# A STUDY ON THE SUITABILITY OF LOCAL STEEL PROFILES FOR THE PRODUCTION OF STEEL DECK REINFORCED COMPOSITE SLABS FOR BUILDING

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*By*

***Rufael Redie***

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**ADDIS ABABA UNIVERSITY**
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**DEPARTMENT OF CIVIL ENGINEERING**

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**RUFAEL REDIE**

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Approved by Board of Examiners

| | | |
|---|---|---|
| Dr. Asnake Adamu | | |
| Advisor | Signature | Date |
| Prof. Negussie Tebedge | | |
| External Examiner | Signature | Date |
| Dr. Shifferaw Taye | | |
| Internal Examiner | Signature | Date |
| Dr. Ing. Adil Zekaria | | |
| Chairman | Signature | Date |

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# ABSTRACT

Floor and roof slabs constructed using composite panels of steel deck and concrete in which the steel deck panels serve both, as formwork and reinforcement are the practice of developed nations. The steel deck manufacturers developed their own cross sectional profiles and provide detail manuals where the thickness of the sheets ranges between 0.8mm to 1.5mm. This new technology has not been yet practiced in our construction industry. Thus, this Thesis Paper aims at assessing the local profile steel sheet products and develop analysis and design techniques for steel deck concrete composite slab for building, which fit our local conditions.

The organization of the study can be viewed to consist of four main components, the first being assessment of the physical and mechanical properties and production process of steel sheet profiles produced by local manufacturers as well as foreign practices. Analysis and design requirement based on local building code requirement, foreign code of practice such as British Standard and international published books on the subject matter are dealt in the second part of the text for various stages of loading. Laboratory test for the assessment of tensile strength of local sheet and chemical as well as mechanical shear interlock between profile sheet and concrete conducted in the capacity of the local laboratory facility can be very well grouped as the third part of the manuscript.

Design example for composite slab together with cost comparison between composite system and conventional slab construction are elaborated in the last section of the text. In general, Serviceability, Strength and Economy aspects of such construction are the main concerns of the study.

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# 1 INTRODUCTION

## 1.1 OBJECTIVES

The conventional method practiced to date for construction of slabs for multi-story building in all over our country is by preparing temporary plat form with proper supporting members. The members should be strong enough to support any loads coming to it such as the freshly poured concrete, reinforcement bars and other moving loads during construction. Elements of the false work should be kept as it is till the concrete hardened to the required strength. Unlike the usual procedure of reinforced concrete slab construction, steel deck reinforced composite slab construction uses prefabricated steel deck panels used as formwork and tensile reinforcement.

A number of advantages attained through the use of composite slabs are summarized below [3].

- The steel deck, easily and quickly laid on the steel floor beams, serves as a working platform to support construction activity and to carry the freshly poured concrete. This eliminates the need for temporary false work and forms.
- The steel deck, with proper attention to design and details, can serve as the main tensile reinforcement for the slab.
- If parts or all of the deck panels are formed in to closed cells, the cells can serve ducts for electric and communication cables, or for heating and air conditioning.

Thus, this project aims at assessing the production and design techniques of composite steel deck panels and concrete, which fit our local condition.

The serviceability, strength and economy aspects of such construction will be the concern of the Thesis.

## 1.2 BACKGROUND

Floor and roof slabs constructed using composite panels of steel deck and concrete in which the steel deck panels serve both, as formwork and reinforcement are the practice of developed nations. Unlike other foreign Building Code of Practices such as British Standard (BS 5950 Part-4, 1994), which elaborates at depth in the area of steel profile concrete composite slab design, Ethiopian Building Code of Standard hardly, provided sufficient design techniques in the subject. As a matter of fact this New Technology has not been practiced in our construction industry. Possibility of material cost saving and fast construction time in comparison to the conventional slab construction attributed the development of the thesis.

## 1.3 METHODOLOGY

To achieve the objectives stated above, literature review, which is related to the topic, has been exhaustively undertaken. In particular, Ethiopian Building Code of practices, British Standards and International edited books served as important documents to develop the basic formulation pertinent to analysis and design of steel sheet & concrete composite building slab construction.

The study is supported with laboratory test verification. Test specimen preparation, laboratory test setup and testing operation were under taken as per the timetable attached in the sixth chapter. The laboratory tests are basically of two types. Tensile strength test for local profiled steel sheets, with three test samples collected from Kality Metal Industry, was conducted by the aid of universal testing machine with an accuracy level of 0.1KN. The second test, know as push out test, which was designed to determine shear resistance developed in between the profiled steel sheet and concrete was cumbersome and time taking. Eight specimens used for the push out test were manufactured with the collaboration of Kality Metal Industry.

All tests were conducted in the material-testing laboratory of Addis Ababa University found at the Faculty of Technology.

Finally, cost assessment has been done to compare the advantage of the composite system under consideration with the conventional reinforced concrete building slab.

## 1.4 SCOPE OF THE STUDY

The division of the chapters is designed to have smooth flow of idea starting from the study of the production process of profiled steel sheet to the advantage of the composite system over the conventional reinforced slab construction practice today in the local construction industry. Brief Explanation about depth of study covered in each chapter is discussed as below.

The second chapter deals with the production process, type in use by other countries as well as local products of profiled steel sheet forms. The physical and mechanical properties of profile sheets are also discussed as per the specification requirement of International Standardization organization (ISO).

Chapter three basically can be seen consisting of two sections, the first section discusses about the procedure for determining the cross-sectional behavior of the profile sheets i.e. moment of inertia and section modulus of the profile sheets which are the important parameters for determining the serviceability and strength requirements for construction stage of loading. The second section of the third chapter deals with the cross sectional behavior of the composite slab for positive and negative moment regions in connection with the situation when the section is in crack and uncracked conditions. The variations of second moment of area along the spans of the composite slab necessitate the development of composite slab cross-section model to account for the sectional property variability. Such model is provided at the end of the third chapter.

In the fourth chapter, appropriate expressions are developed for cross sectional resistance against flexure, shear and deflection for both none composite and composite situation of the steel sheet and concrete slab system.

Analysis and design of steel profile concrete composite slab construction for the three stages of loading i.e. construction, Serviceability & ultimate limit stage of loading are elaborated in the fifth chapter. As far as the analysis is concerned, non-prismatic continuous slab analysis concept is introduced to account for the cross-sectional property variation along the spans of the continuous slab as per the model developed in the third chapter.

Possibility of making use of locally available profile sheet products as a composite system with concrete for building slab construction supported with laboratory tests is dealt in the sixth chapter. Tensile strength of local steel sheet and shear bond interaction between profile sheet and concrete tests, the specimens for the tests collected from Kality Metal Industry, were conducted in the Material Laboratory of Technology Faculty of Addis Ababa University. Steel sheet profile-concrete composite slab design for an office building is also worked out as an example based on the formulations of analysis and design techniques developed in the previous chapters. To demonstrate the advantage of the composite slab construction under consideration over the conventional slab construction method, cost comparison between the two are made in the sixth chapter too.

The study did not give detailing about the subjects mentioned below in connection with steel profile concrete composite slab construction. It is hoped that further similar investigation will include the additional topics to fill the gap.

- Fixing and joining technique of profile sheets to each other either by welding or riveting when laid on the supporting beams and temporary props.
- Discussion on fireproofing provision requirements.
- Proto type shear bond test for parametric value determination for the justification of the validity of the method used for parametric value determination in this study.

# 2 PROFILED STEEL SHEET MATERIAL PRODUCTION AND PROPERTY

## 2.1 PRODUCTIONS

Like all other steel products, steel sheets are produced from basic open-hearth furnaces. The process is principally one of converting pig iron in to a product with smaller controlled quantity of carbon, manganese, phosphorous, sulfur & silicon and having the injurious quantities of oxides removed [11]. Carbon and manganese are the main elements to increase strength over that of pure iron. The category includes material from ingot iron containing essentially no carbon to cast iron, which has at least 1.7% carbon. These steels are divided in to four categories: low carbon content (less than 0.15%); mild carbon content (0.15%-0. 29%); medium carbon content (0.30-0.59%); and high carbon content (0.60-1.7%)[2]. Increased percentage of carbon raises the yield stress but reduces ductility, making welding more difficult. The product of the open-hearth furnace is molded in to ingots and further rolling will result in the formation of thick plate. The thick plate is then again rolled through hot rolling or cold rolling process to bring the end product with the required sheet thickness. Forming machinery then forms profiled steel sheets from the corrugation of the plane sheet.

### *2.1.1 Hot Rolled Steel Sheet*

Hot rolled steel sheet is defined as the product obtained by rolling heated steel through a continuous-type strip mill to the required sheet thickness and tolerances, the product having a surface covered with oxide or scale resulting from the hot-rolling operation. Hot-rolled steel sheet from which oxides or scale has been removed, commonly by pickling in acid solution or by mechanical means such as grit blasting is known as hot-rolled descaled steel sheet . Some changes in properties may result from descaling; some increase in hardness and some loss of ductility may result from descalling if mechanical means such as grift blasting is used. It is commonly produced in the range of thickness 0.8mm to 12.5mm inclusive, and in width of 600mm and over, in coils and cut lengths [11]. Usually galvanization (coating by zinc solution) is provided for hot rolled steel sheets of thickness less than 1.5mm inclusive. Except protection of the sheet against resting, galvanization does not alter the mechanical properties of the sheet metals.

### *2.1.2 Cold Rolled Steel Sheet*

Cold rolled steel sheet is produced from a hot-rolled pickled coil, which has been given substantial cold reduction (rolling at atmospheric temperature). The product is characterized by improved surface, greater uniformity in thickness and improved mechanical properties compared to hot-rolled strip. It is commonly produced in thickness of 6mm and under. Galvanization of lower gauge cold rolled sheets i.e. thickness from 0.2mm to 1.0mm are usually accomplished by the steel sheet manufacturers [11].

### *2.1.3 Types of Profiled Steel Sheet Forms*

A summary of the different sheeting profiles marked for use by British Standard in composite slabs are given in Figure 2.1[3]. Profile heights are usually in the range 38-75mm and sheet thicknesses are between 0.8 and 1.5mm. Plain open profiled sheets should not be used where composite action is required unless accompanied by some means of shear connection. Plain re-entrant angle profiled sheets (Figure-2.1a), embossments in the webs and/or flanges of the sheet (Figure 2.1b, 2.1c and 2.1d), small holes in profiled steel sheets and end anchorage are some of the techniques used today to develop shear connection for composite action.

