviour and failure is shown as per Table 5.6.

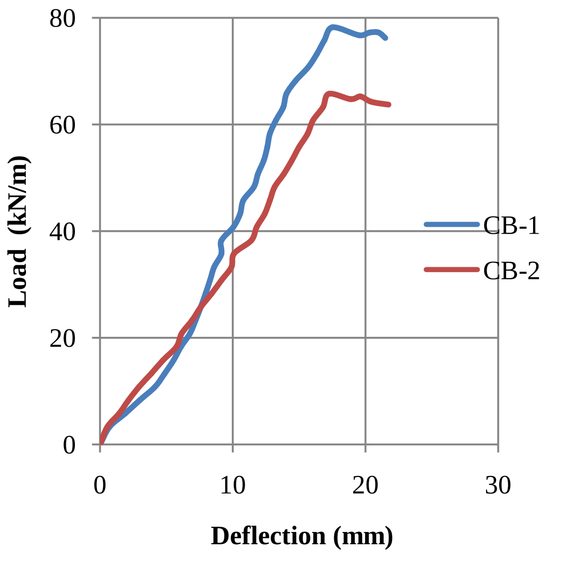


FIGURE 5.12 (a) Load Deflection Curve CB-1-2

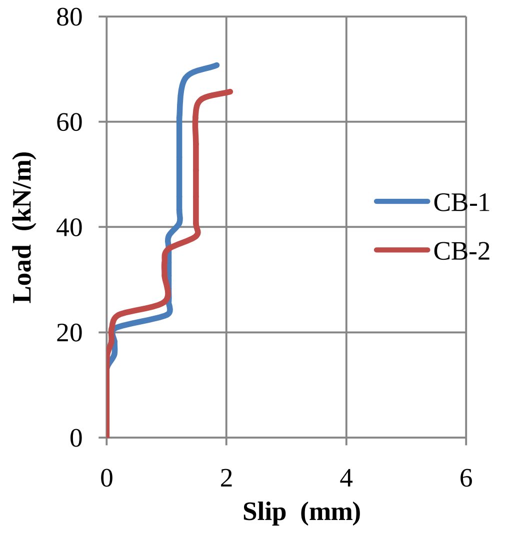


FIGURE 5.12 (b) Load Slip Curve CB -1-2

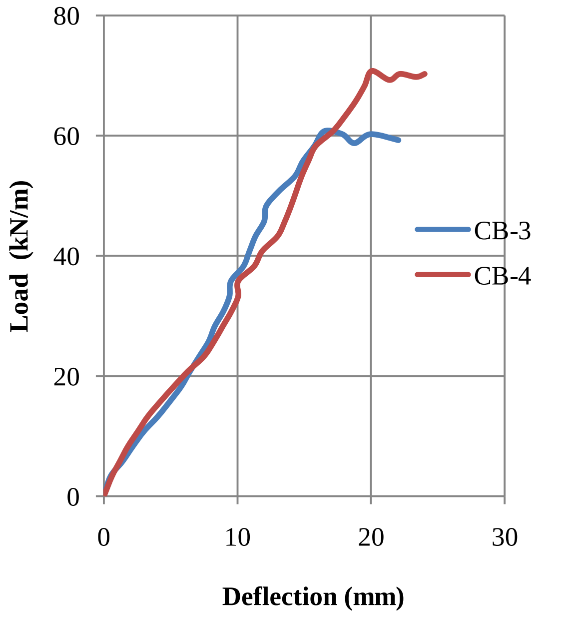


FIGURE 5.13 (a) Load Deflection Curve CB-3-4

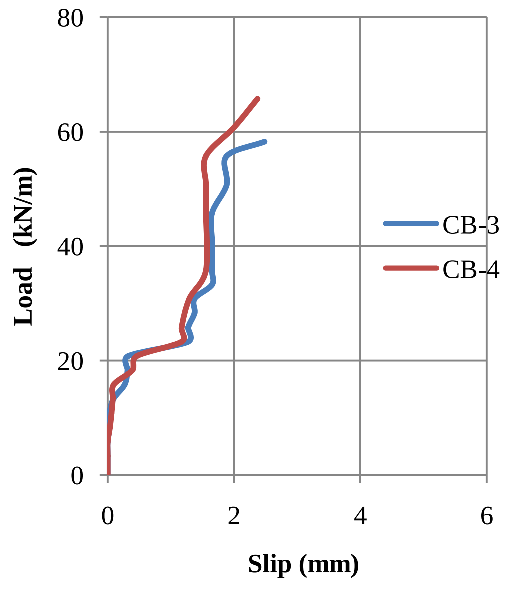
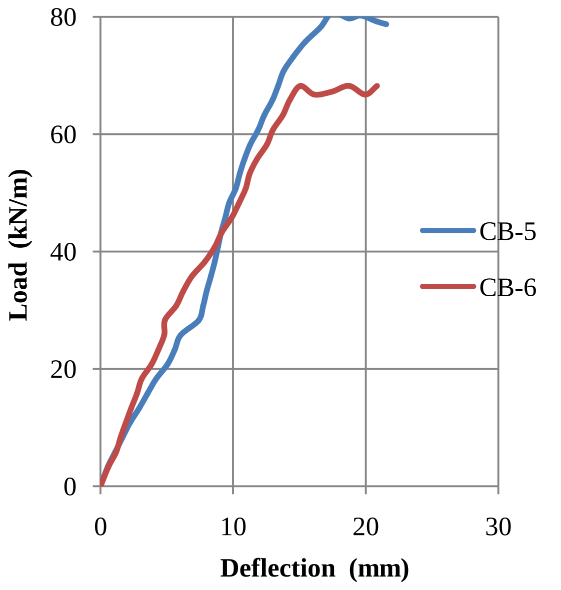


FIGURE 5.13 (b) Load Slip Curve CB-3-4



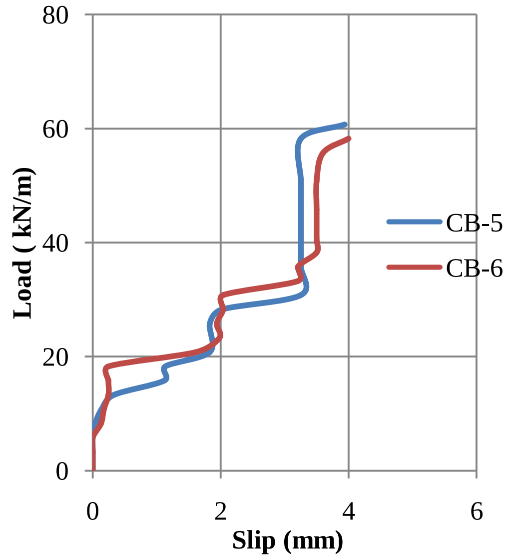
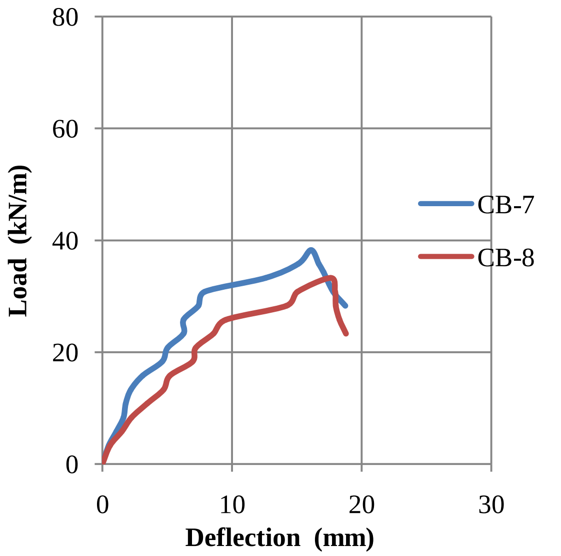


FIGURE 5.14 (a) Load Deflection Curve CB-5-6

FIGURE 5.14 (b) Load Slip Curve CB-5-6



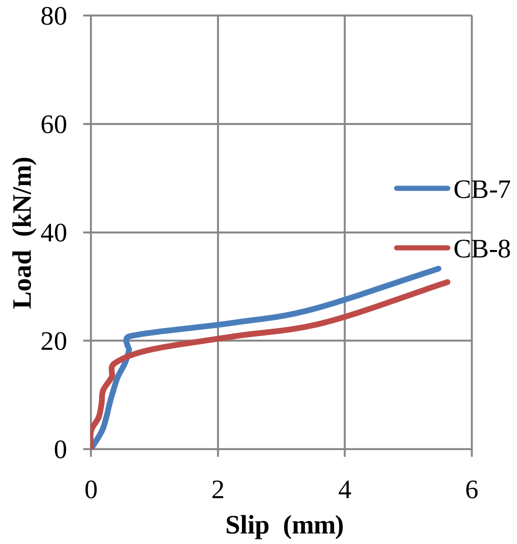
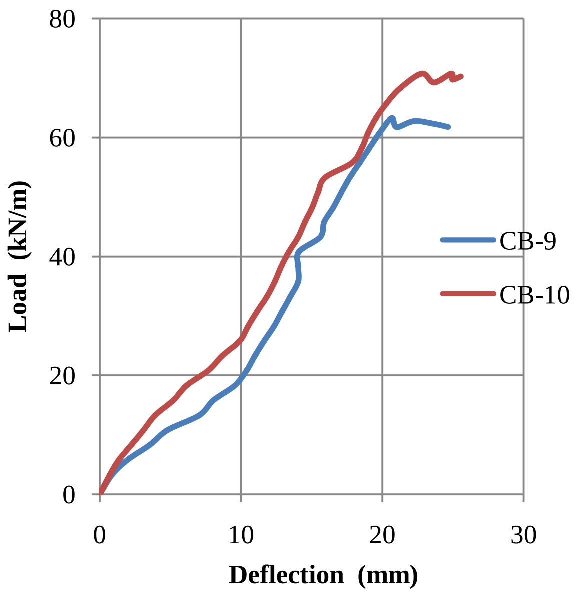


FIGURE 5.15 (a) Load Deflection Curve CB-7-8

FIGURE 5.15 (b) Load Slip Curve CB-7-8



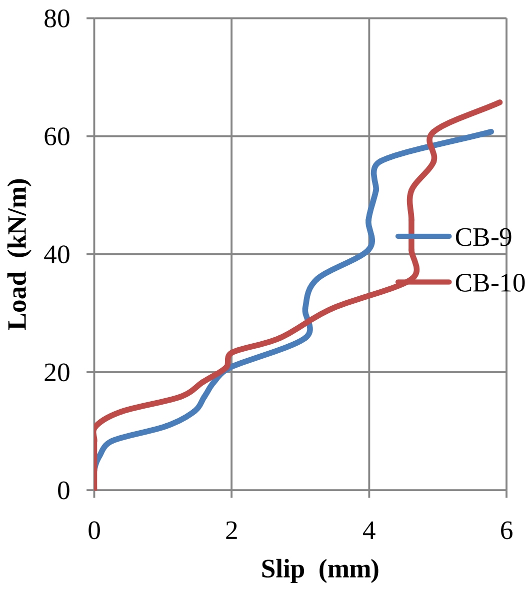
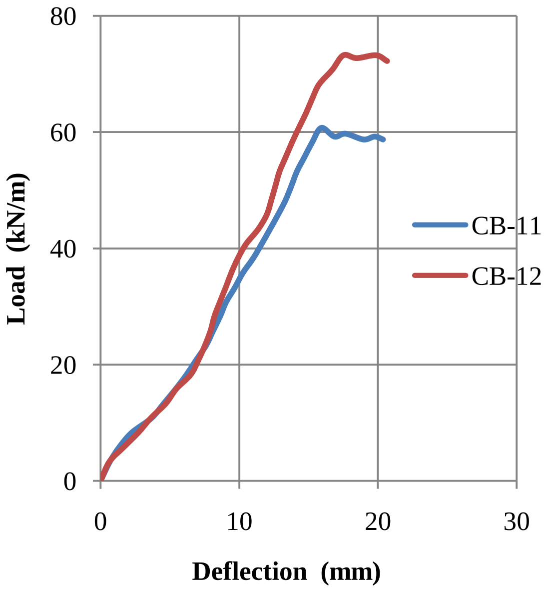


FIGURE 5.16 (a) Load Deflection Curve CB-9-10

FIGURE 5.16 (b) Load Slip Curve CB-9-10



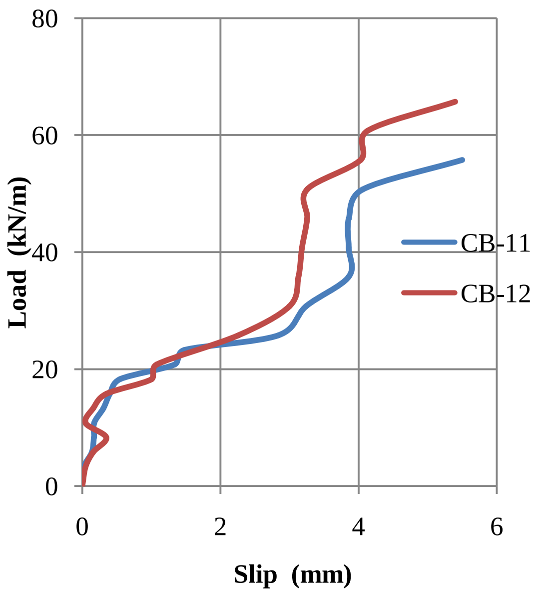


FIGURE 5.17 (a) Load Deflection Curve CB-11-12

FIGURE 5.17 (b) Load Slip Curve CB-11-12

Table 5.5 Significant Load and Deflection Values: Series #2

Specimen	Topology at interface	Avg. Load at first crack	Avg. Load at slip Initiation	Avg. Load at Significant slip	Avg. Maximum Load	Avg. Midspan Deflection	Avg. last Slip
		(kN/m)	(kN/m)	(kN/m)	(kN/m)	(mm)	(mm)
CB1-2	Bolt Head	15.79	24.53	39.51*	71.98	21.62	1.95
CB3-4	Bolt Shank	14.54	23.29	34.52*	65.73	23.02	2.43
CB5-6	Arc Bend and Bolts	20.79	18.29	32.02	74.48	21.20	3.97
CB7-8	Straight Stiffeners	9.55	22.04	24.53	35.77	18.78	4.73
CB9-10	Pressed in – in embossments	12.05	13.29	28.28	66.73	25.65	5.84
CB11-12	Pressed in – out embossments	14.54	19.34	30.78	66.98	20.50	5.45