Figure 2.1 Profiled Steel Sheet forms as per British Standard Specification.

## 2.2 PROPERTIES

### *2.2.1 Chemical Composition*

The chemical composition for hot rolled and cold rolled carbon steel of commercial and drawing qualities, which could be used as structural purposes, and are available in the local market have values not exceeding as given in *Table 2.1* and *Table 2.2* respectively [11].

**Table 2.1 Chemical composition of hot rolled steel sheet**

| Designation | Values as percentage by mass | | | |
|---|---|---|---|---|
| | Carbon Max. | Manganese Max. | Phosphorus Max. | Sulfur Max. |
| HR1 | 0.12 | 0.60 | 0.045 | 0.045 |
| HR2 | 0.10 | 0.45 | 0.035 | 0.035 |
| HR3 | 0.08 | 0.40 | 0.030 | 0.030 |
| HR4 | 0.08 | 0.35 | 0.025 | 0.025 |

**Table 2.2 Chemical composition of cold reduced carbon steel sheet.**

| Designation | Values as percentage by mass | | | |
|---|---|---|---|---|
| | Carbon | Manganese | Phosphorus | Sulfur |
| CS30 | 0.27 to 0.34 | 0.60 to 0.90 | 0.035 | 0.03 |
| CS35 | 0.31 to 0.38 | 0.60 to 0.90 | 0.035 | 0.03 |
| CS40 | 0.36 to 0.44 | 0.60 to 0.90 | 0.035 | 0.03 |
| CS45 | 0.42 to 0.50 | 0.60 to 0.90 | 0.035 | 0.03 |
| CS50 | 0.47 to 0.55 | 0.60 to 0.90 | 0.035 | 0.03 |
| CS55 | 0.52 to 0.60 | 0.60 to 0.90 | 0.035 | 0.03 |
| CS60 | 0.55 to 0.66 | 0.60 to 0.90 | 0.035 | 0.03 |
| CS65 | 0.59 to 0.70 | 0.60 to 0.90 | 0.035 | 0.03 |
| CS70 | 0.65 to 0.76 | 0.60 to 0.90 | 0.035 | 0.03 |
| CS75 | 0.69 to 0.80 | 0.40 to 0.70 | 0.035 | 0.03 |
| CS85 | 0.80 to 0.94 | 0.70 to 1.00 | 0.035 | 0.03 |
| CS95 | 0.90 to 1.04 | 0.30 to 0.60 | 0.035 | 0.03 |

### *2.2.2 Mechanical and Physical Properties*

The tensile strength and ductility (elongation) requirement for hot rolled and cold rolled steel sheets as per the International Standardization Organization recommendation are as given in *Table 2.3* and *Table 2.4* [11]. The modulus of elasticity for sheet profiles shall be taken as 210 KN/mm$^2$ [4]. Hardness is also one of the measures of the mechanical properties of metals and the method of measuring hardness is discussed in *section 6.3.1*.

**Table 2. 3 Mechanical property requirements for hot-rolled carbon steel sheet**

| Base metal quality Designation | Ultimate Strength ($f_u$*) in MPa. | S Min. % | | | |
|---|---|---|---|---|---|
| | | t<3mm | | $3mm \leq t \leq 6mm$ | |
| | | $L_0$ = 80mm | $L_0$ = 50mm | $L_0 = 5.65\sqrt{A_o}$ | $L_0$ = 50mm |
| HR1 | 440 | 23 | 24 | 28 | 29 |
| HR2 | 420 | 25 | 26 | 30 | 31 |
| HR3 | 400 | 28 | 29 | 33 | 34 |
| HR4 | 380 | 31 | 32 | 36 | 37 |

S Percentage elongation
t Thickness of sheet metal
$L_o$ Original length of the testing specimens

**Table 2. 4 Mechanical properties of annealed cold reduced carbon steel strip.**

| Designation | Ultimate strength $f_u$( MPa.) | S | |
|---|---|---|---|
| | | $L_o$ = 50mm | $L_o$ = 80mm |
| CS30 | 585 | 18 | 16 |
| CS35 | 590 | 17 | 15 |
| CS40 | 595 | 16 | 14 |
| CS45 | 600 | 16 | 14 |
| CS50 | 605 | 15 | 13 |
| CS55 | 610 | 15 | 13 |
| CS60 | 620 | 14 | 12 |
| CS65 | 630 | 13 | 11 |
| CS70 | 640 | 12 | 10 |
| CS75 | 640 | 12 | 10 |
| CS85 | 670 | 12 | 10 |
| CS95 | 680 | 12 | 10 |

S Percentage elongation
$L_o$ Original length of the testing specimens

* Ultimate Strength ($f_u$ ) is defined as the maximum tensile strength of the profile steel sheet before failure point is reached.

## 2.3 LOCALLY AVAILABLE PROFILED STEEL SHEET FORMS

There are a number of local manufacturers, which are importing steel sheets in the form of coils and cut lengths; from which profiled steel sheets are produced by forming machinery mainly for roofing and cladding purposes. *Table 2.5* shows the range of sizes of steel sheets imported by local factories for the production of construction materials such as rectangular and circular tubes, tankers, profiled steel sheets, etc. .The standard forms of the profiled steel sheets are all the same for all the manufacturers locally available. Appendix-A shows the standard forms of profiled steel sheets produced by local manufacturers [13]. These forms of profile sheets i.e. EGA-300, EGA-400, EGA-500, EGA-600 & EGA-700 have been hardly used for composite slab construction for buildings. The study therefore aims at assessing the possibility of introducing concrete and steel profile composite slab out of the profiled steel sheets produced locally.

**Table 2.5 Type of sheet metals imported by local factories**

| Item No. | Production Firm | Steel Sheet Type | Sheet Metal Thickness in mm | Plan Dimension |
|---|---|---|---|---|
| 1 | Kality Metal Factory | Galvanized steel sheets | 0.25,0.3,0.4,0.5,0.6,0.7, 0.8 & 1.0,1.5 | One meter width in coils |
| | | Non-galvanized steel sheets | 0.8,1.0,1.2, 1.5,1.6,1.8, 2.0,2.5 & 3.0 | One meter width in coils |
| | | | 4.0,5.0,6.0, 8.0,10.0,12.0 | 1.0 Meter Wide & 2.0 meters length |
| 2 | GATEPRO Metal Industry | Galvanized steel sheets | 0.25,0.3,0.4,0.5,0.6,0.7, 0.8 & 1.0 | One meter width in coils |
| | | Non-galvanized steel sheets | 4.0,5.0,6.0, 8.0,10.0,12.0 | 1.5 Meter Width & 6.0 meters length |
| 3 | Maru Tefera Metal Factory | Galvanized steel sheets | 0.25,0.3,0.4,0.5,0.6,0.7, 0.8 & 1.0,1.5 | One meter width in coils |
| | | Non-galvanized steel sheets | 1.5,2.0,3.0,4.0,5.0,6.0, 8.0,10.0,12.0 | In coils for sheets less than 4.0mm & in cut lengths for 4mm and above thickness |
| 4 | Kombelcha Steel Product industry (KOSPI) | Galvanized steel sheets | 0.25,0.3,0.4,0.5,0.6,0.7, 0.8 | One meter width in coils |
| | | Non-galvanized steel sheets | - | - |

# 3 CROSS-SECTIONAL PROPERTIES

## 3.1 PROFILED STEEL SHEET

The purpose of providing the profiled cross-section is to attain stiff flexural deck, which could not be achieved if the flat sheet without such profiled shape had been used as a load-bearing element. During construction stage only the profiled steel sheet is acting as load bearing structure to support the self weight of the sheet, the weight of wet concrete & the construction load .The cross-sectional property should therefore be first determined to compute the flexural strength, shear resistance and deflections of the profile during construction stage. Position of the neutral axis, second moment of area about the axis of bending and the sectional modulus of the profiled steel sheet are the cross sectional properties to be defined before any further analysis. Using the designation given in Figure 3.1 of typical trapezoidal profiled steel sheet, the sectional properties are calculated.

### *3.1.1 Position of the Neutral Axis*

Taking the moment of the area of the section about top of the profile sheet the location of the neutral axis ($y_c$ ) can be computed as :

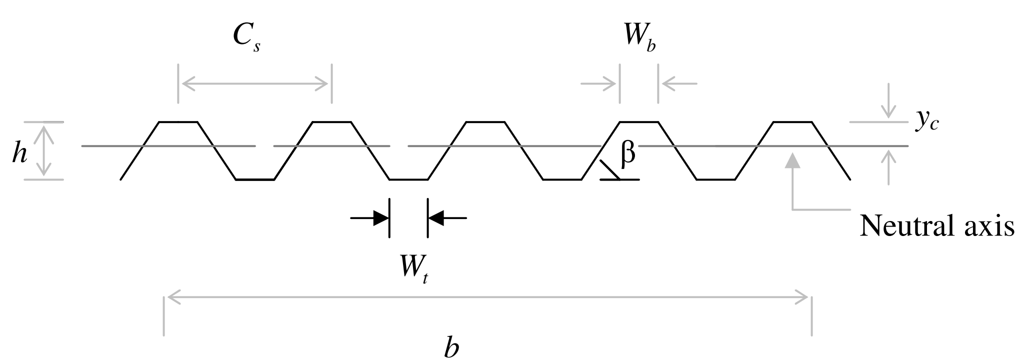



Figure 3.1 Typical cross-section of trapezoidal profiled sheet

$$y_c = \left[\left(W_b t^2/2\right)\left(b/C_s + 1\right) + W_t t\left(h - t/2\right)b/C_s + \left(h/\sin\beta\right)th\left(b/C_s + 1\right)\right]/A_{sp} \tag{3.1}$$

Where

$A_{sp}$ is cross sectional area of profiled steel sheet for a width of b units as in Figure 3.1, which is equal to $d_{lb} t$.

$t$ is thickness of the steel sheet.