* Potential slip < 3 mm

Table 5.6 Significant Observations: Series #2

Specimen	Topology at interface	Observation of slip	Observation of vertical separation	Failure
CB1-2	Bolt Head	Negligible	Negligible	Ductile
CB3-4	Bolt Shank	Negligible	Negligible	Ductile
CB5-6	Arc Bend and Bolts	Minor	Minor	Ductile
CB7-8	Straight Stiffeners	Major	Major	Brittle
CB9-10	Pressed in – in embossments	Major	Major	Ductile
CB11-12	Pressed in – out embossments	Major	Major	Ductile

5.4.3 Significant Observations: One wavelength Specimen

All specimens were tested under uniformly distributed load in such a way that one-way bending takes place. The full composite action was found in the loading range prior to the onset of cracking. Subsequently, separation between the steel deck and concrete was observed in the specimens. The load at various important points were converted including the loads of I-girder and load cell to kN/m. Experimental values at load at initiation of slip[50] (1 mm) and load at a significant loss of composite action (3 mm) (Patric,Brigge,1994) are tabulated as per Table 5.5. The strength of composite specimens at failure was very high in most of the cases, owing to the ductility of the system and post ultimate load behaviour of cold-formed steel sheet. It is also due to the addition of reinforcement for temperature and shrinkage which is provided below the calculated neutral axis of the specimen. Precise measurement of final end slip was not obtained because of vibration occurring in the loading frame so at last final slip readings were checked with Vernier calipers. Crack patterns and failure of specimen CB-1 to CB-12 are shown in Appendix-II.A.2.4.

The different slip values and behaviour of diverse bond patterns were observed from the graphs. The point at which the deflection plot first dropped corresponds to the point when the bond was broken. Additional load resistance was due to the friction between the deck and concrete, and the mechanical resistance of a particular bond pattern.

5.4.3.1 CB-1-2

These specimens with the bolt head at interface were tested in the same manner as test in series # 1.The load was applied at the increments of 5 divisions (3.54 kN). The first crack at the centre of the span was observed at the load of 15.79 kN/m. The slip was initiated at a load of 24.53 kN/m. Because of ductility and rigidity of bolt as a bond, not much interface slip was observed and the maximum slip measured was 1.95 mm. Maximum load of 71.98 kN/m was recorded for these specimens. No local buckling was observed in CB-1-2. Specimens showed no vertical separation and negligible horizontal separation.

5.4.3.2 CB-3-4

The concrete compressive strength was 32.58 MPa. The loading program was proceeded by load increment of approximately 5 divisions. The first crack was observed at 14.54 kN/m. The first flexural crack appeared in positive moment region. After that debonding noise was produced, which showed the breakdown of bond and slip was initiated at a load of 23.29 kN/m. Significant slip was observed at a load of 34.52 kN/m. Maximum load applied was 65.73 kN/m. CB-3-4 showed a flexure crack followed by diagonal crack near support. These specimens showed slip of 2.43 mm and negligible vertical separation.

5.4.3.3 CB-5-6

The test procedure and loading sequence was similar to that of CB-1-2 and CB-3-4 tests. Tested concrete strength was 33.48 MPa. Cracking in the flexure region occurred at a load of 20.79 kN/m. Before first crack, at load of 18.29 kN/m slip initiation was observed. It seemed that the first crack might have occurred internally before slip or might not be recorded earlier. Then after load was increased and at a load of 32.02 kN/m major slip occurred in the slab. Further, the load was increased to a maximum value of 74.48 kN/m. Specimen CB-5-6 showed a slip of about 3.97 mm and minor vertical separation.

5.4.3.4 CB-7-8

The loading sequence for this test was similar to previous tests. Concrete compressive strength for these specimens was 30.07 MPa. At a load of 9.55 kN/m minor first crack appeared. Then at load of 22.04 kN/m interface slip between concrete and steel sheet occurred. On minor increase of load as 24.53 kN/m again major slip was observed. Maximum load applied to the specimen was 35.77 kN/m with the measured slip of 4.725 mm.CB-7-8 failed suddenly with major separation and slip as depicted in Appendix II A.2.4. In this particular case, position of stiffening plate was in the same line of action as line load due to I-girder, which might have generated the region of stress concentration.

5.4.3.5 CB-9-10

The compressive strength of pressed in-in embossment specimen was 29.92 MPa and steel yield strength was 365 MPa. The first crack at 12.05 kN/m was occurred followed by first slip at load of 13.29 kN/m. Then load was increased and significant slip occurred at a load of 28.28 kN/m. Further, the load was increased by the maximum value of 66.73 kN/m. CB-9-10 specimen showed a major visible slip of about 5.838 mm as well as major vertical separation. The concrete of the slab over ride to the embossments in these cases. Although yield strength of the specimens was high, they failed at almost same load levels as specimen with bolts. These specimens showed major amount of slip and vertical separation as shown in Appendix-II A.2.4. In these specimens, the failure of the deck is occurred by formation of wedge-shaped cone of concrete around the embossment.

5.4.3.6 CB-11-12

The compressive strength of the concrete for these specimens was 30.21 MPa. The crack was obtained at load of 14.54 kN/m for pressed in-out alternate embossment case CB-11-12. Once the crack initiated, mechanical interlock started working for composite action. The slip was observed at 19.34 kN/m load. In previous case of in -in embossment the load at initial slip was 13.29 kN/m . At the load of 30.78 kN/m, significant slip is observed. Maximum load is applied to the specimen were 66.98 kN/m. Same as pressed in-in case, these specimen failed with major slip and separation.

5.5 Results and Discussions

The experimental studies have been performed on three and one wavelengths composite specimen with diverse bond patterns. The investigation was focused on effect of slip and completeness of interaction between steel and concrete. The initial slip and significant slip were measured at the end of the composite deck, which are important parameters for composite action.

Most of the one wavelength specimens with different bond patterns behaved in the same way as three wavelength specimen in terms of first crack, slip initiation and slip resistance. Different composite actions were achieved in both the series as types of mechanical interlock used were different. Both the specimens bend along the direction of rib of profile deck. Local bucking was not observed in series # 2 specimen because of use of 1 mm thick profile sheet. All tests of six sets of one wavelength specimen were identical in cross section, nearly same concrete strength, and loading condition. The members differed only in type of bond and higher steel grade in CB-9 to CB-12. It is significant to observe that , the maximum end slip of the member with the three wavelengths oval embossment was approximately four times the maximum slip of the member with the bolt head on top. The following section describes the behavior of three and one wavelength test specimen.

5.5.1 Results and Discussions: Three Wavelength Composite Deck

1. In the first phase of experiments on three wavelengths (Series #1), it was observed that, among all the four bond patterns, chemical bond specimen (CS-3) showed brittle behaviour with least load carrying capacity, maximum slip and sudden loss of composite action. Slip reading for specimen CS-3B was not recorded, an instrumentation error was suspected to have occurred.
2. Maximum load carried by all the specimen was 2 to 2.6 times higher as compared with load at slip initiation.
3. Most of the specimens in series #1 (CS-1 to CS-4) failed by significant slips recorded and small cracks due to bending in the middle of the span.
4. No attempt was made to measure the vertical separation. However, it was observed that the concrete and the decks were separated vertically as a result of concrete overriding in CS-1 (Oval embossments).
5. Hemisphere type bond pattern (CS-2) developed good composite action with negligible slip and no vertical separation. The aforesaid pattern shows 8% improvement in composite action as compared with oval shape embossment, 41% with respect to chemical bond and 20% improvement over cross stiffener case.
6. In specimen with the hemisphere, there was almost full composite interaction while comparing initial slip and significant slip, with 40% slip resistance and 1.9 mm end slip. However, lesser maximum load carrying capacity was observed as compared to cross stiffeners. The reason for its comparative low load carrying capacity is because of decrease in the strength of parent plate due to welding.
7. Hemisphere case composite specimen CS-2 showed considerable slip resistance from initial slip to significant slip by 40% as compared to only 19 % for oval embossment & chemical bond case and 35% in cross stiffener case.

5.5.2 Results and Discussions: One Wavelength Composite Deck

Most of the specimens were having nearly identical concrete strength and past studies show that concrete strength has no significant effect on composite action. Decks with measured concrete strengths of 25 MPa to 33 MPa behaved almost in a similar way.

1. No curling away tendency was noticed in one wavelength test specimens. This reduces the observed strength of the slab in the test but may not be of importance in

practical situations where the breadth of the slab formed by several sheets side by side is effectively very wide.

2. Bolt head at interface bond pattern (CB-1-2) developed good composite action with negligible slip and no vertical separation. The aforesaid pattern showed 25% improvement in composite action as compared with arc bend and 45% with respect to in - in embossment.
3. The composite specimens with lateral stiffening plate (CB-7-8) failed in a sudden manner, showing potential crack at the location of the stiffener.
4. The failures of the specimens with pressed in-in (CB-9-10) and in - out embossment (CB-11-12) were found with major slip and vertical separation. In spite of having high yield strength of sheets and higher cost as compared to other systems, much higher load carrying capacity was not observed in pressed embossment decks. These specimens failed by loss of bond between the deck and concrete with a diagonal tension crack forming at approximately at quarter of the slab span. When the specimen failed by bond, the concrete slipped along the quarter span causing cracking at the critical section. The crack grew until top fiber. The concrete did not fail by crushing.
5. No notable difference is found between the performance of the "in in" and "in and out" patterns. Therefore, the selection for production should be made based on cost and ease of fabrication.
6. The strength of composite specimens at failure was high in most of the cases, owing to the ductility of the system , flexibility in one wavelength and reserved strength. It is also due to the addition of reinforcement for temperature and shrinkage which is provided below the calculated neutral axis of the specimen. Also, because of providing the number of line load which relates to uniformly distributed load, the higher load carrying capacity was observed.

The experimental investigations show that different mechanical interlocking systems exhibit different composite action and different failure modes. Three wavelength specimens CS-3A-3B with chemical bond and one wavelength specimen CB-7-8-vertical stiffner at interface exhibited brittle failure. This gives an idea to the user that profile sheet used in construction without any mechanical interlock or with inferior mechanical interlock can lead to sudden failures. In cases of all diverse bond patterns, specimens with a chemical bond, straight stiffeners and pressed embossment showed major slip and vertical separation. So the aforesaid bond patterns are not suggested for further development.

Composite specimen with bolt as a bond pattern had favorable ductile behavior, maintaining significant additional load capacity of about 61% after the initiation of slip. The insertion of bolt, as bond pattern provided restraint against slip and higher end slip has been delayed from occurring. The deck with proposed bond pattern showed much more improved composite performance over deck with the oval shape embossments. The comparison between three wavelengths and one wavelength can be done by comparing the flexural capacities of both the series analytically and experimentally, which is discussed in chapter-6.

CHAPTER-6

Analytical Studies on Composite Deck

6.1 General

Many researchers have performed mathematical analysis on the results of the composite slab-derived from experiments. Different design and strength prediction methods were evaluated in many parts of the world. Most of these methods were related to m-k coefficients which in turn required test values from a minimum of six tests. The analytical study considering the effect of composite action for flexural capacity is scant. In this chapter, various analytical methods are reviewed considering effect of composite action. It is absolutely essential to have general slab strength formule else the deck manufacturers are faced with endless testing for new products. Reasonable understanding of how certain panel parameters affect performance of the deck allows orderly design and then testing can be minimised for verification of the expected test results.

6.2 Objective of Analytical Studies with Composite Action

The objective of this chapter is to review the various analytical methods for evaluating the performance, including the interface interaction property in analysis. The flexural capacities are calculated from no bond to full bond case. The study also proposes analytical method for external bond pattern with the concept of composite beam analogy. The analytical study is conducted to determine effect of various bond patterns on flexural capacity of composite slab.

6.3 Composite Action in Deck

Calculated connection strength shall be based on models that satisfy equilibrium of internal forces and limitations of strength of component materials based on potential failure modes. Force transfer between structural steel and concrete in composite deck is only to be considered to occur through direct bearing and/or friction. Studies on composite slabs revealed that the actual load capacity of the slabs is very high compared to the standard design loads. Generally, the load capacity of composite slabs is greater than required for the intended use [51].The collapse load may be greater than or equal to the calculated value. Most of these solutions do not coincide with upper-bound solutions [52]. Once the slip initiates, the mechanical interlock starts acting and at the significant slip composite deck losses composite action. Then the system will have two neutral axes separately for steel and Concrete. The load carrying capacity of composite slabs is dictated either by the bond - enhanced by interlock or by yielding of the decking. From tests of previous research, it is known that the slip initiates at 1 mm and there is significant loss of composite action at 3 mm when bond generally breaks down [49]. An initial slip, which is associated with the breakdown of the chemical bond, may occur at a lower level of load. The interlock resistance is therefore due to the performance of the bond in the deck, which cause the concrete to ‘ride-over’ the decking [4].To analyse the strength of deck considering the effect of composite action, five different methods were studied for the decks under consideration.