$d_{lb}$ is development length of the profiled steel sheet for a width of b units as in Figure 3.1.

### *3.1.2 Second Moment of Area & Section Modulus*

Second moment of area ($I_x$) can be calculated as:

$$
\begin{aligned}
I_x = &\left(\frac{1}{12}W_b t^3 + W_b t(y_c - t/2)^2\right)(b/C_s + 1) + \left(\frac{1}{12}W_t t^3 + W_t t(h - y_c - t/2)^2\right) b/C_s \\
&+ 2\left(\frac{th^3}{12\sin\beta} + \frac{th}{\sin\beta}(h/2 - y_c)^2\right)(b/C_s + 1)
\end{aligned}
\tag{3.2}
$$

The sectional modulus of the profiled sheet $S_{xt}$ & $S_{xb}$ for the top and bottom fiber respectively are calculated as:

$$S_{xt} = I_x/y_c \tag{3.3}$$

$$S_{xb} = I_x/(h-y_c) \tag{3.4}$$

It should be noted that for profiled sheeting having different shape from the one illustrated above slight modification in the computation of sectional properties is necessary.

## 3.2 PROFILED STEEL SHEET-CONCRETE COMPOSITE SLAB

The sectional property of the profiled steel sheet and concrete during the composite action is computed for two cases. These cases are when the section is cracked on and uncracked on.

### *3.2.1 Second Moment of Area for Cracked Section*

When the tensile stress in the composite slab subjected to flexural stress exceeds the modulus of rupture of concrete, the section is said to be cracked [1]. For cracked section one need to account of the fact that all the concrete, which is stressed in tension, is assumed cracked, and therefore effectively absent. For positive (sagging) moment the section is then consists of the concrete in compression above the neutral axis and the steel profile at the bottom in tension. For negative (hogging) moment the concrete below the neutral axis is subjected to compression and the top reinforcements are in tension. Due to local buckling effect the profile steel sheet is not effective for compressive force resistance, and hence shall not be accounted in the sectional property calculation.

The expressions in Equation 3.7 & Equation 3.9 are the second moment of area for cracked section subjected to positive and negative moments, respectively. The position of the neutral axis $y_{cc}$ to be used for the computation of second moment of area shall be determined by taking moment of the area of the concrete in the compression region and the moment of the transformed area of the profile steel sheet for positive moment regions or the moment of the transformed area of reinforcement bars for negative moment regions about the neutral axis.

i) *Second moment of area for positive moment*

Taking moment of the area about the neutral axis, the position of the neutral axis can be determined as from the top of the slab ($y_{cc}$) as shown in Figure 3-2(a).

$$\frac{y_{cc}^2}{2}b - nA_{SP}(d - y_{cc}) = 0 \tag{3.5}$$

With further simplification and the practical solution is:

$$y_{cc} = d\left[\left(2\rho n + (\rho n)^2\right)^{\frac{1}{2}} - \rho n\right] \tag{3.6}$$

Second moment of area for cracked section ($I_c$) is therefore can be calculated as:

$$I_{cp} = \frac{1}{3}by_{cc}^3 + nI_x + nA_{sp}y_{cs}^2 \tag{3.7}$$

Where:

ρ is the ratio of the profiled steel sheet area to the gross cross sectional area($A_{sp}/bd$).

n is the ratio of the modulus of elasticity of steel sheet($E_s$) to modulus of elasticity of concrete($E_c$).

$A_{sp}$ is the cross sectional area of the profiled steel sheet for slab width of b units.

*d* is the depth of the slab from top fiber of the composite slab to the center mass of the profiled steel sheet or to the neutral axis position of the profile sheet as shown in Figure 3.1 .

h is depth of profile steel sheet.

$y_c$ is neutral axis position of the profile steel sheet (Figure 3.1) from top fiber of the profile steel sheet.

d , b , $y_{cc}$ & $y_{cs}$ are as indicated in Figure 3-2, and

$I_x$ is as defined in equation (3.2).

ii) *Second moment of area for negative moment*

Taking moment of the area about the neutral axis, the position of the neutral axis can be determined as from the top of the slab ( $y_{cc}$ ) with reference to Figure 3.2(b) and Figure 3.1 as:

$$\left((D - y_{cc})^2/2\right)b - \frac{b}{C_s}\left[W_b h(D - y_{cc} - h/2) + hl(D - y_{cc} - h/3)\right] - nA_S\left(d - (D - y_{cc})\right) = 0 \qquad (3.8)$$

Equation (3.8) is a quadratic equation in $y_{cc}$ , and hence the neutral axis depth can be obtained by solving Equation(3.8).

Second moment of area for cracked section subjected to negative moment ($I_{cn}$) is therefore can be calculated as:

$$I_{cn} = \frac{b}{C_s}\left[\frac{1}{3}(D - y_{cc})^3 W_t + \frac{1}{3}(D - y_{cc} - h)^3 (C_s - W_t) + hl_i\left(\frac{h^2}{18} + \left(D - y_{cc} - \frac{2}{3}h\right)^2\right)\right] \qquad (3.9)$$
$$+ nI_{rb} + A_s\left(y_{cc} - c\right)^2$$

Where:

$D$ is overall depth of steel profile concrete composite slab.

$A_s$ is the area of negative reinforcements for the width of the slab equal to b units.

$I_{rb}$ is the second moment of area of negative reinforcement bars about its own neutral axis but it small & can be neglected.

$c$ is negative reinforcement concrete cover plus half of the diameter of negative reinforcement bar.

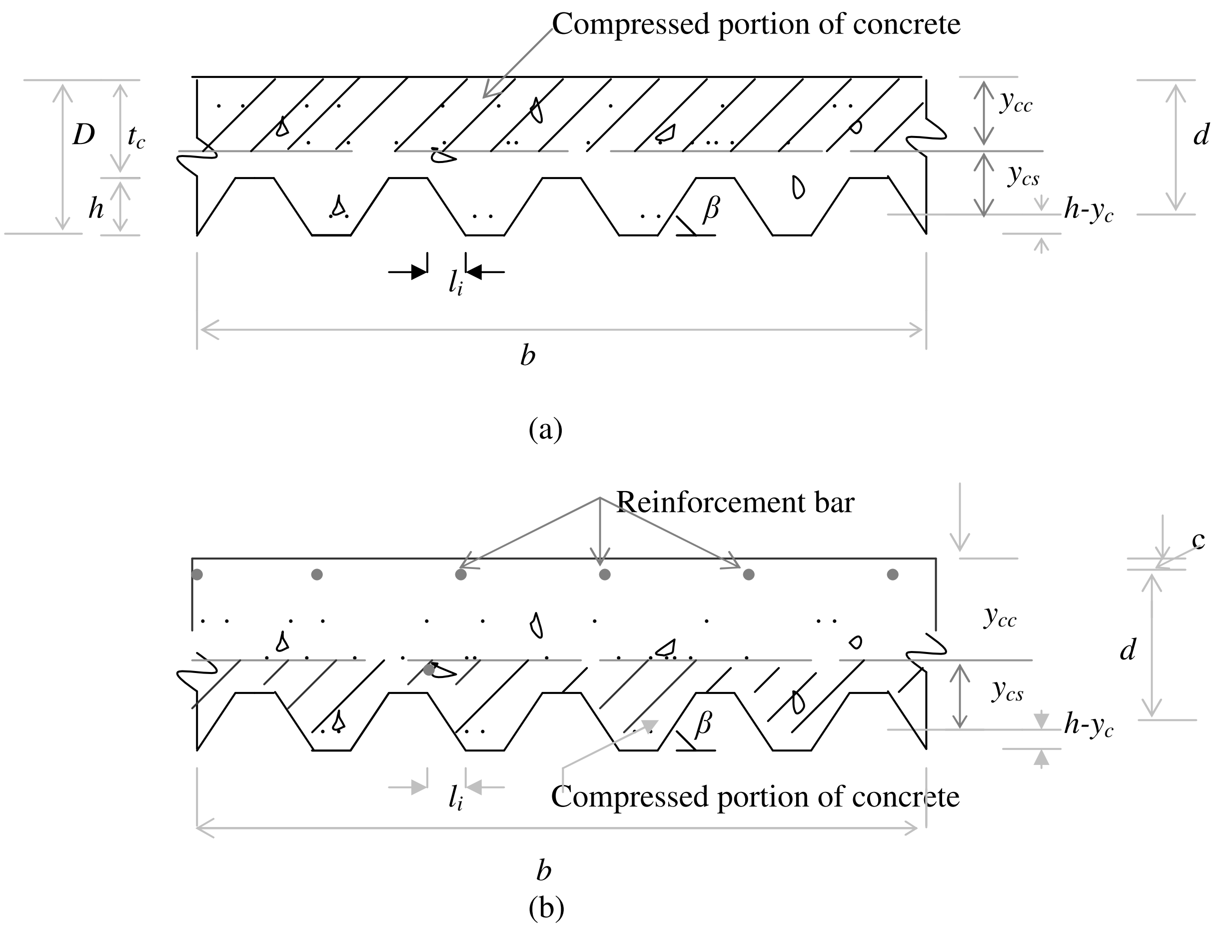



Figure 3-2 Typical cross-section of steel profile & concrete composite slab .
(a) positive moment case and (b) negative moment case.

### *3.2.2 Second Moment of Area for Uncracked Section*

When the tensile stress in the composite slab is subjected to flexural stress less than the modulus of rupture of concrete, the section is said to be uncracked [1]. For un cracked section all the concrete area which is stressed in tension is assumed to carry tensile stress. The second moment of area for un cracked section for positive and negative moment can be determined as below.

*i) Second moment of area for positive moment ( uncracked section)*

Figures 3.2(a) & 3.2(b) can be used for the computation of sectional property of the composite slab for the uncracked situation too.