6.4 Analytical Approaches for Flexural Capacity

The analytical flexural capacity of the decks is predicted using five different approaches (i) Full bond as per Euro standard (ii) No bond (iii) First Yield Approach (iv) Luttrell’s Lug Factor Approach (v) Modified Composite Beam Approach. As there are variations in parameters of the composite deck such as concrete strength, steel strength and sheet thickness, flexural capacitates of different specimens are calculated analytically considering all material and geometric variations. To facilitate comparison of these methods with experimental results, experimental calculations were made by comparing the moment of a uniformly loaded simply supported specimen at slip initiation and significant slip. For three wavelength specimens, series #1 and one wavelength specimens in series #2, flexural capacity is found based on loads and effective span lengths. Properties of the bond patterns required for analytical methods are taken from the data of experiments as mentioned in chapter 5.Programs are developed for all five discussed method as listed in Annexure -III.

6.4.1. Full Interaction : Euro Standard (M_{tf})

Fully composite interaction provides enough mechanical interlock so that flexural strength is controlled by the full strength of steel and concrete working together. Calculations for flexural capacity "M_{tf}" by Euro standards [40] considering the full interaction between concrete and steel is made as per discussions in chapter 4. Flexural strength of deck is limited in case of poor mechanical interlock. In that case, significant slip is anticipated at the interface.

6.4.2. No interaction ((M_{tn})

In case, if there is no mechanical interlock present at the interface or if interface is oiled, steel and concrete will behave independently. In such case, only strength of the steel deck is contributing to flexural strength. Flexural capacity of steel deck can be found out using simple bending theory as per Eq.6.1.

$$M_{tn} = \frac{I_{Deck}}{y_{Deck}} * f_{Deck} \qquad \text{(Eq.6.1)}$$

Where,

M_{tn}	=	Flexural capacity for no bond in kN.m/m
I_{Deck}	=	Moment of inertia of profile deck in mm^4
y_{Deck}	=	Distance from the extreme fiber to centroidal axis of profile deck in mm
f_{Deck}	=	Strength of steel deck in MPa

6.4.3. First Yield Approach (M_{ty})

The flexural capacity of the slabs is predicted, using the moment at first yield. First yield method was developed by Heagler[9] (1993) for the flexural capacity of composite deck. It is based on a transformed area and by dividing the tensile force of the deck to each of the flanges (T_1, T_3) and the web (T_2) separately as per Fig.6.1.This procedure gives three tensile forces with their respective moment arms (e_1, e_3, e_2), This development is particularly advantageous for predicting the performance of a deck considering steel at different levels. This method limits the predicted strength of the composite slab, which is considered fully composite, to the load that causes first yield of the bottom flange of the deck. The concrete is assumed to have cracked so that only concrete above the neutral axis contributes to compressive strength and the tensile loads are distributed into components in the top flange (T_1), the web (T_2), and the bottom flange (T_3) of the deck as per Eq. 6.3, 6.5 and 6.7 respectively. The forces are assumed to act through moment arms e_1 ,e_2 , e_3 on the top flange, web and bottom flange of the deck, respectively, as per Eq. 6.4, 6.6 and 6.8. The method predicts the performance of a deck considering steel at different levels. The effectiveness of protrusion is not considered in this method. Variables of the equations are shown in Fig. 6.1.'n' is modular ratio and 'ρ' is ratio of steel to concrete cross section.

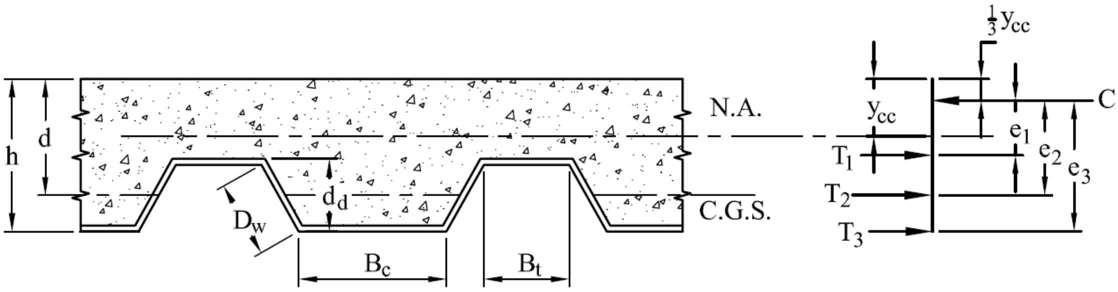


FIGURE 6.1 First Yield Approach

$$M_{ty} = (T_1 e_1 + T_2 e_2 + T_3 e_3) \quad \text{(Eq.6.2)}$$

$$T_1 = f_{yc} (B_t * t) \frac{h - y_{cc} - d_d}{h - y_{cc}} \quad \text{(Eq.6.3)}$$

$$e_1 = e_3 - d_d \quad \text{(Eq.6.4)}$$

$$T_2 = f_{yc} (2 * D_w * t) \frac{h - y_{cc} - \frac{d_d}{2}}{h - y_{cc}} \quad \text{(Eq.6.5)}$$

$$e_2 = e_3 - \frac{d_d}{2} \quad \text{(Eq.6.6)}$$

$$T_3 = f_{yc} (B_c * t) \quad \text{(Eq.6.7)}$$

$$e_3 = h - \frac{y_{cc}}{3} \quad \text{(Eq.6.8)}$$

$$f_{yc} = f_y - f_c \quad \text{(Eq.6.9)}$$

$$y_{cc} = d (\sqrt{2\rho n + (\rho n)^2} - \rho n) \quad \text{(Eq.6.10)}$$

6.4.4. Luttrell's Lug Approach (M_{tl})

This method was developed by Luttrell and Prassanan [8], [23] at West Virginia University and is based on the slab behavior found from numerous full-scale slab tests of deck with various characteristics. A statistical analysis was performed on the test results to determine the effect of the various deck characteristics on strength, resulting in the development of factors. The method is based on determination of flexural capacity of the deck by applying three "relaxation factors", which influences shape, deck dimension and bond/protrusion configuration. Lurell reconsidered the assumption that in the flexure mode, the slab behaves like a reinforced concrete section with the deck's tensile force acting at its centroid. He argued that the steel deck behaves differently than embedded reinforcing bars because the deck is only bonded on one surface and is free to deflect on the other surface. Therefore, the geometry of the deck has a great effect on the slip resistance. In this study generalized program for flexural capacity is developed considering Lutrell approach for any means of mechanical interlock. The flexural capacity of the composite deck is found by Eq.6.11 to 6.18. "k" reflects the influences as shape, deck dimension and bond configuration. Deck width parameter is also considered in factor 'k'. Variables for profile and bond details are as per Fig.6.2.

$$M_{tl} = k\, M_{tf} \qquad \text{(Eq.6.11)}$$

$$k = \frac{k_3}{k_1 + k_2} \qquad \text{(Eq.6.12)}$$

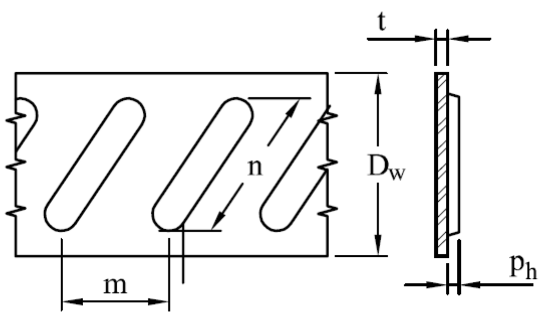


FIGURE 6.2Lutrell's Approach: Bond Details

Relaxation Factor:

$$Ps = 12 * \frac{n}{m} \qquad \text{(Eq.6.13)}$$

$$\text{If } P_s * P_h < 0.6$$

$$k_1 = \frac{1}{\sqrt{D_d}} + \frac{t}{P_h D_d} \qquad \text{(Eq.6.14)}$$

$$k_2 = \frac{100\,(t)^{1.5}}{D\sqrt{P_h}} \qquad \text{(Eq.6.15)}$$

$$k_3 = 0.87 + 10^{-3}\left(\frac{B}{Bc}\right)\left(69 - 2.2\frac{B}{Bc}\right) \qquad \text{(Eq.6.16)}$$

$$\text{If } P_s * P_h > 0.6$$

$$k_1 = (t - 0.03)\left(1700\, Ph^2 \frac{\sqrt{P_s}}{\sqrt{D_d}} - 32\right) + 2.4 - \sqrt{P_s * P_h} \qquad \text{(Eq.6.17)}$$

$$k_2 = \frac{(D - 27.5\sqrt{t})\sqrt{P_s}{D_d}^2}{1316\,(0.01 + 0.85P_h)} \qquad \text{(Eq.6.18)}$$

Where,

D_d = Depth of profile sheet in mm/inch

D = Overall depth of deck in mm/inch

t = Thickness of profile sheet in mm/inch

B = Width of one cell in mm/inch

B_c = Total width of deck in mm/inch

The method discussed takes into account most of the geometrical parameters of profile deck and bond patterns.

6.4.5. Modified Composite Beam Analogy (M_{tm})

Composite slabs are made from similar components as a composite beam, namely a steel section (profiled sheet) and a concrete slab which are connected to resist longitudinal slip. It is therefore assumed that a composite slab with bolts as connectors, will behave as a composite beam. In the calculation, the connectors for full interaction and connectors provided are put as input values. The flexural capacity of the composite deck is found out by Eq.6.19.

$$M_{tm} = M_{tf} + \frac{N_P}{N}(M_{tf} - M_{tn}) \qquad \text{(Eq.6.19)}$$

Flexural capacity of steel deck M_{tn} is found out by knowing the inertia property, centroidal axis and yield strength of steel (As per section 6.4.2). Flexural capacity of composite slab M_{tf} is found by considering full interaction between steel and concrete as per Euro standards (As per section 6.4.1).'N' provides number of connectors required for full interaction. 'N_P' is a number of connectors provided. 'N' can be calculated by estimating longitudinal force which is same as compressive force at the interface. As per Ollgaard et al. (1971)[53], the capacity of connector depends on the concrete compressive strength, elastic modulus, shank area and tensile strength of the connector as per Eq. 6.20.

$$P_u = 0.5A_{sc}\sqrt{f'_c\,E_c} \leq A_{sc}\ F_u \qquad \text{(Eq.6.20)}$$

Where,

A_{sc} = Area of connector

f_c' = Compressive strength of concrete

E_c = Modulus of elasticity of steel

F_u = Tensile strength of connector

If the strength of the external mechanical connector is known, Eq.6.19 of composite beam analogy can be modified by incorporating the width of the deck. As reported by Prasannan (1983) the effect of number of ribs as a width factor can be taken into consideration and the flexural capacity of the composite deck is found out by Eq.6.21.

$$M_{tm} = k_w\left[M_{tn} + \frac{N_P}{N}(M_{tf} - M_{tn})\right] \qquad \text{(Eq.6.21)}$$

where $k_w = 0.87 + 10^{-3}\left(\frac{B}{Bc}\right)\left(69 - 2.2\frac{B}{Bc}\right)$

Where the 'k_w' in the Eq.6.21 is adopted from Luttrell approach.Developed composite Beam Analogy method can be applied considering the effect of actual connectors and connectors for full interaction in relevant cases. Further improvisation in the method is possible based verification from numbers of test data.

6.5 Results: Analytical Approach

Different methods of analysis for flexural capacity have been studied. Lutrell lug approach and Composite beam analogy approach consider the effect of the composite action in flexural capacity. Another method such as Euro standard and first yield method analyses flexural resistance considering full composite action. The aforesaid methods are used as one of the input parameters for Lutrell lug approach and Modified composite beam analogy approach. Modified composite beam analogy approach is used only in the cases with bolt as connectors. Lutrell lug approach is considered for the specimens with known dimensions of protrusion, so these methods are used in appropriate cases only in this work. Representation of analytical results is as per Table 6.1.

Table 6.1 Analytical Flexural Capacities with Different Approaches

Specimen	Mt_f (kN. m) Full bond: Euro Code	Mt_n (kN. m) No bond	Mt_y (kN.m) First yield approach	Mt_l (kN. m) Lutrell's approach	Mt_m (kN. m) Modified composite beam analogy
CS1A-B	14.22	4.36	9.91	11.80	--
CS2 A-B	14.22	4.36	9.91	9.34	--
CS3 A-B	14.22	4.36	9.91	--	--
CS4 A-B	14.22	4.36	9.91	--	--
CB1-2	5.34	1.32	3.65	5.07	4.69
CB3-4	5.33	1.32	3.65	5.13	4.12
CB5-6	5.34	1.32	3.65	--	--
CB7-8	5.29	1.32	3.65	--	--
CB9-10	7.93	2.09	5.78	4.61	--
CB11-12	7.94	2.09	5.78	4.62	--

The Lutrell's method is slightly more calculative than the First yield method but seems more logical for the prediction of flexural capacity because it includes deck and bond pattern factors that describe the composite action of the specific deck under consideration. Modified composite beam analogy method can be used if the bond between steel and concrete is through external means. In this research the method is extended for composite deck and also modification is made considering width factor into account which makes the method reasonably accurate. The first yield method gives conservative result of flexural capacity considering full bond as compared to Euro code.