Position of the neutral axis, taking the first moment of the area of the uncracked composite slab about top of the slab can be determined as:

$$y_{cc} = \frac{\frac{b}{C_s}\left[\frac{D^2}{2}W_t + \frac{t_c^2}{2}(C_s - W_t) + l_i h\left(D - \frac{2}{3}h\right)\right] + nA_{sp}d}{\frac{b}{C_s}\left[W_t D + t_c(C_s - W_t) + l_i h\right] + nA_{sp}} \tag{3.10}$$

Second moment of area for uncracked section subjected to positive moment ($I_{up}$) is then given as:

$$\begin{aligned} I_{up} &= \frac{bt_c^3}{12} + bt_c(y_{cc} - 0.5t_c)^2 + W_t h\frac{b}{C_s}\left[\frac{h^2}{12} + (D - y_{cc} - 0.5h)^2\right] \\ &+ l_i h\frac{b}{C_s}\left[\frac{h^2}{18} + \left(D - y_{cc} - \frac{2}{3}h\right)^2\right] + nI_x + nA_{sp}(t_c - y_{cc} + y_c)^2 \end{aligned} \tag{3.11}$$

*ii) Second moment of area for negative moment (uncracked section)*

The assumption of the in effectiveness of the profiled sheet for compressive force will also hold true for uncracked case.

Position of the neutral axis, taking the first moment of the area about bottom of the slab:

$$y_{cc} = D - \frac{\frac{b}{C_s}\left[\frac{D^2}{2}W_t + t_c(C_s - W_t)\left(\frac{t_c}{2} + h\right) + \frac{2}{3}l_i h^2\right] + nA_s(D-c)}{\frac{b}{C_s}\left[W_t D + t_c(C_s - W_t) + l_i h\right] + nA_s} \tag{3.12}$$

Second moment of area for uncracked section when subjected to negative moment ($I_{un}$) is calculated as:

$$\begin{aligned} I_{un} = {} & \frac{bt_c^3}{12} + bt_c(y_{cc} - 0.5t_c)^2 + W_t h\frac{b}{C_s}\left(\frac{h^2}{12} + (D - y_{cc} - 0.5h)^2\right) \\ & + l_i h\frac{b}{C_s}\left(\frac{h^2}{18} + \left(D - y_{cc} - \frac{2}{3}h\right)^2\right) + nI_{rb} + nA_s(y_{cc} - c)^2 \end{aligned} \tag{3.13}$$

The average second moment of area for positive moment region ($I_p$) and for negative moment region ($I_n$) are thus given by Equations (3.14) & (3.15) respectively.

$$I_p = \frac{I_{cp} + I_{up}}{2} \tag{3.14}$$

$$I_n = \frac{I_{cn} + I_{un}}{2} \tag{3.15}$$

## 3.3 CROSS SECTIONAL VARIATION OF THE COMPOSITE SLAB ALONG THE SPAN

From the discussion of variation of cross sectional property of the composite slab, depending on either the section is subjected to negative or positive bending moment, the cross section of continuous composite slab is modeled to represent the actual sectional behavior when subjected to design loads. Inflection point, the point in a member at which the bending moment changes in sign, is approximated to draw the non-prismatic continuous composite slab. The points of inflection under uniformly distributed load are known to be located at *0.211Lsi* from the supports of the spans as shown in Figure 3.3[1].

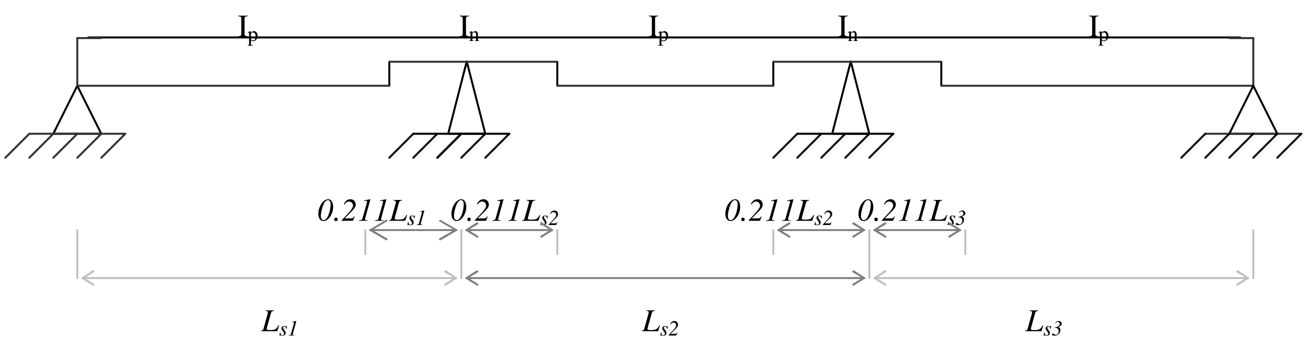



Figure 3.3 Cross-sectional Model of Composite Slab

# 4 CROSS-SECTION RESISTANCE

The steel sheet thickness and depth of profiled steel sheet are usually determined by the requirement that during construction and before concrete hardening the profiled steel sheet must carry its own weight and that of the concrete plus such other construction loads as may occur. This means that even though the profiled sheet in the finished structure serves as reinforcement for the slab, its cross sectional dimensions and thus its areas ($A_{sp}$) are generally governed by the temporary construction condition just described [3]. In consequence, composite slabs may be over reinforced or under reinforced depending on the particular combination of spans, loads, material strengths and shoring conditions. It is not possible to avoid over reinforced slabs in contrast to conventional slabs with the usual bar reinforcement. The other essential feature of composite slab construction is that flexural bond stresses along the interface between steel and concrete must not cause horizontal slip between the two components. Just as was true for ordinary-reinforcing bars, natural bond alone is generally not sufficient. For this reason, a variety of shear transfer devices should be provided. Deflection of the steel profile during the construction stage and the deflection of the steel concrete composite slab during service period should be below the allowable limiting value for proper functional requirement.

## 4.1 FLEXURE

According to the stress and deformation relation under load, the flexural resistance of the profiled steel sheet and composite slab are determined either based on elastic theory or plastic theory of structural resistance behavior as will be discussed in *section 4.1.1* and

*4.1.2* below. Structural behavior in the elastic region under working load follows Hook's law, i.e. stress and strain relation in the structure are with in the proportional limit up to the yielding stress. Plastic theory has been established as a practical tool in structural design for the determination of the ultimate-load capacity of structures. Typically, structural steel yields at a strain of the order 0.1%, and there after the strain increases at constant stress to at least ten times of the yielding strain before strain hardening occurs at a large strain of the order 1.5%. The assumption of perfect plastic after the yield stress is reached amounts to ignoring the effects of strain hardening and errs on the safe side. For a member subjected to flexural stress, yielding moment is reached when the extreme fiber stress just equal to the yielding stress. As the moment is increased beyond the yielding moment, the extreme fiber strain will continue to increase and the maximum strains will exceed the yielding strain, but the maximum stress remain constant. As the moment further increase, more and more fibers become plastic until finally the whole cross-section is plastic; the bending moment at full plasticity is called the plastic moment resistance or simply the plastic moment [10].

### *4.1.1 Flexural Resistance of Profiled Steel Sheet*

The loads carried by the profiled steel sheeting should be the dead load of the sheets, weight of wet concrete and the construction loads. Moreover, the effect of any temporary propping used at this stage may also be considered.

The Elastic moment capacity of the section using Figure 4.1 can be given as:

$$M_{ps} = \frac{f_{al} I_X}{y} \tag{4.1}$$

Where

$M_{ps}$ is the moment carrying capacity of the profiled steel sheet

$f_{al}$ is the allowable stress of the profiled sheet

$y$ is the largest of $y_c$ and $y_b = h - y_c$ and $I_x$ is as defined in 3.1

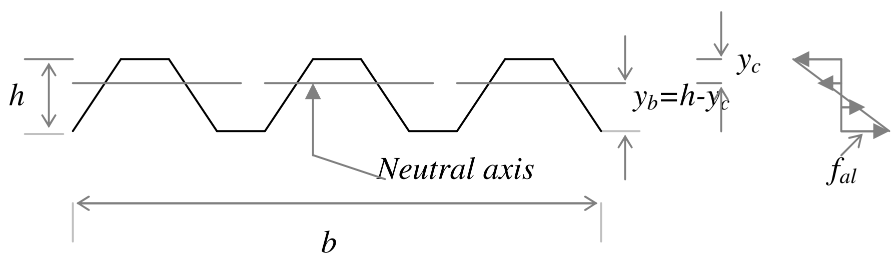



Figure 4-1 (a) Cross section of profiled steel sheet xx

(b) Elastic stress distribution in the section

#### *4.1.2 Flexural Resistance of Profiled Steel Sheet and Concrete Composite Slab*

For relatively shallow decks and deep slabs i.e., when the slab thickness $D$ is considerably larger than the depth $h$ of the steel deck (see Figure 4.2 (a)), yielding is likely to spread over the entire depth $h$ before the concrete compressive strain has reached the limiting value $\varepsilon_u$ =0.0035*. Then the steel tensile force acts at the centroid of the steel deck section. In this case, the slab is under reinforced and the usual equation for the normal flexural resistance applies:

* Ethiopian Code of standard (EBCS-2, 1995) recommends the ultimate limit strain for concrete ($\varepsilon_u$) is equal to 0.0035

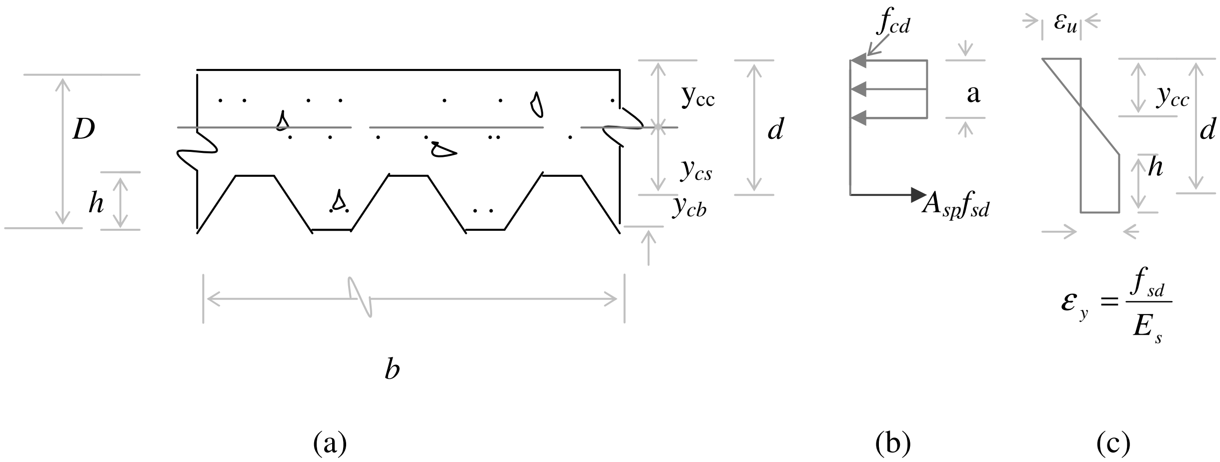



Figure 4-2 (a) Cross-section of profiled steel sheet and concrete composite slab
(b) Stress distribution of a cross-section for ultimate Limit State
(c) Strain diagram for balanced composite section ( ultimate strains for concrete and profiled steel sheet reach at the same time)

$$M_{psc} = A_{sp} f_{sd}\left(d - \frac{a}{2}\right) \qquad (4.2)$$

Where

$$a = \frac{A_{sp} f_{sd}}{f_{cd} b} = \beta_1 y_{cc}$$

$d$ is effective depth from top of slab to steel profile centroid (see figure 4-2(a)),