6.6 Discussions: Analytical Approach

The strength prediction given by Euro code stress block theory gives the upper bound value. Lutrell's lug factor method is advantageous as it gives reliable lower bound strengths and it is easy to calculate. But if there is no proper quality control over protrusion or if product information does not provide sufficient details about protrusion, the method should be applied to check and compare the results of the small-scale experiments. Additional composite strength due to the continued slip resistance of the bond after slip initiation is not incorporated in above analytical methods. However, these methods are useful to decide the lower bound strength value for determining the behavior of the slab, instead of just at failure. For design purposes, it is recommended to use modified composite beam analogy method if external means of bond pattern is applied or Luttrell approach if dimensions of bond patterns are known and if there is uniformity in protrusion.

6.7 Comparison of Analytical and Experimental Flexural Capacity

Experimental moments are calculated at quarter span, which is a critical case [42] for composite slab loaded with uniformly distributed load. Experimental results are found for moment at slip initiation (Me_i) and moment at significant slip (Me_s). Analytical methods as per Table 6.1 are listed for comparison. Ratios of experimental and analytical values are tabulated. Ratio of maximum experimental moment (Me_m) to Theoretical moment under full bond (Mt_f) is included.The comparison between experimental flexural capacity at slip initiation, potential slip and analytical capacity is presented in Table 6.2.

Table 6.2 Analytical and Experimental Flexural capacity

Specimen	Me_i (slip-i)	Me_s (slip-s)	Mt_f Euro Code	Mt_l Lutrell's approach	Mt_m Modified composite beam analogy	Mt_n No Bond	$\frac{Mei}{Mtn}$	$\frac{Mei}{Mtf}$	$\frac{Mem}{Mtf}$
	Experimental (kN.m)		Analytical (kN.m)				Ratio		
CS1A-B	7.83	9.38	14.22	11.80	--	4.36	1.98	0.55	1.47
CS2 A-B	8.51	11.98	14.22	9.34	--	4.36	2.15	0.60	1.17
CS3 A-B	6.00	7.16	14.22	--	--	4.36	1.52	0.42	0.95
CS4 A-B	7.06	9.57	14.22	--	--	4.36	1.79	0.50	1.21
CB1-2	4.64	7.47	5.34	5.07	4.69	1.32	3.87	0.87	2.55
CB3-4	4.40	6.53	5.33	5.13	4.12	1.32	3.67	0.83	2.33
CB5-6	3.46	6.05	5.34	--	--	1.32	2.89	0.65	2.64
CB7-8	4.17	4.64	5.29	--	--	1.32	3.48	0.79	1.28
CB9-10	2.51	5.35	7.93	4.61	--	2.09	1.32	0.32	1.59
CB11-12	3.66	5.82	7.94	4.62	--	2.09	1.92	0.46	1.59

Comparison shows that specimen with bolt patterns CB-1-2 attains almost full flexural capacity. These specimen also resist significant slip after slip initiation. The comparison between experimental flexural capacity at slip initiation and theoretical capacity with full bond and no bond are presented in Fig.6.3 and Fig.6.4

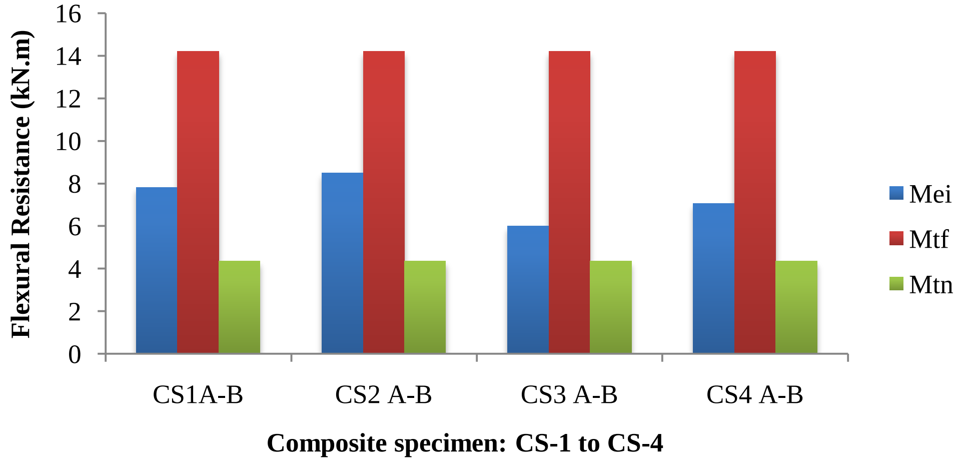


FIGURE 6.3 Flexural Resistance - Specimen CS

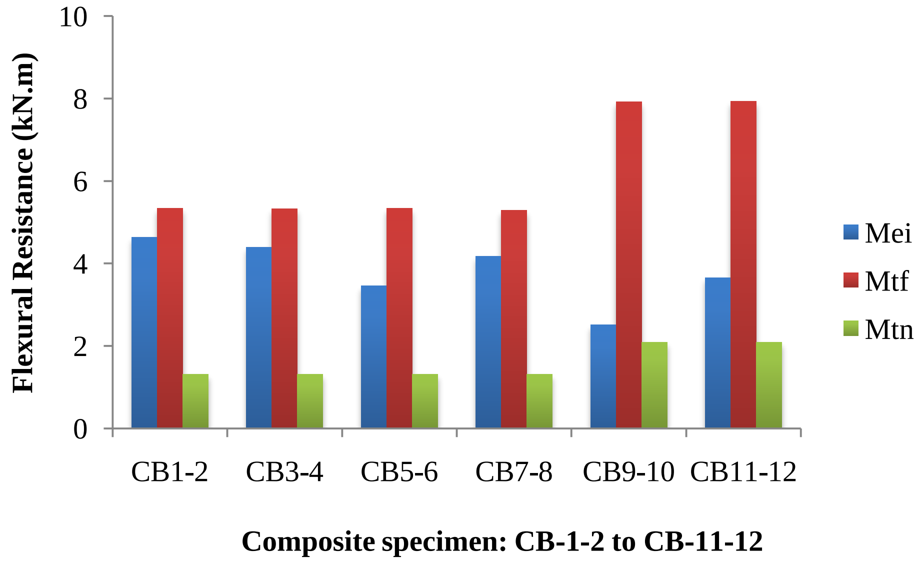


FIGURE 6.4 Flexural Resistance - Specimen CB

6.8 Discussions: Analytical and Experimental Flexural Capacity

1. Comparison between three wavelengths and one wavelength patterns shows, one wavelength specimens with suggested patterns behaved in the same manner and gives satisfactory results.
2. Because of comparatively weak mechanical interlock, three wavelength specimens did not show good composite action and the ratio of experimental flexural capacity to analytical full bond flexural capacity ranges between 0.42 to 0.6.Similar response is observed in one wavelength specimen with weak interlocks. Ratio of experimental to analytical full bond flexural capacity for one wavelength specimen CB-9 to CB-12 also ranges between 0.32 to 0.46.
3. The insertion of external means of interlocking can significantly improve composite action of poorly manufactured profile deck. Otherwise, deck behaves as 'No bond' case and flexural capacity significantly reduces by one-fourth to that of 'Full Bond'.
4. As shown in Fig.6.4, experimental flexural capacities of one wavelength composite specimen CB-1 to CB-4 are almost same as analytical flexural capacity under full bond.
5. In specimen with bolt head at interface CB-1-2 superior ductile behaviour and composite action are observed with the ratio of experimental flexural capacity to analytical full bond flexural capacity as 0.87.The specimen has also maintained significant additional load capacity after the initiation of slip.
6. For specimen with bolt as interface topology, good agreement is observed between experimental (M_{ei}) and analytical flexural resistance using Modified composite beam analogy (M_{tm}).
7. Specimens with the proposed bolt pattern provide significantly improved composite interaction; it can be further analyzed from the point of view of commercial applicability.
8. The one wavelength test is quick, simple and economical to perform and can yield essential information about the composite action. Small scale one wavelength test is a proposed for evaluating the composite action of deck, which can be simply implemented by Indian small scale industry. It can also be used to develop new mechanical interlock pattern by the local user without much cost escalation.

9. Analytical approaches such as Lutrell's method and Modified composite beam analogy method takes into consideration the width of the specimen and bond pattern details, which can be used to verify experimental flexural capacity of the composite deck.

CHAPTER-7

Conclusions

7.1 Summary

This research work comprises of studies on theoretical as well as experimental flexural capacity of the composite deck and effectiveness of bond. In that regard, a comprehensive review is carried to study governing parameters and flexural capacity with various codes of practice. The standards investigated include: Euro standard EN 1994-1-1:2004, British standard BS-5950: Part-IV,1994 and Steel Deck Institute-ANSI-2011.Based on the code studies, generalized program is developed to analyse flexural capacity and neutral axis for comparative and parametric variations. The developed program is compared with 'ComFlor' software. These studies are useful to estimate flexural capacity of any configuration and any set of data.

Experimental work is carried out on three wavelengths and one wavelength composite specimen with diverse bond patterns subjected to line loads. In cases of all diverse bond patterns, specimens with a chemical bond, straight stiffeners and pressed embossment showed major slip and vertical separation. Specimens with bolts have better composite interaction over all other bond patterns. Results of one wavelength test with ductile failure showed good agreement with three wavelength tests. However, because of comparatively weak mechanical interlock, three wavelength specimens did not show good composite action. Analytical strength prediction models for flexural capacities are investigated from 'no bond' to 'full bond' cases incorporating bond factor.

Comprehensive discussions on the outcome of research and relevant results are listed at the end of Parametric, Experimental and Analytical studies chapters.

7.2 Conclusions

This study involves evaluation of flexural strength of composite deck system analytically and experimentally. Studies are carried out on existing international standards, considering complete interaction. Experiments are conducted with different bond patterns to achieve the better composite action. The effect of composite action is studied analytically by various strength predictions procedures.The following conclusions are drawn based the observations from studies:

1. Comparative studies of International standards demonstrate that Euro and British standards estimate 4 % and 6% higher value of flexural resistance respectively, as compared to Indian standard stress block.
2. Parametric studies indicate that estimated value of flexural resistance increases by 64.78% and 57.10% on increasing steel grade and overall concrete depth respectively. However, increase in concrete grade does not show noteworthy increase in value of flexural resistance.
3. Neutral axis factors are developed to verify under-reinforced section theoretically. Steel grade of 365 MPa is optimum for analysed deck. Whereas use of 450 MPa steel grade makes the section over - reinforced, which can trigger brittle failure.
4. A generalized program is developed to perform comparative and parametric studies on flexural capacity of composite deck which can be used to analyse any variation of profile deck geometry and material parameters. Guidelines are prepared for limiting parameters, flexural resistance and neutral axis as per Indian scenario.
5. Experimental and analytical studies show that three wavelengths and one wavelength specimen with suggested bond patterns behaved in a similar manner. However, three wavelength specimens showed inferior composite action as their experimental to analytical flexural capacity ratio is observed as 0.42 to 0.6.
6. In cases of all diverse bond patterns, specimens with bolts head at the interface have significantly improved composite interaction as compared to other patterns. These specimen provided slip resistance of 61% and experimental to analytical flexural capacity ratio of 0.87.

7. One wavelength specimen is proposed to investigate composite action of a deck for any bond pattern. It reduces the experimentation cost without compromising with the bending behaviour and recommended for future development of any profile sheet.
8. Analytical strength prediction models such as Lutrell approach and Modified composite beam analogy approach are prescribed in relevant cases to verify test results. These approaches include bond properties and deck geometry into consideration.

7.3 Recommendations and Major Contributions

Economic advantages of the composite floor system are clearly evident. Standards and specifications are scattered amongst various brochures and design codes. A holistic study carried out in this research for flexural capacity of composite deck directs to following recommendations and contributions:

1. Flexural resistance of composite deck under full bond should be calculated based on partially parabolic and partially rectangular stress block, with the strain value '0.0035'. Factor of safety for steel is suggested as 1.15 for Indian design criteria. Ductility clause is not mentioned in most of the existing standards, except in American standard. Neutral axis factors are suggested for commonly used steel grades in profile deck construction, which should be included in Indian guidelines to ensure under- reinforced section.
2. Governing geometrical parameters such as depth of the deck, overall height of concrete, dimensions of web and flanges, profile depth and thickness should be considered same as Euro standards.
3. Strength of normal weight and light weight concrete cube should range from 25 MPa to 75 MPa and 22 MPa to 66 MPa respectively. The yield strength of the profile deck should range from 235 MPa to 460 MPa.This is in accordance with Euro code, with alteration of considering concrete cube strength instead of cylinder strength in design.

4. Developed programs as per Appendix - I should be used to analyse flexural capacity, neutral axis and parametric variations under full bond. These simple programs will assist the user/ structural designer to estimate flexural capacity of any configuration of the deck.
5. Experiments should be conducted on one wavelength test specimen for any existing/newly developed profile shape and bond variations to ensure proper composite action.
6. Upper and lower bound values of the flexural capacity should be estimated using 'Full bond' and 'No bond' approaches. Relaxation factor 'k' should be applied to incorporate bond properties and deck geometry. Simple programs for these approaches are developed as per Appendix -III. The program can be used to verify experimental results and to appreciate effect of bond on strength of composite deck.
7. General guidelines are prepared for composite deck construction. Design professional/ builder may review the same as per applicability to a specific job.

7.4 Future Scope of Research

The present study includes detailed analytical and experimental investigations on composite deck. There are several aspects that can be further investigated.

1. Analytical study with the International code can be further extended for other limit states. Computer programs can be further extended to compare other limit states.
2. Bond patterns with variations in number and spacing of protrusions can be further investigated through experiments.
3. Commercial adaptability of the bolt pattern can be studied. The combination of company manufactured bond pattern with bolt interface topology together can be investigated.
4. Efforts can be put in the direction of minimizing specimen size without compromising on bending behavior. Experiments can be designed with small size specimen that fits into standard Universal testing machine. This will reduce the cost of testing and also the need for sophisticated structural loading frame.

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List of Awards and Publications

1. **Principal Investigator, Awarded by GUJCOST Research Grant, Gujarat Council on Science and Technology, Department of Science and Technology, Government of Gujarat, (2015)**

2. 'Comparative study of moment carrying capacity of composite deck' , *Journal of information, knowledge and research in civil Engineering*, ISSN: 0975 –6744, Volume 2, Issue 1, pp. 60-62, (2012)

3. 'Structural cold formed steel, steel-concrete composite and structural stainless steel: A critical review of research and opportunities', *Structural Engineering Convention*, SEC-2012, S.V.National Institute of Technology, pp. 729-735, (2012)

4. 'Parametric Study of Open Trough Steel Concrete Composite Deck' , *Nirma Journal of Engineering and Technology*, ISSN: 2231-2870, Volume 3, No 1, pp. 12-14, (2014)

5. 'Load carrying capacity of composite deck', *EUROSTEEL*, *7th European Conference on Steel and Composite Structures, Italy*, 2014, pp. 591- 592,(2014)

6. ' Investigations on composite deck with different interface connections', *Journal of Structural Engineering*, Vol. 42, no. 5, pp. 386-392, (2016)

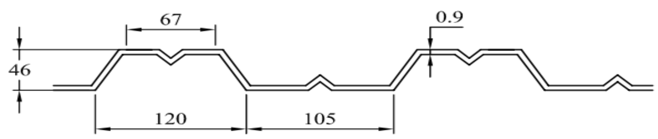
7. 'Flexural capacity of steel concrete composite deck: Analytical approaches considering the effect of composite interaction', Abstract accepted, The *Open Civil Engineering Journal*, *Bentham Publications*, (2016)

APPENDIX - I

Flexural Resistance of Composite Deck:

International Standards and 'ComFlor' Software

A.1.1 Flexural Resistance: EN-1994-2004



Type: Trapezoidal or Reentrant Composite Deck				
INPUT PARAMETERS				
Details	**Notations**	**Value**	**Unit**	**Specification as per EN-1994-2004**
Depth of profile sheet	d_p	46	mm	40 mm to 80 mm
Width of top flange	b_t	67	mm	
Width of bottom flange	b_c	105	mm	
Gross Thickness of sheet	t_g	0.9	mm	0.7 mm to 1.2 mm
One wave length	b_s	225	mm	
Ratio b_t/b_s	b_t/b_s	0.30		$b_t /b_s <=0.4$
Design thickness	t	0.86	mm	$(t_g$-0.04)
Angle of web	θ	59.89^0		$\theta=55^0$ to 90^0
Depth of composite Deck	D	110	mm	Minimum 80 mm
Length of web	D_w	53.09	mm	

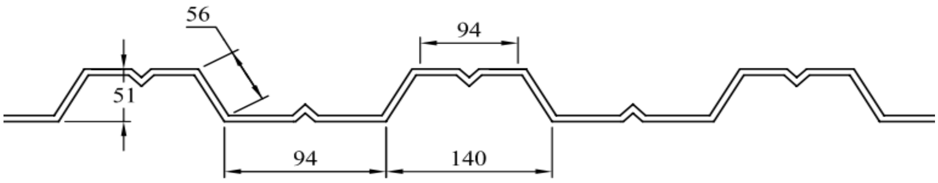
MATERIAL PROPERTIES				
Details	**Notations**	**Value**	**Unit**	**Specification as per EN-1994-2004**
Yield strength of steel	f_y	280	MPa	$f_y \geq 230$ MPa
Concrete cylinder strength	f_{cd}	25	MPa	(If cube strength is given, $f_{cd}=0.8f_{cu}$)
Concrete cylinder strength	f_{cu}	31.25	MPa	

CENTROID AND MOMENT OF INERTIA				
Elements	**Notations**	**Length**	**Distance of C.G. from top**	**Unit**
1:Top flange	b_t	67	0.45	mm
2:Two webs	D_w	106.17	23	mm
3:Bottom flange	b_c	105	45.55	mm
Length per meter	L_p	1236.33		mm per m
Area per unit wave length	A	250.36		mm^2 per one wavelength
Area per meter	A_p	1112.70		mm^2
Length per wave length	L	278.17		mm per one wavelength
Centroid from Bottom of deck	y	19.86		mm
Effective Depth	d_p	90.14		mm
Inertia @ Centroid x-x axis				
1:Top flange	I_t	45787.78		mm^4
2:Two webs	I_w	41406.08		mm^4
3: Bottom flange	I_b	19770.20		mm^4
Inertia @ CG /wavelength	I_{cg}	106964.05		mm^4 per one wavelength
Inertia per meter	I	475395.78		mm^4 per meter
Width at Centroid	b_o	127.88		mm

FLEXURAL RESISTANCE (Full Bond)				
Details	**Notations**	**Value**	**Unit**	**Remarks**
Width of Deck	b	1000	mm	
Actual Neutral Axis	x_u	19.99	mm	
Balanced Neutral Axis	x_{max}	48.23	mm	x_{max}/d_p = 0.517 (For f_y=280 MPa)* Developed
Compressive Force	C	283.23	kN / m	
Tensile Force	T	283.23	kN/ m	
Lever arm	z	80.15	mm	
Flexural Resistance	M_{tf}	22.70	kN.m	Per m

FLEXURAL MOMENT (From Load)				
Details	**Notations**	**Value**	**Unit**	**Remarks**
Dead load	D.L.	2	kN/m^2	
Imposed Load	L.L.	7	kN/m^2	
Total load	T.L.	9	kN/m^2	
Effective span	L_{eff}	1.45	m	Single Span
Factored Flexural Moment	M	3.55	kN / m	
Unity Factor for flexure	**M/M_{tf}**	**0.156**		**Safe (If <1)**

A.1.2 Flexural Resistance: (BS-5950-Part-IV-1994)



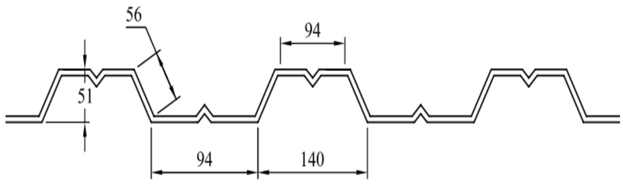
Type: Trapezoidal or Reentrant Composite Deck				
INPUT PARAMETERS				
Details	**Notations**	**Value**	**Unit**	**Specification as per BS-5950-Part-IV-1994**
Depth of profile sheet	d_p	51	mm	>= 50 mm
Width of top flange	b_t	94	mm	
Width of bottom flange	b_c	94	mm	
Gross Thickness of sheet	t_g	1	mm	>=0.75 mm 0.9 mm to 1.2 mm (Normally)
One wave length	b_s	230	mm	
Ratio b_t/b_s	b_t/b_s	0.40		b_t /b_s <=0.4
Design thickness	t	0.86	mm	(t_g-0.04)
Angle of web	θ	65.60^0		$\theta=55^0$ to 90^0
Depth of composite Deck	D	110	mm	Minimum 90 mm
Length of web	D_w	56	mm	
MATERIAL PROPERTIES				
Details	**Notations**	**Value**	**Unit**	**Specification as per BS-5950-Part-IV-1994**
Yield strength of steel	f_y	230	MPa	$f_y \geq 220$ MPa
Concrete cube strength	f_{cu}	25	MPa	

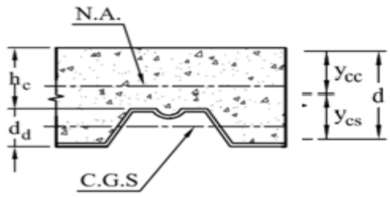
CENTROID AND MOMENT OF INERTIA				
Elements	**Notations**	**Length**	**Distance of C.G. from top**	**Unit**
1:Top flange	b_t	94	0.5	mm
2:Two webs	D_w	56	28	mm
3:Bottom flange	b_c	94	51	mm
Length per meter	L_p	1296.99		mm per m
Area per unit wave length	A	298.31		mm^2 per one wavelength
Area per meter	A_p	1296.99		mm^2
Length per wave length	L	298.31		mm per one wavelength
Centroid from Bottom of deck	y	25.5		mm
Effective Depth	d_p	84.5		mm
Inertia @ Centroid x-x axis				
1:Top flange	I_t	61123.50		mm^4
2:Two webs	I_w	61123.50		mm^4
3: Bottom flange	I_b	23909.40		mm^4
Inertia @ CG /wavelength	I_{cg}	146156.40		mm^4 per one wavelength
Inertia per meter	I	635462.62		mm^4 per meter
Width at Centroid	b_o	136		mm

FLEXURAL RESISTANCE (Full Bond)				
Details	**Notations**	**Value**	**Unit**	**Remarks**
Width of Deck	b	1000	mm	
Actual Neutral Axis	x_u	24.66	mm	
Balanced Neutral Axis	x_{max}	45.20	mm	x_{max}/d_p = 0.535 (For f_y=230 MPa)* Developed
Compressive Force	C	277.42	kN / m	
Tensile Force	T	277.42	kN/ m	
Lever arm	z	72.16	mm	
Flexural Resistance	M_{tf}	20.02	kN.m	Per m

A.1.3 Flexural Resistance: (ANSI-SDI-2011)

Yield Moment Theory





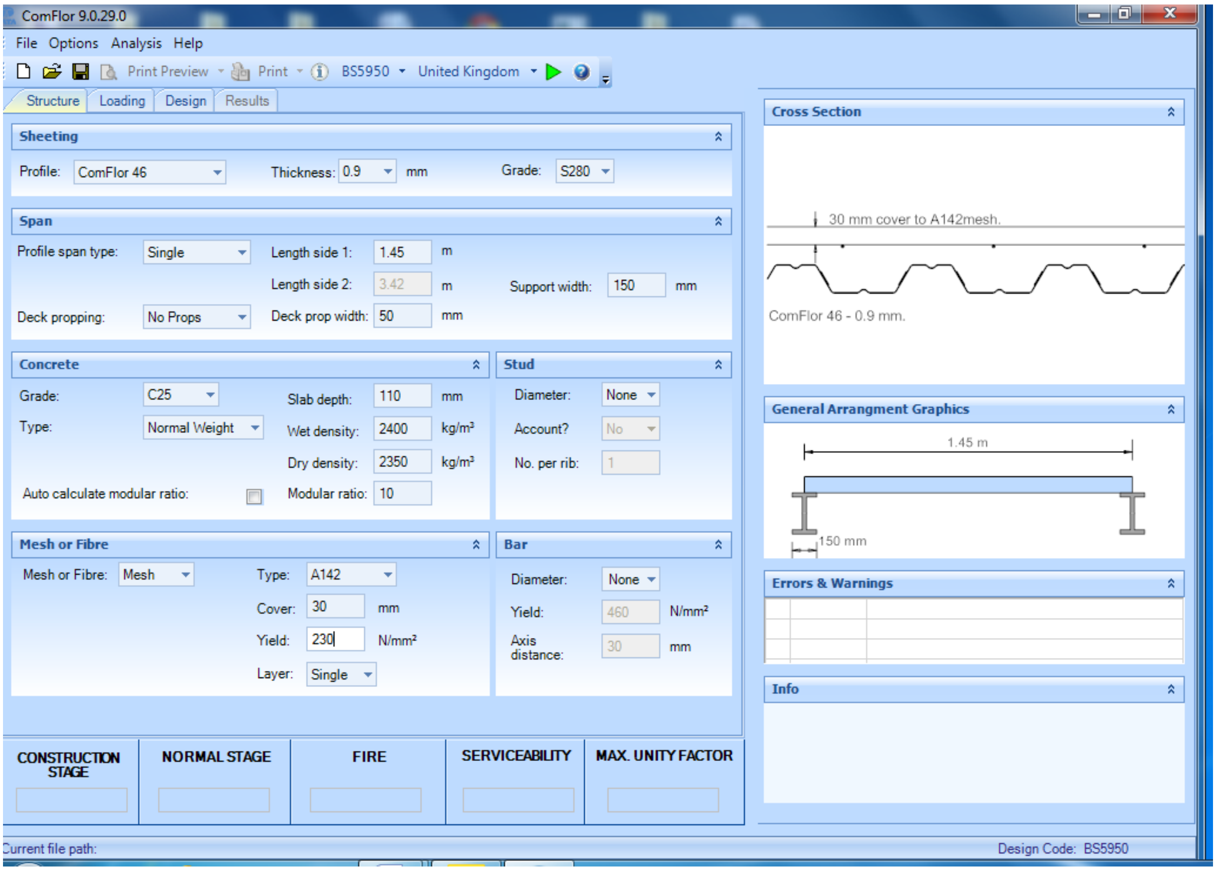
Type: Trapezoidal or Reentrant Composite Deck				
INPUT PARAMETERS				
Details	**Notations**	**Value**	**Unit**	**Specification as per ANSI-SDI-2011**
Depth of profile sheet	d_p	51	mm	>= 50 mm
Width of top flange	b_t	94	mm	
Width of bottom flange	b_c	94	mm	
Gross Thickness of sheet	t_g	1	mm	22 to 16 Gauge (0.75 to 1.52 mm)
One wave length	b_s	230	mm	
Ratio b_t/b_s	b_t/b_s	--		-
Design thickness	t	0.86	mm	(t_g-0.04)
Angle of web	θ	65.60^0		$\theta=55^0$ to 90^0
Depth of composite Deck	D	110	mm	Minimum 75 mm
Length of web	D_w	56	mm	
MATERIAL PROPERTIES				
Details	**Notations**	**Value**	**Unit**	**Specification as per ANSI-SDI-2011**
Yield strength of steel	f_y	230	MPa	$f_y \geq$ 33ksi (230 MPa)
Concrete cube strength	f_{cu}	25	MPa	3000-6000 psi (21 - 42MPa)