$b$ is width of unit strip,

$A_{sp}$ is area of profiled steel cross section,

$f_{sd}$ is design yielding strength of profiled steel sheet, c

$$f_{sd} = \frac{f_{yk}}{\gamma_s}$$

$f_{cd}$ is compressive design strength of concrete,

$$f_{cd} = \frac{0.85 f_{ck}}{\gamma_f}$$

$f_{ck}$ is characteristics compressive strength of concrete, and

$f_{yk}$ is characteristics yielding strength of profiled sheet,

$\gamma_s$ and $\gamma_f$ material partial safety factors for steel profile and concrete respectively,

$\beta_1$ is a coefficient for equivalent rectangular stress block.

An equation for the balanced steel ratio is easily derived, based on the condition that, for the balanced case, the tensile strain at the top surface of the steel deck panel reaches the yield strain at exactly the same instant of loading that the top surface of the concrete reaches its limit strain $\varepsilon_u$ =0.0035. The derivation is exactly analogous to that for the balanced steel ratio for ordinary bar-reinforced beams.

From the stress block diagram of figure 4-2(b) and equilibrating the compressive and tension forces:

$$A_{sp} f_{sd} = f_{cd} ab = f_{cd} \beta_1 y_{cc} b$$

$$\rho_b f_{sd} bd = f_{cd} \beta_1 y_{cc} b$$

From the strain diagram of Figure 4.2(c)

$$\frac{y_{cc}}{\varepsilon_u} = \frac{D-h}{\varepsilon_u + \varepsilon_y}$$

=>

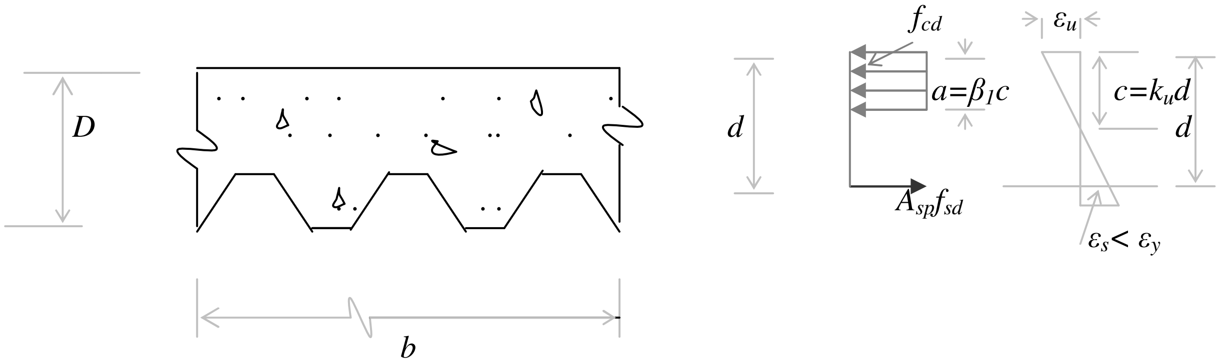
$$\rho_b = \beta_1 \frac{f_{cd}}{f_{sd}} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_y} \frac{D-h}{d}$$

where

$$\rho_b = \frac{A_{sp}}{bd} (\mathit{forbalanced}\ \mathrm{sec}\, \mathit{tion})$$

Composite slabs with a steel ratio less than $\rho_b$ are under reinforced, while those with a steel ratio above that limit are over reinforced with the steel stress less than $f_{sd}$ when the concrete strain limit is reached. Calculation of the flexural strength of over reinforced slabs is complicated by the fact that the strains in the metal deck section vary with the depth depending up on their loading history, i.e., whether the steel deck was loaded in a no composite stage, whether it was shored and how, etc. For the case where continuous shoring is provided in the structure, where all loads are composite, and with the approximation that the steel stress through the full dept of the deck is equal to its value at the centroid, a strain compatibility analysis leads to the following results [1].

Referring to a typical composite slab cross section as shown in Figure 4-3(a), with strains at flexural failure as shown in Figure 4-3(c),




(b) (b) (c)

Figure 4-3 Flexural strain compatibility analysis for over reinforced composite slab:

(a) Cross-section; (b) Stresses and forces; (c) Strain distribution.

$$\varepsilon_s = \varepsilon_u \frac{d - k_u d}{k_u d}$$

$$f_s = \varepsilon_u E_s \frac{1 - k_u}{k_u d}$$

Summing forces in the X-direction in Figure 4-3(b) and setting equal to zero results in :

$$\rho b d \varepsilon_u E_s \frac{1 - k_u}{k_u} = \beta_1 f_{cd} b k_u d \qquad \text{(a)}$$

Defining the material parameter,

$$\lambda = \frac{\varepsilon_u E_s}{\beta_1 f_{cd}} \qquad \text{(b)}$$

Equation (a) can be written as a quadratic in $k_u$ as follows:

$$K_u^{\,2} + \rho\lambda k_u - \rho\lambda = 0 \qquad \text{(c)}$$

From which

$$k_u = \sqrt{\rho\lambda + (\rho\lambda/2)^2} - \rho\lambda/2 \qquad (4.4)$$

The flexural resistance can then be found from the equation:

$$M_{psc} = f_{cd} ab(d - a/2) \tag{4.5}$$

If continuous shoring is not provided during construction, prior strains in the steel panel due to construction loads affect the strain distribution in the composite section at failure, and the simple strain diagram of Figure 4-3(c) is not strictly valid. However, neglect of these prior strains caused by construction load for unshored construction case, is fully consistent with assuming that the entire deck section acts at its centroidal stress $f_{sd}$ as was done in the previous analysis, and would usually lead to only a small error [1]. Prior strain in the profile steel sheet, during the construction stage due to unshored condition, is considered as shown in Figure 4.4(b) for driving the resistance equation valid for partly or fully unshored construction condition. The strain $\varepsilon_{sc}$( compressive strain) and $\varepsilon_{st}$( tensile strain) represent the prior strain at the top and bottom fiber of the profile steel sheet. Due to the removal of temporary shoring and application of additional imposed loads soon after the completion of construction phase, the strain diagram changes its original shape and takes the form as shown in Figure 4.4(c). The assumption made to draw the new strain diagram is that the profile sheet is allowed to reach yielding strain at only one point i.e. at the bottom outer most fiber. The strain at the top fiber would be $\varepsilon_y -(\varepsilon_{sc}+\varepsilon_{st})$ and from similarity of triangles relation, the strain at the center of the profile sheet would become $\varepsilon_y -(\varepsilon_{sc}+\varepsilon_{st})(1-y_c/h)$.

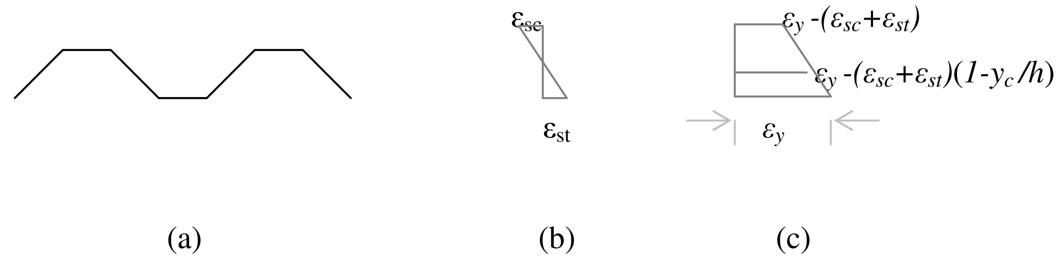
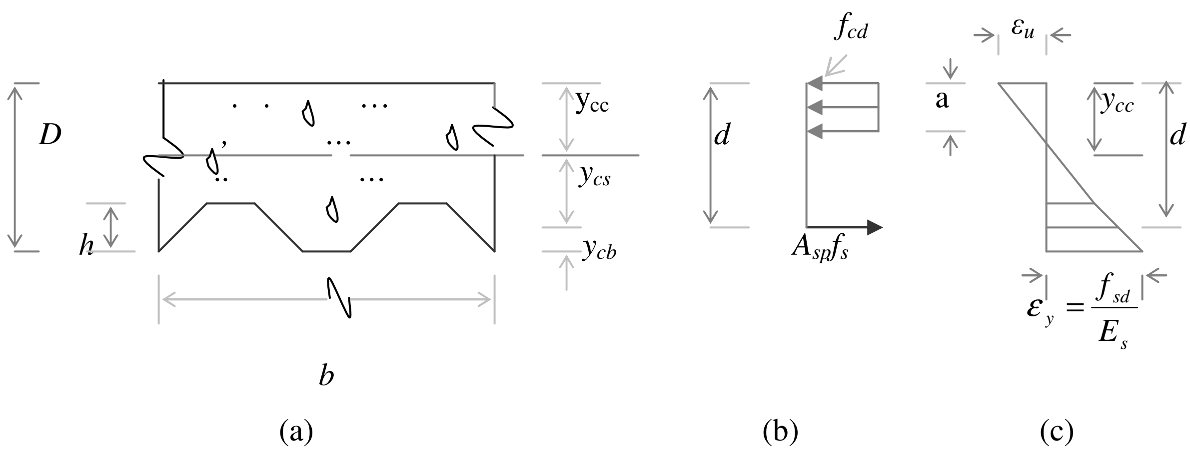



Figure 4.4 Strain diagram for un shored construction for profiled steel sheet.

(a) profile sheet cross-section
(b) strain at construction stage
(c) strain after completion of construction

Because of unshored construction  the stress and strain block for the composite slab will look Figure 4.5(b) and(c).




where $f_s = E\left(\varepsilon_y - (\varepsilon_{sc} - \varepsilon_{st})\left(1 - \frac{y_c}{h}\right)\right)$

Figure 4.5   Stress and Strain diagram for the composite slab for un shored construction.

(a) Composite slab cross-section
(b) Stress diagram
(c) Strain diagram

Therefore from the stress and strain diagram, moment resisting capacity and balanced steel ratio are given by:

$$M_{psc} = A_{sp}E\left(\varepsilon_y - (\varepsilon_{sc} + \varepsilon_{st})\left(1 - \frac{y_c}{h}\right)\right)(d - a/2) \tag{4.6}$$

$$\rho_b = \beta_1\left[\frac{f_{cd}}{E\left(\varepsilon_y - (\varepsilon_{sc} + \varepsilon_{st})\left(1 - \frac{y_c}{h}\right)\right)}\right]\left[\left[\frac{\varepsilon_u}{[\varepsilon_u + \varepsilon_y - (\varepsilon_{sc} + \varepsilon_{st})]}\right]\frac{D-h}{d}\right] \tag{4.7}$$

## 4.2 SHEAR

### *4.2.1 Shear on the Profiled Steel Sheet*

During construction stage, long profiled steel sheets may be governed by deflection and medium length beams are usually controlled by flexural stress, short-span beams may be governed by shear[ 4 ]. To review the development of the shear stress equation for the profiled steel sheet, consider the slice *dx* of the profiled steel sheet of Figure 4.6(a) where the stress on the element is shown by Figure 4.6(c). If the unit shear stress $\upsilon$ at a section $y_1$ from the neutral axis is desired, it is observed from Figure 4.6 (d) that

$$dC' = \upsilon t' dx \tag{4.6}$$

Where $t'$ is the sum of the thicknesses of the profile sheet with in $b$ unit width intersected by the horizontal line passing through the plane where shear stress is to be determined.

The horizontal forces arising from bending moment are

$$C' = \int_{y_1}^{y_2} \sigma_X \, dA$$

$$C' + dC' = \int_{y_1}^{y_2} (\sigma_x + d\sigma_x) dA$$

Subtracting,

$$dC' = \int_{y_1}^{y_2} d\sigma_x dA \tag{4.7}$$

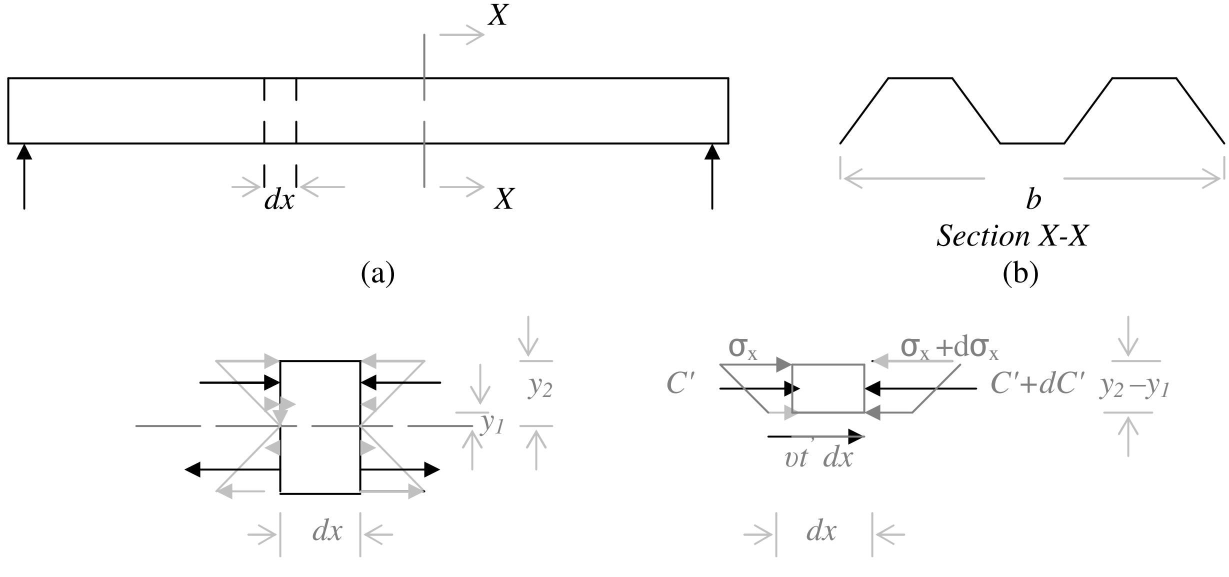



Figure 4.6 Flexural stress on profiled sheet for derivation of shear stress equation

$$d\sigma_x = \frac{dMy}{I} \tag{4-8}$$

$$dC' = \int_{y_1}^{y_2} \left(\frac{dMy}{I}\right) dA = \left(\frac{dM}{I}\right) \int_{y_1}^{y_2} y dA \tag{4.9}$$

Substituting Eq. (4-9) in to Eq. (4-6) and solving for the shear stress $\upsilon$ gives

$$\upsilon = \left(\frac{dM}{dx}\right)\left(\frac{1}{tI}\right)\int_{y_1}^{y_2} ydA \tag{4.10}$$

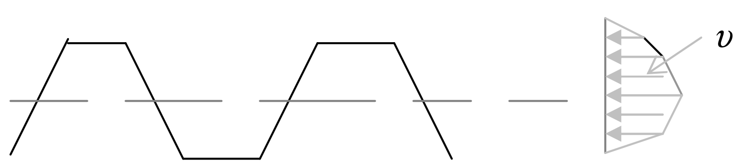
*and up on recognizing that* $V = \dfrac{dM}{dx}$ *and letting*

$$Q = \int_{y_1}^{y_2} ydx$$

the familiar equation,

$$\upsilon = \frac{VQ}{It^{'}} \tag{4.11}$$

is obtained where $Q$ is the first moment of area about the neutral-axis of the area between the extreme fiber and the particular location at which the shear stress is to be determined , i.e., the area lying between the limits $y_1$ and $y_2$ in Figure 4.6(c). From Equation 4.11, the shear stress distribution along the cross-section can be computed and the resulting possible distribution is drawn in Figure (4.7) b.




(a) (b)

Figure 4-7 Shear stress distributions along the cross-section of the profile sheet

### *4.2.2 Shear Flow at Interface*

As far as proper inter lock between the profiled steel sheet and concrete is maintained, the shear force developed due to flexural bending could be determined from equilibrium of forces. Consider an infinitesimal element of the composite slab *dx* shown in Figure 4.7(a)

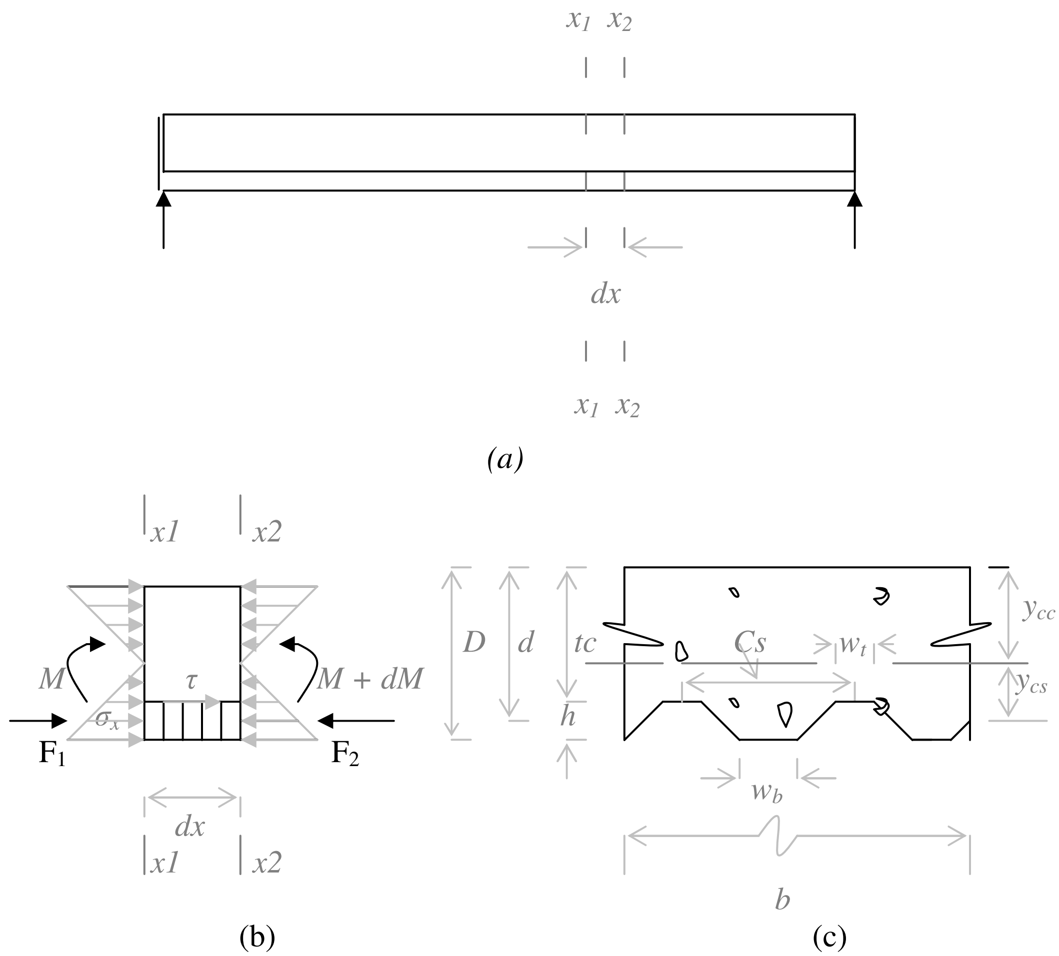



Figure 4 –7(a) composite slab

(b) infinitesimal element from the composite slab with the actions indicated

(c) cross-section of composite slab

From the equilibrium of the shaded region of the infinitesimal element,

$$F_1 = \int_{D-h}^{D} \sigma_x dA = \int_{D-h}^{D} \left(\frac{My}{I}\right) dA$$

Like wise ,

$$F_2 = \int_{D-h}^{D} \left( \frac{M + dM}{I} \right) ydA$$

$$dF_s = \tau b dx = F_2 - F_1$$

Substituting for $F_1$ and $F_2$ ,

$$dF_s = \frac{dM}{I} \int_{D-h}^{D} ydA = \frac{dM}{I} Q$$

Where,

$$Q = \int_{D-h}^{D} ydA$$

$$\int_{\tau=0}^{\tau=\tau_{MAX}} dF_s = \int_{\tau=0}^{\tau=\tau_{\max}} \frac{dM}{I} Q$$

The above integral equation can be decomposed in two parts,

(i) Total surface shear flow in the positive moment region ( $F_s^{+ve}$ ).