FLEXURAL RESISTANCE (Full Bond)				
Details	**Notations**	**Value**	**Unit**	**Remarks**
Width of Deck	b	1000	mm	
Resistance factor	ϕ_s	0.85	mm	
Modulus of elasticity of concrete	$Ec = 0.043 * (\rho c)^{1.5}\{\sqrt{fcu}\}$	25278.73	MPa	
Modulus of elasticity of steel	Es	203000	MPa	(As per ANSI)
Modular Ratio	$n = \frac{E_s}{E_c}$	8.03		
Factor	$\rho = \frac{A_s}{b\,d}$	0.015		
Depth of N.A.	$ycc = d\left\{\sqrt{2\rho n + (\rho n)^2} - \rho n\right\} \leq h_c$	32.81	mm	
Height of Concrete	$h_c = D - d_p$	59	mm	
Depth	$y_{cs} = d_p - y_{cc}$	51.68	mm	
Cracked M.I.	$I_{cr} = \frac{b}{3n} y_{cc}{}^3 + A_s y_{cs}{}^2 + I_{sf}$	5566921	mm^4	
Flexural Resistance	$M_y = \frac{F_y I_{cr}}{(h - y_{cc})}$	14.09	kN.m	Per m

A.1.4 'ComFlor' Software* (BS EN 1994 1-1)

(a) ComFlor Software: Datasheet 1: INPUT DATA

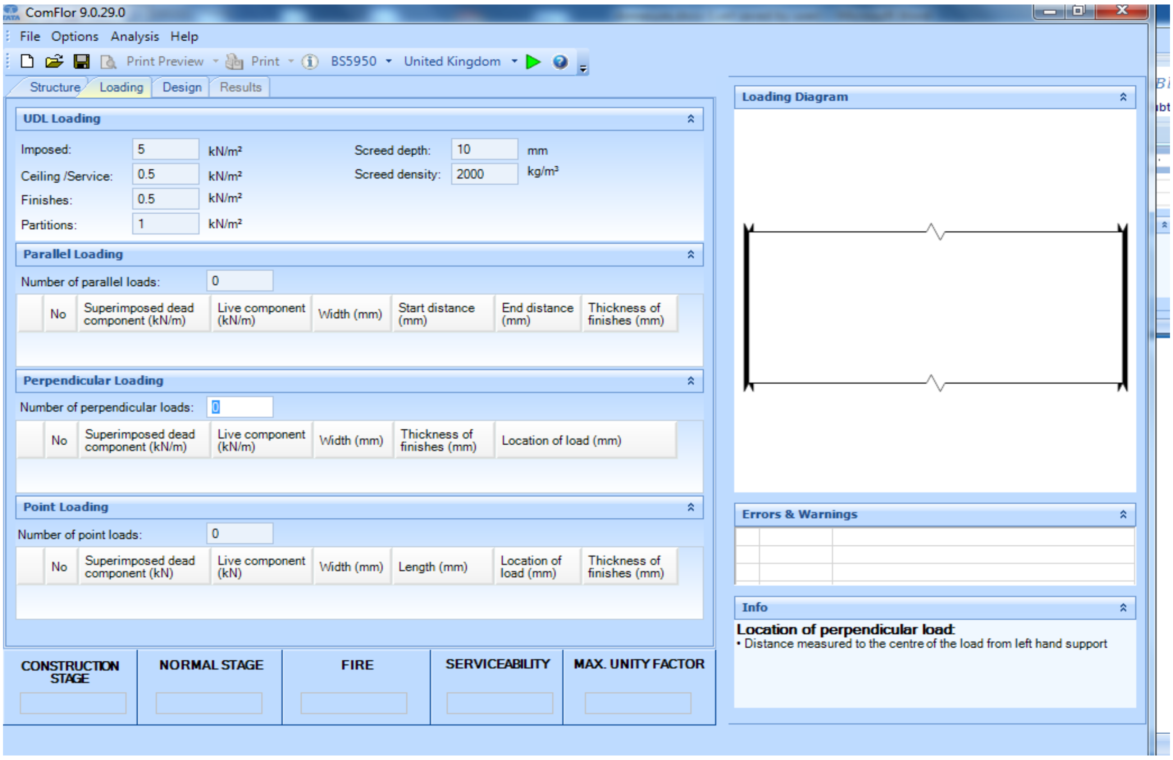
- Type of profiled steel sheet: ComFlor 46
- c/c distance between support : 1.45 m single span
- Total depth of slab:110 mm
- Concrete class:C25
- Steel Grade: 280 MPa
- Thickness:0.9 mm
- Mesh Type:A142 (Anticrack mesh)



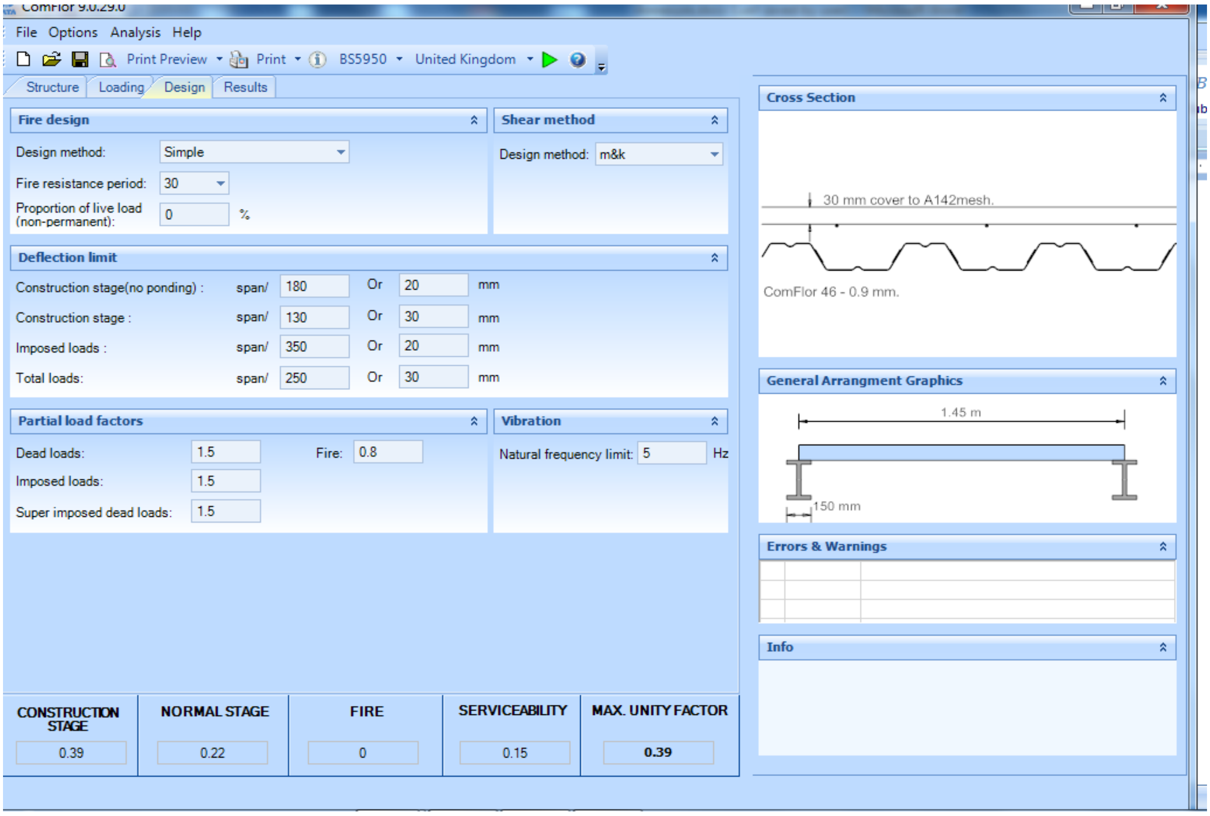
*Source: Tata ComFlor software

(b) ComFlor Software : Datasheet 2 :Load Data

- Dead Load: Program Calculated
- Imposed Load: 5 kN/m^2
- Other (Finish) Load:2 kN/m^2

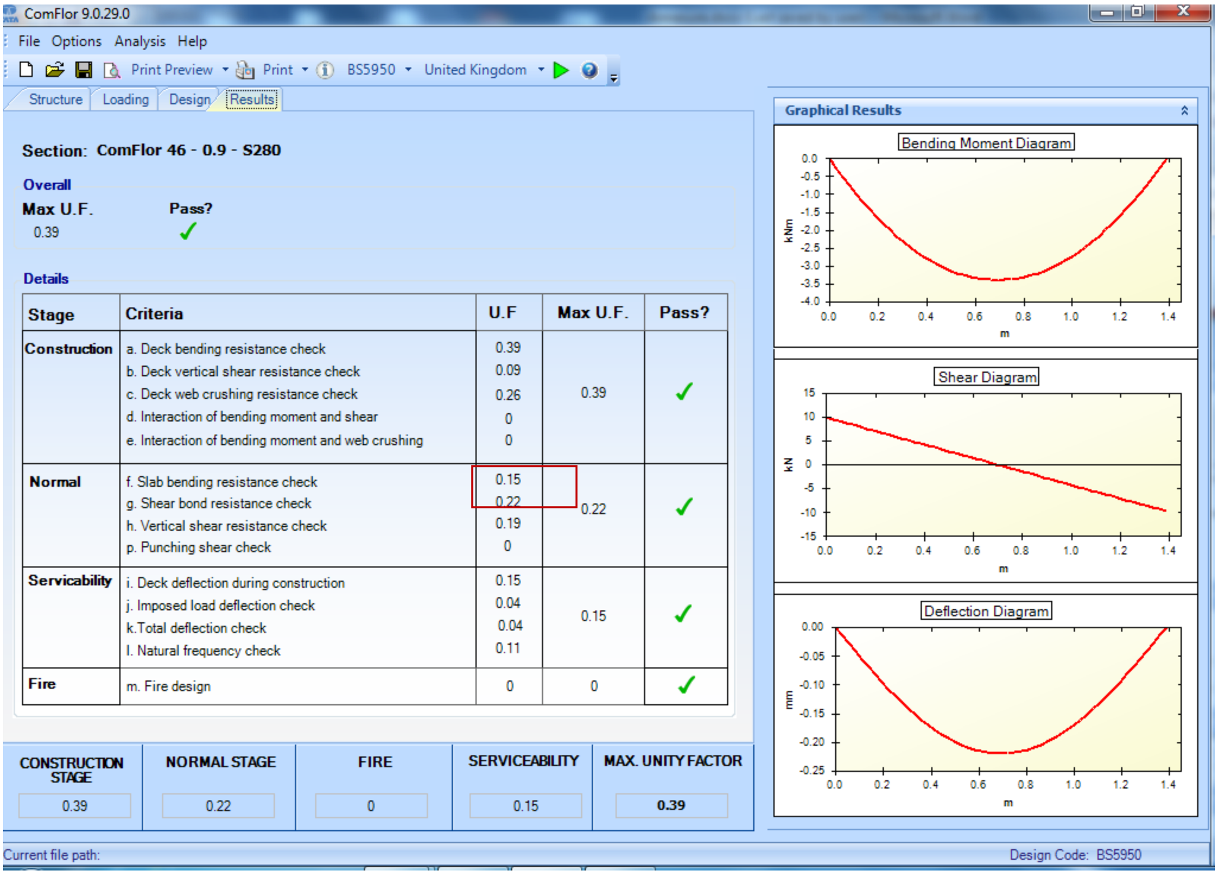


(c) ComFlor Software : Datasheet 3: Analysis



(d) ComFlor Software: Datasheet 4: Results

Design checks



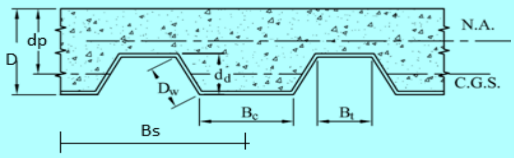
Stage	Criteria	U.F	Max U.F.	Pass?
Construction	a. Deck bending resistance check	0.39	0.39	✓
	b. Deck vertical shear resistance check	0.09		
	c. Deck web crushing resistance check	0.26		
	d. Interaction of bending moment and shear	0		
	e. Interaction of bending moment and web crushing	0		
Normal	f. Slab bending resistance check	0.15	0.22	✓
	g. Shear bond resistance check	0.22		
	h. Vertical shear resistance check	0.19		
	p. Punching shear check	0		
Servicability	i. Deck deflection during construction	0.15	0.15	✓
	j. Imposed load deflection check	0.04		
	k.Total deflection check	0.04		
	l. Natural frequency check	0.11		
Fire	m. Fire design	0	0	✓

Comparison of Flexural resistance by 'ComFlor' Software and Developed Program:

- 'ComFlor' Unity Factor (Normal: slab bending resistance check) = 0.15
- Developed Program Unity Factor = 0.156

Program on International Standards

INPUT PARAMETERS



Details	Notations	Value	Unit
Enter Depth of profile sheet	dp	51	mm
Profile Deck	PD	PD 51	
Enter Width of top flange	Bt	94	mm
Enter Width of bottom flange	Bc	94	mm
Enter Thickness of profile sheet	t	1	mm
Enter One wave length	Bs	230	mm
Check Bt/Bs	Bt/Bs	0.409	
Angle of web	θ	67.44	
Overall Depth	D	110	mm
Length of web	Dw	55.15	mm

CALCULATIONS:CENTROID AND MOMENT OF INERTIA

Elements	Length(mm)	Distance of C.G. from top (mm)	
1:Top flange	94	0.5	
2:Two webs	110.31	25.5	
3:Bottom flange	94	50.5	
Length per meter	L/m	1296.99	mm per m
Area per unit wave length	A	298.31	mm^2 per one wavelength
Area per meter	A/m	1296.99	mm^2
Length per wave length	L	298.31	mm per one wavelength
Effective Depth	d_p	84.50	mm
Centroid from Bottom of deck	y	25.50	mm
Inertia @ Centroid x-x axis	I1:Top flange	61123.50	mm^4
	I2:Two webs	61123.50	mm^4
	I3:Bottom flange	23909.40	mm^4
Inertia @ CG /wavelength	I	146156.40	mm^4 per one wavelength
Inertia per meter	I	635462.62	mm^4
Top empty width	bo	115.00	mm

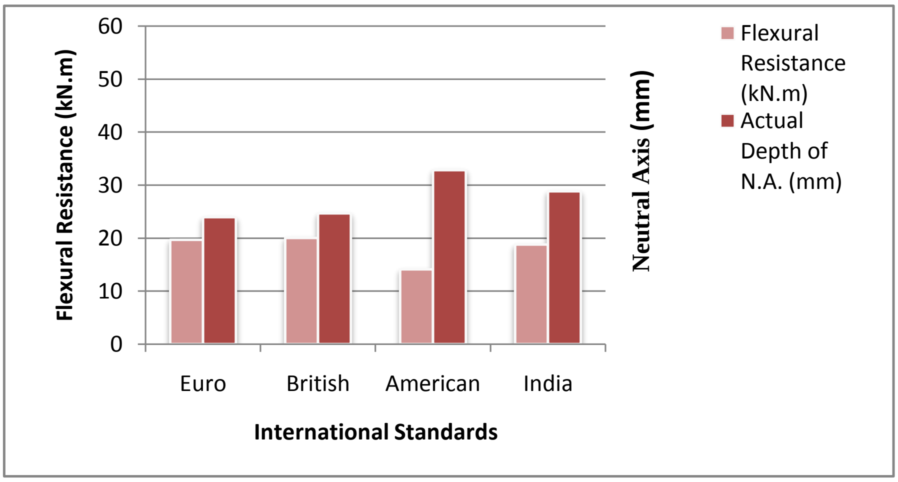
INPUT PARAMETERS: MATERIAL PROPERTIES			
Yield strength of steel	fy	230	Mpa
Concrete Cube strength	fcu/fck	25	Mpa
FLEXURAL RESISTANCE- EN (FULL BOND)			
Neutral Axis	x_u	23.95	mm
Flexural Resistance:Euro Code	M.R.(EN)	19.67	kN.m per meter
FLEXURAL RESISTANCE- BS (FULL BOND)			
Neutral Axis	x_u	24.66	mm
Flexural Resistance: British Code	M.R.(BS)	20.02	kN.m per meter
FLEXURAL RESISTANCE- IS (FULL BOND)			
Neutral Axis	x_u	28.82	mm
Flexural Resistance:Indian code	M.R.(IS)	18.78	kN.m per meter
FLEXURAL RESISTANCE- ANSI (FULL BOND)			
Neutral Axis	y_{cc}	32.81	mm
Cracked Moment of Inertia	I_{cr}	5566921.14	mm^4
Flexural resistance:American code	M.R.(ANSI)	14.10	kN.m per meter

RESULTS

CONFIGURATION OF PROFILE DECK **PD 51**

TOTAL LENGTH OF DECK PER m **1296.99** **mm**

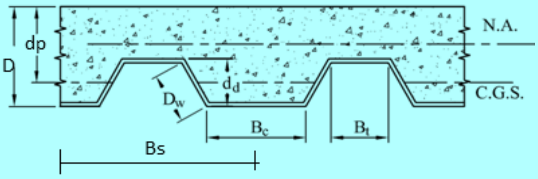
International Standards	Actual Depth of N.A. (mm)	Flexural Resistance (kN.m)	% Difference as compared to IS
Euro	23.95	19.67	4.74
British	24.66	20.02	6.62
American	32.81	14.10	-24.92
India	28.82	18.78	--



Program on Parametric Variations

Type: Trepezoidal or Reentrant Deck

INPUT PARAMETERS



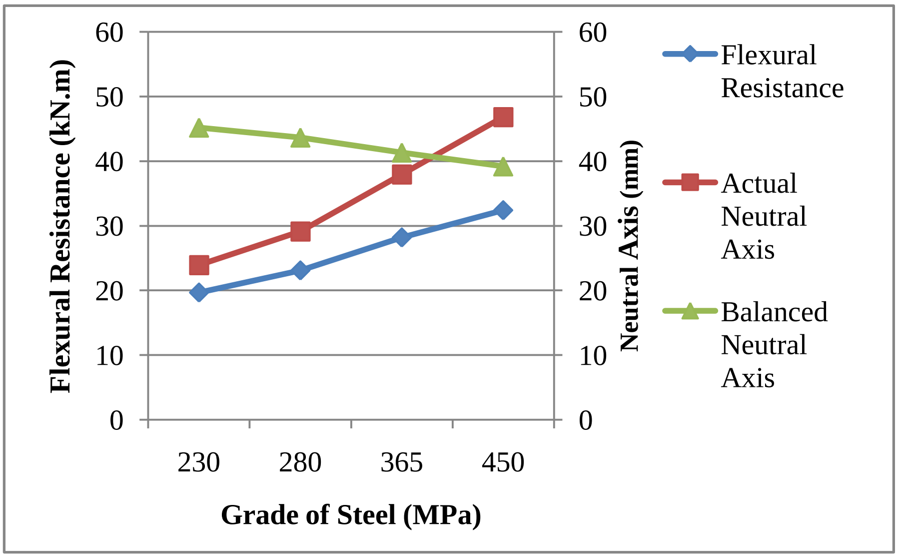
Details	**Notation**	**Value**	**Unit**	**Codal Specification as per EN-1994-2004**
Enter Depth of profile sheet	d_p	51	mm	40 mm to 80 mm
Profile Deck	PD	PD51		
Enter Width of top flange	B_t	94	mm	
Enter Width of bottom flange	Bc	94	mm	
Enter One wave length	Bs	230	mm	Bs>Bt+Bc
Check Bt/Bs	Bt/Bs	0.409	-	Bt/Bs=0.4
Angle of web	θ	67.44		$\theta = 55^0$ to 90^0
Length of web	D_w	55.15	mm	
Total Length of Deck	L	1296.99	mm	

PARAMETRIC VARIATIONS

Type of variable	Grade of Steel (MPa)			
Enter four values of Grade of Steel (MPa)	**1**	230		
	2	280		
	3	365		
	4	450		
Enter Overall Depth (mm)		110		
Enter Grade of Steel* (Mpa)		230		
Enter Grade of Concrete (Cube)		25		
Enter Thickness of profile sheet (mm)		1		

*Select Grade of Steel from:230,250,280,310350,365,450 Mpa

RESULTS				
CONFIGURATION OF PROFILE DECK			**PD51**	
TOTAL LENGTH OF DECK PER m			**1296.99**	**mm**
VARIATIONS			**Grade of Steel (MPa)**	
% DIFFERENCE IN FLEXURAL RESISTANCE			**64.78**	**kN.m**
Grade of Steel (MPa)	Actual Depth of N.A. (mm)	Balanced N.A. (mm)	Flexural Resistance (kN.m)	% Difference in Flexural Resistance
230	23.93	45.18	19.67	
280	29.13	43.67	23.09	
365	37.97	41.31	28.19	
450	46.82	39.20	32.41	64.78



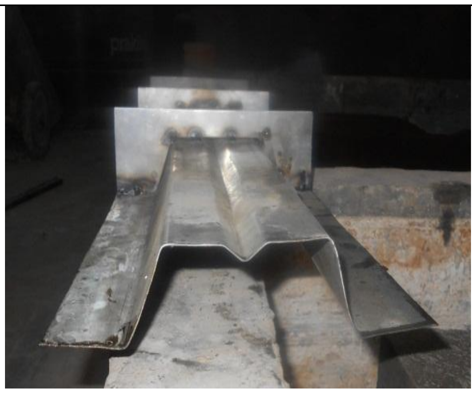
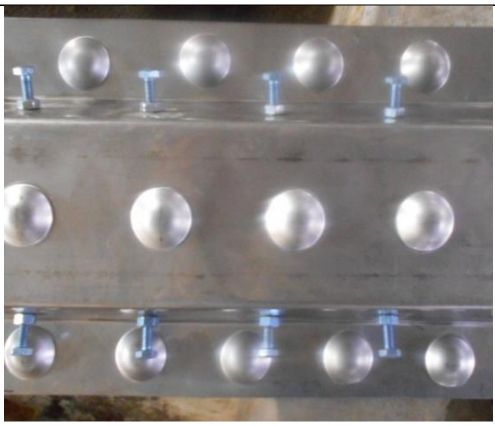
APPENDIX - II

Interface Topologies and Failure of Composite Deck

A.2.1 Interface Topologies for Three Wavelengths Specimen

	
Deck with embossment:CS-1A-B	Deck with hemisphere: CS-2A-B
	
Deck with chemical bond:CS-3A-B	Deck with cross stiffener:CS-4A-B

A.2.2 Interface Topologies for One Wavelength Specimen

	
Bolt head facing interface:CB-1-2	Bolt shank facing interface:CB-3-4
	
Arc Bend on web:CB-5-6	Straight Stiffeners:CB-7-8
	
Pressed in – in embossments :CB-9-10	Pressed in – out embossments:CB-11-12

A.2.3 Experimental Set Up and Failure: Three wavelength Specimen

	
Test load set up	Dial Gauges on the side of the deck:CS2
	
Separation and slip :CS1	Diagonal Crack and Slip :CS2
	
Damage: CS3	Flexure crack : CS4

A.2.4 Experimental Set Up and Failure: One wavelength Specimen

Flexure cracks :Negligible vertical separation -CB1

Flexure crack at centre

Flexure cracks :Negligible vertical separation -CB2

Negligible slip

Flexure Bond Failure :Negligible vertical separation -CB3

Negligible slip

Flexure Bond Failure: Negligible vertical separation-CB4

Crack at side

Flexure Bond Failure :Negligible vertical separation- CB5

Minor Slip

A.2.4 Failure: One wavelength Composite Specimens....Continued

Flexure-Bond Failure: Negligible vertical separation -CB6

Crack at side

Sudden Failure from Stiffner-CB-7

Major crack

Sudden Failure from Stiffner, Major vertical separation-CB-8

Major Slip

Bond Failure: Major vertical separation- CB-9

Major slip

Bond Failure : Major vertical separation-CB-10

Bond failure -Loss of composite action

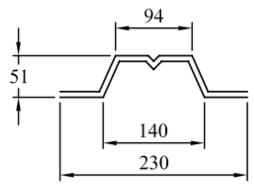
A.2.4 Failure: One wavelength Composite Specimens....Continued

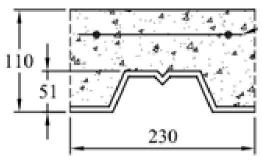
	
Bond failure: Loss of composite action-CB-11	Vertical separation-Loss of composite action
	
Bond failure : Loss of composite action-CB-12	

APPENDIX - III

Analytical Studies: Approaches using Composite Action

A.3.1 Full Composite Action: Euro standard EN-1994-2004





Type: Trapezoidal or Reentrant Composite Deck:				
INPUT PARAMETERS: AS PER CB-1				
Details	**Notations**	**Value**	**Unit**	**Specification as per EN-1994-2004**
Depth of profile sheet	d_p	51	mm	40 mm to 80 mm
Width of top flange	b_t	94	mm	
Width of bottom flange	b_c	94	mm	
Gross Thickness of sheet	t_g	1	mm	0.7 mm to 1.2 mm
One wave length	b_s	230	mm	
Ratio b_t/b_s	b_t/b_s	0.408		$b_t /b_s <= 0.4$
Angle of web	θ	55.14^0		$\theta = 55^0$ to 90^0
Depth of composite Deck	D	110	mm	Minimum 80 mm
Length of web	D_w	55.15	mm	