(ii) Total surface shear flow in the negative moment region ( $F_s^{-ve}$ ).

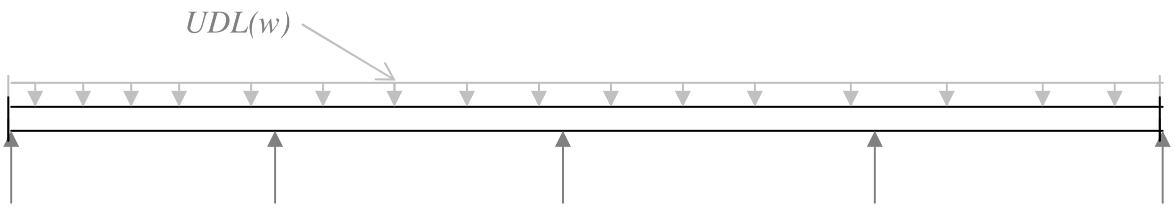
Therefore,

$$F_s^{+ve} = \frac{M^{+ve} Q}{I} \qquad (4.14a)$$

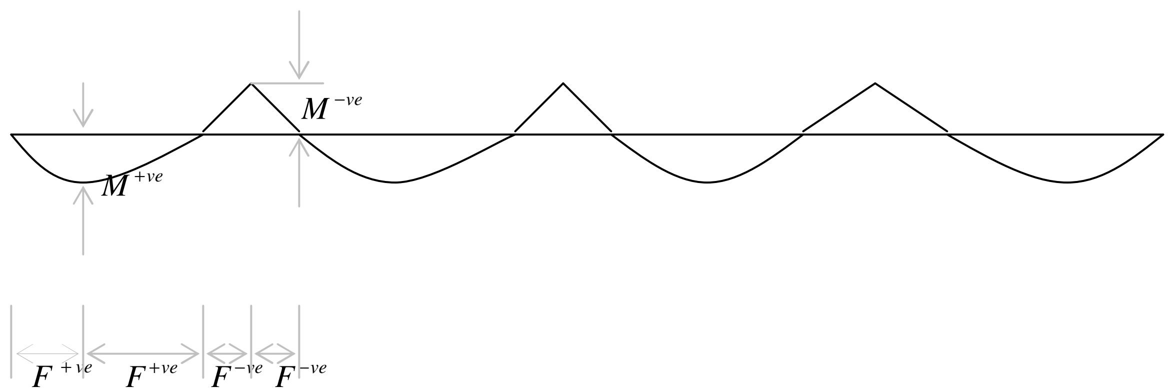
$$F^{-ve} = \frac{M^{-ve} Q}{I} \qquad (4.14b)$$

Where $M^{+ve}$ & $M^{-ve}$ are the positive and negative moments respectively in the composite profiled steel sheet and concrete slab.

Figure 4-8 shows how the shear flow distribution in a unit strip of continuous slab loaded with uniformly distributed load appears according to the moment envelope.




(a)




(b)

Figure 4.8 (a) Continuous composite slab of unit width strip loaded with uniformly distributed load (*w*)
(b) Moment envelope of the slab and surface shear force distribution between the interface of the profiled sheet and the concrete.

### *4.2.3 Chemical and Mechanical Shear Interlock*

Experiments have shown that bending moment resistance can only be reached, if proper inter lock is maintained at the interface between the profiled steel sheet and the concrete. Vertical shear is only of concern in the special cases of short spans with high loads levels. Most commonly, failure happens due to the insufficient slip resistance [17].

Shear interlock or slip resistance develops as the concrete slab gets hardened. Chemical bond is established on the surface of the sheeting by adhesive force. This connection is brittle in nature and its strength is strongly influenced by the surface quality. Since chemical bond is unreliable from the design point of view, it is usually neglected. If the composite slab is loaded and the chemical bond breaks, mechanical mechanism must be present to continue the longitudinal shear transfer. Small embossments, with various patterns, can be rolled or break pressed in to the flange or the web of the sheeting. They increase the geometrical irregularity of the surface, and by this way higher slip resistance develops [17]. Shear interactions established between profiled steel sheet and concrete could be assessed in two basic methods discussed below.

#### *4.2.3.1 Two point load proto type shear test*

An essential feature of composite slab construction is that flexural bond stresses along the interface between steel and concrete must not cause horizontal slip between the two components. Just as was true for ordinary reinforcing bars, natural bond alone is generally not sufficient. For this reason, a variety of shear transfer devices are used. In most cases these consist of closely spaced embossments or holes, welded buttons, or transverse wires. They act in a way similar to the ribs of deformed rebars.

Under load, a composite slab may fail in a mode that combines shear and bond failure, and hence is known as shear bond failure, with certain features peculiar to this type of construction.

A shear bond failure is shown in Figure 4.9 as it occurs in a laboratory test under two point loading. A major diagonal shear crack develops close to one of the loads. This cause loss of bond in the immediate vicinity of the crack, and this loss of bond generally propagates to the nearby end of the slab, causing bond failure over the entire length $L_{v.}$ From this it is seen that shear bond failure is related to diagonal shear cracking, and this is reflected in the design approach to shear bond.

On the bases of several hundred tests it has been found that the shear bond strength of deck-reinforced composite slabs can be expressed in terms of an equation similar in form to shear strength of conventionally reinforced flexural members:

$$V_n = \left(1.9\sqrt{f_{ck}^{'}} + 2500\rho\frac{V_u d}{M_u}\right)bd \tag{3.15}$$

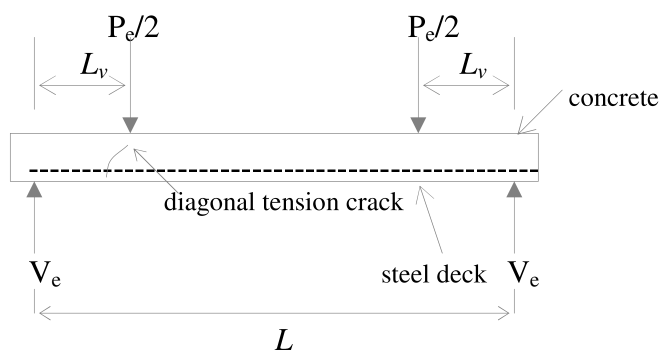



Figure 4.9 Shear-bond failure of composite slab

The constants 1.9 and 2500 in US customary unit, of course, are not applicable to composite decks since they hold for ordinary composite decks since they hold for ordinary bar reinforced beams and slabs. For composite slabs, the values of these constants will depend on the peculiarities of the given deck, e.g. on cross-sectional shape, spacing and depth of ribs, sheet thickness, shape, spacing, and location of shear-transfer devices, e.t.c.

Thus, because of considerable variety of decks being produced by the various manufacturers it is necessary to determine the appropriate constants for each type by tests.

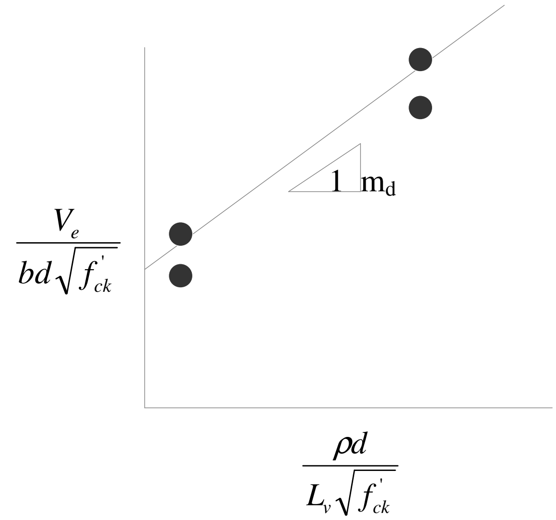



Figure 4.10Typical shear bond failure test result

These tests are standardized and are carried out on full-scale slab specimens loaded in bending by two point loading; i.e., for a test span L two equal loads are applied, each at a distance $L_v$ from the near by support, as shown in Figure 4.9. Thus, in equation 4.15 for $V_n$ one has $M_u/V_u= L_v = L/4$, the shear span. Denoting the unknown constants by $k_d$ and $m_d$, the equation for shear bond strength can be written as,

$$V_n = bd\left(k_d\sqrt{f_{ck}^{'}} + m_d\frac{\rho d}{L_v}\right) \tag{4.16}$$

Where

$V_e$ is the vertical shear at the support at failure per unit width of a simply supported slab in Newton N,

b unit width usually 1000mm

*d* in (mm) is as defined in Section 4.1.2.

$f_{ck}^{'}$ ( N/mm$^2$) is the characteristics cube compressive strength of concrete,

$\rho$ as defined in Section 3.2.1 ,

$m_d$ and $k_d$ are constants to be determined for each type of deck by standardized testing and evaluation procedure.