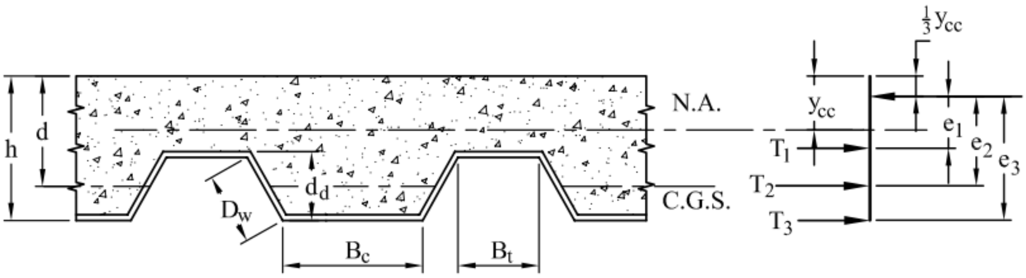
MATERIAL PROPERTIES				
Details	**Notation**	**Value**	**Unit**	**Specification as per EN-1994-2004**
Yield strength of steel	f_y	230	MPa	$f_y \geq 230$ MPa
Concrete cylinder strength	f_{cu}	32.88	MPa	Cube strength
CENTROID AND MOMENT OF INERTIA				
Elements	**Notations**	**Length**	**Distance of C.G. from top**	**Unit**
1:Top flange	b_t	94	0.45	mm
2:Two webs	D_w	110.30	23	mm
3:Bottom flange	b_b	94	45.55	mm
Length per meter	L_p	298.30		mm per m
Area per unit wave length	A	298.30		mm^2 per one wavelength
Area per meter	A_p	1112.70		mm^2
Length per wave length	L	278.17		mm per one wavelength
Centroid from Bottom of deck	y	25.5		mm
Effective Depth	d_p	84.5		mm
Inertia @ Centroid x-x axis				
1:Top flange	I_t	61123.5		mm^4
2:Two webs	I_w	61123.5		mm^4
3: Bottom flange	I_b	23909.40		mm^4
Inertia @ CG /wavelength	I_{cg}	146156.40		mm^4 per one wavelength
Inertia per meter	I	635462.61		mm^4 per meter

FLEXURAL RESISTANCE				
Details	**Notations**	**Value**	**Unit**	**Remarks**
Width of Deck specimen	b	230	mm	
Actual Neutral Axis	x_u	13.33	mm	
Balanced Neutral Axis	x_{max}	45.20	mm	$x_{max}/d_p = 0.535$ (For f_y=230 MPa) Developed
Tensile Force	T	68.54	kN	Per section
Lever arm	z	77.83	mm	
Flexural Resistance	M_{tf}	**5.34**	**kN.m**	**Per section**

A.3.2 No Bond

FLEXURAL RESISTANCE: NO BOND				
INPUT PARAMTERS:AS PER CB-1				
Details	**Notation**	**Value**	**Unit**	**Remarks**
Intertia of deck	I_{Deck}	146156.4	mm^4	Per section
Centroidal axis of deck	Y_{Deck}	25.5	mm	Per section
Section Modulus	Z_{Deck}=I/y	5731.62	mm^3	Per section
M_{Deck} = M_{tn}	**F_y* Z_{deck}**	**1.32**	**kN. m**	**Per section**

A.3.3 First Yield Approach



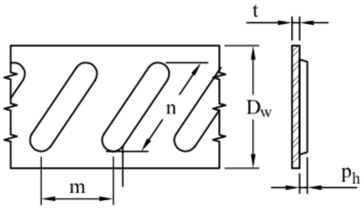
Input Parameters: As per A.3.1

Material Properties: As per A.3.1

Centroid and Inertia calculation: As per A.3.1

INPUT PARAMETERS: AS PER SPECIMEN CB-1-2				
Details	**Notations**	**Value**	**Unit**	**Remarks**
Reinforcement Ratio	$A/(b^* d_p)$	0.0153	--	
Modular Ratio	$n= E_s/E_c$	10	--	**Assumed**
Neutral axis Ycc	$y_{cc} = d(\sqrt{2\rho n} + (\rho n^2 - \rho n)$	35.61	mm	
Neutral axis of bottom flange e_3	$e_3 = h - \frac{y_{cc}}{3}$	98.12	mm	
Neutral axis of Top flange e_1	$e_1 = e_3 - d_d$	47.12	mm	
Neutral axis of web e_2	$e_2 = e_3 - \frac{d_d}{2}$	72.62	mm	
Tensile force of top flange T_1	$T_1 = f_{yc}(B_t * t)\frac{h - y_{cc} - d_d}{h - y_{cc}}$	6797.58	N	
Tensile force of bottom flange T_3	$T_3 = f_{yc}(B_c * t)$	21620	N	
Tensile force of web T_2	$T_2 = f_{yc}(2 * D_w * t)\frac{h - y_{cc} - \frac{d_d}{2}}{h - y_{cc}}$	16673.96	N	
Flexural Resistance	$\mathbf{M_{ty} = (T_1 e_1 + T_2 e_2 + T_3 e_3)}$	**3.65**	**kN. m**	**Per section**

A.3.4 Lutrell Lug Approach



INPUT PARAMETERS: AS PER SPECIMEN CS-1			
Details	**Notation**	**Value in mm**	**Remarks**
Bond Pattern centre line length	n	20	
Bond Pattern spacing	m	40	
Bond Pattern height	P_h	2.5	
Overall Depth of Deck	D	110	
Depth of steel deck	D_d	51	
Thickness of deck	t	0.8	
Slab to flute width ratio	B/B_c	3.04	

FLEXURAL CAPACITY-LUTRELL APPROACH			
Details	**Notation**	**Value in Inch**	**Remarks**
Single line bond	$P_s = 12(n/m)$	6	
Factor	$P_s{*}P_h$	0.59	If $P_s{*}P_h < 0.6$
Bond Factor k_1	$k1 = \frac{1}{\sqrt{D_d}} + \frac{t}{P_h\, D_d}$	0.865	
Bond Factor k_2	$k2 = \frac{100\,(t)^{1.5}}{D\sqrt{P_h}}$	0.411	
Bond Factor k_3	$k3 = 0.87 + 10^{-3}\left(\frac{B}{Bc}\right)(\,69 - 2.2\,\frac{B}{Bc})$	1.059	
Bond Factor k	$K = \frac{k_3}{k_1 + k_2}$	0.830	
Flexural capacity	$M_{tl} = K\, M_{tf}$	0.83* 14.22	M_{tf} =14.22 kN. m (As per A.3.1)
Lutrell Flexural Resistance	M_{tl}	11.80	kN. m/Section

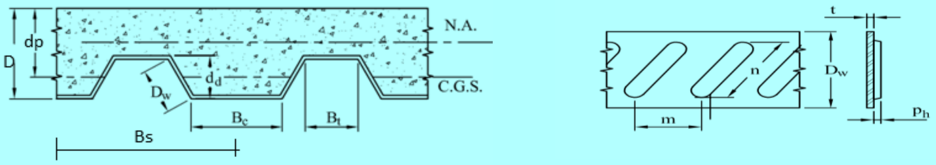
A.3.5 Modified Composite Beam Analogy

Input Parameters, Material Properties, Centroid and Inertia calculation: As per A.3.1

Capacity of Connector				
Details	**Notation**	**Value**	**Unit**	**Remarks**
Width of Deck	b	230	mm	
Tensile Force	T	62309.09	N	
Shank Dia. (Average)	d	9.300	mm	
Area of Bolt	A_{sc}	67.895	mm^2	
Tensile strength of bolt	F_u	400	MPa	
Capacity of Bolt = P_{u1}	$P_{u1} = A_{sc} * F_u$	24.689	kN	
Capacity of Bolt = P_{u2}	$P_{u2} = 0.5A_{sc}\sqrt{f'_c E_c}$	27.529	kN	$E_c = w^{1.5} * 0.0428\sqrt{f'_c}$
Capacity of connector (Less of P_{u1} and P_{u2})	Q_p	24.689	kN	
Longitudinal Comp. Force	$F_{p1} =< A_p f_y$	68.540	kN	
Longitudinal Comp. Force	$F_{p2} =< 0.8 * f_{cu}$ *area of conc. within effective c/s	80.635	kN	
Higher Longitudinal Comp. Force	$F_p = F_{p2}$	80.635	kN	BS-5950,Part 3.1-Table -5 for higher dai)

FLEXURAL RESISTANCE: MODIFIED COMPOSITE BEAM ANALOGY				
Details	**Notation**	**Value**	**Unit**	**Remarks**
No. of connector for full interaction	$N = F_p/Q_p$	3.266		
No. of connectors provided	N_p	3.000		(In a plane)
Ratio of connectors	N_p/N	0.918		If $N_p > N$ Take $N_p/N = 1$
Flexural Capacity Full bond	$M_{full} = M_{tf}$	5.34	kN. m	(A-3.1)
Flexural Capacity No bond	$M_{no} = M_{tn}$	1.32	kN. m	(A-3.2)
Width of deck factor k_w	$kw = 0.87 + 10^{-3}\left(\frac{B}{Bc}\right)\left(69 - 2.2\frac{B}{Bc}\right)$	0.937		
$M_{comp} = M_{tm}$	$M_{tm} = k_w[M_{tn} + \frac{N_P}{N}(M_{tf} - M_{tn})]$	4.69	kN. m	
Flexural Resistance - Modified Comp. beam Analogy	$\mathbf{M_{tm}}$	**4.69**	**kN. m**	

Program on Analytical Approaches



Details	Notations	Value	Unit
Enter Depth of profile sheet	dp	51	mm
Profile Deck	PD	PD51	
Enter Width of top flange	Bt	94	mm
Enter Width of bottom flange	Bc	94	mm
Enter Thickness of profile sheet	t	1	mm
Enter One wave length	Bs	230	mm
Check Bt/Bs	Bt/Bs	0.41	
Angle of web	θ	67.44	
Length of web	Dw	110	
Total Length of Deck	L	55.15	mm
Exp. Width of Deck	B/Section	230	mm

CALCULATIONS:CENTROID AND MOMENT OF INERTIA

Elements	Length(mm)	Distance of C.G. from top (mm)	
1:Top flange	94	0.5	
2:Two webs	110.31	25.5	
3:Bottom flange	94	50.5	
Length per meter	L/m	1296.99	mm per m
Area per unit wave length	A	298.31	mm^2 per one wavelength
Area per Section	A/Section	298.31	mm^2
Length per wave length	L	298.31	mm per one wavelength
Effective Depth	d_p	84.50	mm
Centroid from Bottom of deck	y	25.50	mm
Inertia @ Centroid x-x axis	I1:Top flange	61123.50	mm^4
	I2:Two webs	61123.50	mm^4
	I3:Bottom flange	23909.40	mm^4
Inertia @ CG /wavelength	I	146156.40	mm^4 per one wavelength
Inertia per Section	I/Section	146156.40	mm^4
Top empty width	bo	115.00	mm

INPUT PARAMETERS: MATERIAL PROPERTIES			
Yield strength of steel	fy	230	Mpa
Concrete Cube strength	fck	32.88	Mpa
FLEXURAL RESISTANCE : FULL BOND			
Neutral Axis	x_u	13.34	mm
Balanced Neutral Axis	x_{max}	45.21	mm
Lever arm	z	77.83	mm
Flexural resistance:Full Bond	Mtf	5.34	kN.m/ Section
FLEXURAL RESISTANCE:NO BOND			
Flexural resistance:No Bond	Mtn	1.32	kN.m/ Section
FLEXURAL RESISTANCE-:FIRST YIELD APPROACH			
Resistance factor	φ	0.85	
Modular Ratio	n=Es/Ec	10.00	
Factor	ρ=As/bd	0.015	
Neutral axis	ycc	35.61	mm
Neutral axis of bottom flange e3	$e_3 = h - \frac{y_{cc}}{3}$	98.13	mm (h=D)
Neutral axis of Top flange e1	$e_1 = e_3 - d_d$	47.13	mm (dd=dp)
Neutral axis of web e2	$e_2 = e_3 - \frac{d_d}{2}$	72.63	mm
Tensile force of top flange T1	$T_1 = f_{yc}(B_t * t)\frac{h - y_{cc} - d_d}{h - y_{cc}}$	6797.58	kN
Tensile force of bottom flange T3	$T_3 = f_{yc}(B_b * t)$	21620.00	kN (Bb=Bc)
Tensile force of web T2	$T_2 = f_{yc}(2 * D_w * t)\frac{h - y_{cc} - \frac{d_d}{2}}{h - y_{cc}}$	16673.96	kN
Flexural Resistance	$\mathbf{M_{TY} = (T_1\, e_1 + T_2\, e_2 + T_3\, e_3)}$	3.65	kN.m/Wavelength
Flexural Resistance:First Yield Theory	Mty	3.65	kN.m/Section

FLEXURAL RESISTANCE:LUTRELL APPROACH			
Details	**Notations**	**Value in mm**	**Value in Inch**
Enter interlock centreline length	n	9.333	0.367
Enter interlock spacing	m	105	4.134
Enter interlock height	Ph	14	0.551
Enter overall Depth of Deck	D	110	4.331
Enter depth of steel deck	Dd	51	2.008
Enter thickness of deck	t	1	0.039
Slab to flute width ratio	B/Bc	1	1.000
Ps = 12(n/m)	Ps		1.067
	Ps*Ph		0.588
If PsPh < 0.6	k1		0.741
	k2		0.243
	k3		0.937
Bond Factor -I	K=k3/(k1+k2)		0.952
If PsPh > 0.6	k1		4.861
	k2		-0.007
	k3		0.937
Bond Factor -II	K=k3/(k1+k2)		0.193
Governing Bond Factor	K		0.952
Flexural Resistance:Lutrell Approach	Mtl	5.08	kN.m/Section

FLEXURAL RESISTANCE:MODIFIED BEAM ANALOGY			
Details	**Notations**	**Value**	**Unit**
Enter Modulus of Elasticity Ec	Ec	20000	(Assume: 20000 MPa)
EnterConnector/Shank Average Diameter	n	9.300	mm
Area of Connector	Asc	67.895	mm^2
Enter Tensile strength connector	Fu	400.000	Mpa
Minimum Capacity of connector	Qp	24.689	kN
Higher Logitudinal Comp.Force	Fp	80.719	kN
Enter No. of connectros provided	N	3.000	
Np/N	Np/N	0.918	
Deck Width Factor	kw	0.937	
Flexural Resistance:Modified Beam Analogy Approach	Mtm	4.692	kN.m/Section