For purposes of test evaluation, it is convenient to rewrite Equation 4.16 as,

$$\frac{V_e}{bd\sqrt{f_{ck}^{'}}} = k_d + m_d \frac{\rho d}{L_v \sqrt{f_{ck}^{'}}} \tag{4.17}$$

In these terms, the equation represents a straight line, so that when $\frac{V_e}{bd\sqrt{f_{ck}^{'}}}$ is ploted against $\frac{\rho d}{L_v \sqrt{f_{ck}^{'}}}$ , $k_d$ represents the intercept on the $\frac{V_e}{bd\sqrt{f_{ck}^{'}}}$ axis and $m_d$ is the slope of the line. Here $V_e$ is the end reaction in the test at failure. As per BS 5950: Part 4 : 1994, the shear bond capacity $V_u$ with partial safety factor of 1.25 is computed as,

$$V_u = 0.80bd\left(k_d \sqrt{f_{ck}^{'}} + m_d \frac{\rho d}{L_v}\right) \tag{4.18}$$

In this particular study, determination of the the constants $k_d$ and $m_d$, through the procedures described above was found beyond the scope of the study due to the requirement of considerable cost and laboratory equipments not locally found. However, an indirect technique is devised determine the parameters as discussed in the sixth chapter.

*4.2.3.2 Slip resistance by direct shear test*

Even though the shear resistance capacity for the composite slab is usually measured by the procedure just discussed above, effort is also being made for the assessment of shear interlock through the preparation of specimens smaller than the actual slab dimensions.

Makelainen had conducted experiment with small size specimens to measure the shear resistance offered by embossments. Series of seven slip block tests were done to study the effectiveness of different patterns in establishing mechanical composite interlock. Table 4.1 summarizes the main geometrical variables and the experimental measured slip resistance [17].

**Table 4.1: Slip-block tests of Makelainen**

| Notation | Embossment height [mm] | Embossment length [mm] | Embossment distance [mm] | Slip resistance test result $V$ [KN/m] |
|---|---|---|---|---|
| HD5 | 1.43 | 45 | 40 | 22.5 |
| HD6 | 2.31 | 45 | 40 | 45.9 |
| HD07 | 1.62 | 45 | 40 | 33.8 |
| HDV7 | 1.92 | 45 | 40 | 43.9 |
| VD2S | 1.76 | 45 | 40 | 39.0 |
| VD20 | 1.98 | 45 | 40 | 36.8 |

## 4.3 DEFLECTION

### *4.3.1 General*

As per the serviceability limit state of design, the computed deflection at any instant should be below the value recommended in the specific code requirement for proper functionality. During the construction stage, the profiled steel sheet is designed as a load-bearing element to carry the self-weight of the profiled steel sheet, the weight of the wet concrete, and construction load imposed during the construction process. It is therefore important to determine the deflection of the profiled steel sheet due to these loads according to the arrangements of supports. The supports are of two kinds, (i) columns and beams that are designed as permanent elements and, (ii) temporary prop shoring with joist beams, which are used for supplementing the permanent supports. Temporary supports are essential for the use of economical lighter gauge profiled steel sheet to assist the profiled steel sheet against flexural stress above the specified limit and excessive deformation during the construction stage. Even if one of the advantages of using profiled steel sheet for slab construction was said to be the avoidance of false work forms, the formwork used in this case is very small compared to the conventional reinforced concrete slab construction. After the concrete has hardened to the required strength, composite cross-section resistance is developed. This is the stage when all temporary supports are removed and the slab is ready to serve for the intended purpose. The deflection of the composite slab due to service load (Live load, dead load and load due to removal of temporary shoring) should be calculated and checked with the specified deflection limit.

### *4.3.2 Deflection for the Profiled Steel Sheet*

The maximum deflection of the profiled steel sheet occurs when only one span of the continuous supported profiled steel sheet of unit width is loaded and the other spans are unloaded. Figure 4.9 shows the situation of loading for maximum deflection.

The elastic deflection equation derived from continuous beam with uniformly distributed load $w$ of one span as shown in Figure 4.9 below can be used for the computation of maximum deflection occurring at the center of the loaded span as:

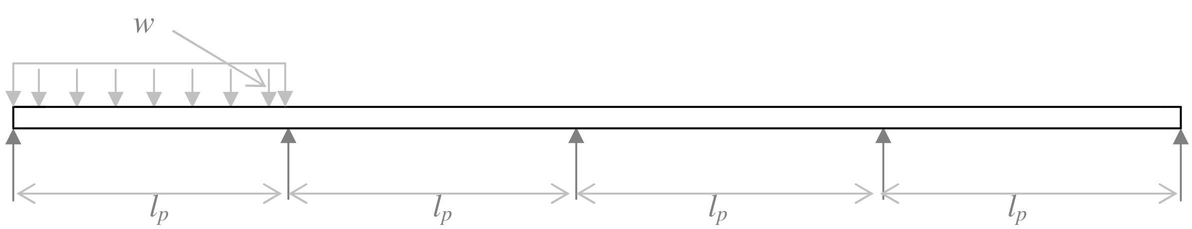



Figure 4.9 Load arrangements for maximum deflection of the profiled steel sheet.

$$\delta = \frac{1}{106}\frac{wl_p^4}{EI_x} \tag{4.16}$$

Where

$\delta$ the maximum deflection at the mid point of the loaded span

$w$ service uniformly distributed load during construction stage (self weight of steel, weight of concrete, and construction load for unit width of slab).

$l_p$ effective span of profiled steel sheets, which is the smallest of:

a) distance between centers of permanent or temporary supports, and

b) clear span between permanent or temporary supports plus overall depth of profiled sheets

$E$ modulus of elasticity of the profiled steel sheet

$I_x$ second moment area of the profiled steel sheet

### *4.3.3 Deflection Limit of the Profiled Steel Sheet for Temporary Loads*

As per the British Standard, BS 5950: Part 4: 1994[3], the deflection of the profiled sheet due to the serviceability loads for the construction stage shall be comprised from the weight of the profiled steel sheets and the wet concrete only. The deflection shall not normally exceed the following[3]:

a) $l_p$/180(but<20mm) when the effects of pounding are not taken into account;

b) $l_p$/130(but<30mm) when the effects of pounding are taken in to account, i.e. the weight of additional concrete due to the deflection of the sheeting is included in the deflection calculation;

These limits may be increased only where it can be shown that greater deflections will not impair the strength or efficiency of the slab. These limits shall be reduced, if necessary, where soffit deflection is considered important, e.g. for service or aesthetic requirements. When the deflection calculated exceeds D/10, where D is the overall depth of the composite slab, the additional weight of concrete due to the deflection of the sheeting should be taken into account in the self-weight of the composite slab.

### 4.3.4 *Deflection of Profiled Steel Sheet and Concrete Composite Slab*

#### *4.3.4.1 Short Term Deflection*

The short-term deflection of the composite slab shall be calculated using serviceability loads $W_{ser}$ excluding the self-weight of the composite slab. The deflection of the profiled steel sheeting due to its own weight and the weight of wet concrete as calculated in section 4.3.3 shall not be included here as new item. For uniformly distributed loading and concentrated load, the following expressions can be used to calculate the short tem deflection [3].

a) For simply supported spans (with nominal reinforcement over intermediate supports)

$$\delta = \frac{5}{384} \frac{W_{ser} L_s^4}{E_c I_p} \tag{4.17}$$

b) For end spans of continuous slabs (with full continuity reinforcement over intermediate supports) of approximately equal span, i.e., within 15% of the maximum span.

$$\delta = \frac{1}{100} \frac{W_{ser} L_s^4}{E_c I_p} \tag{4.18}$$

c) For two span slabs (with full continuity reinforcement over the internal support)

$$\delta = \frac{1}{135} \frac{W_{ser} L_s^4}{E_c I_p} \tag{4.19}$$

d) For concentrated load;

$$\delta = \frac{Pa^2b^2}{3L_s E_c I_p} \tag{4.20}$$

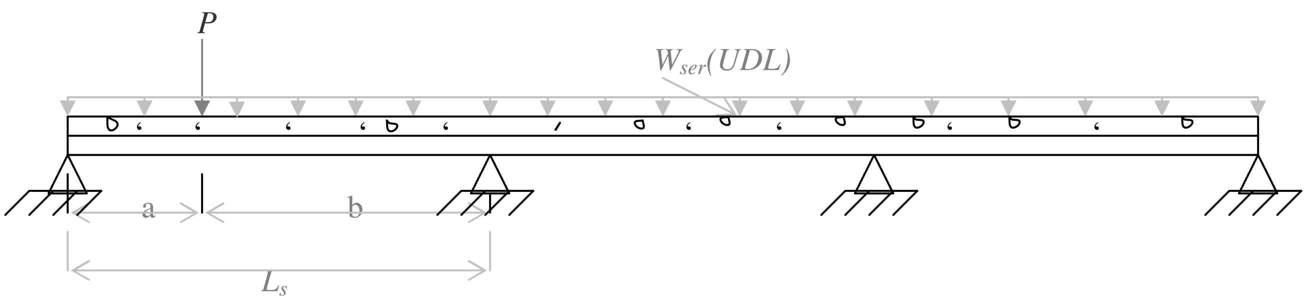



Figure 4.10 Typical longitudinal cross section of the composite slab

Where

| | |
|---|---|
| $\delta$ | deflection at mid point of loaded span |
| $E_c$ | the modulus of elasticity of concrete; |
| $I_p$ | second moment of area defined by Equation (3.14) |
| $L_s$ | effective span of composite slab, which is the smallest of:<br>a) distance between centers of permanent supports, and<br>b) clear span between permanent supports plus effective depth of composite slab |
| $W_{ser}$ | serviceability load |
| $P$ | concentrated load |

*4.3.4.2 Long Term Deflection*

The additional long-term deflection of the composite slab can be obtained by multiplying the immediate deflection caused by the sustained load considered, computed in accordance with section 4.3.4.1, by the factor [5];

$$\left(2-1.2\frac{A_s}{A_{sp}}\right)\geq 0.6 \tag{4.21}$$

Where:

$A_{sp}$ and $A_s$ are as defined in Section 3.1.1 and 3.2.1 respectively

*4.3.3.3 Limiting Values of Deflection*

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

a) Deflection due to the imposed load (loads other than the self-weight of composite slab)

$$\frac{L_s}{350} \quad \textit{Or 20mm, which} \text{ ever is the lesser;}$$

b) Deflection due to the total load and prop removal if prop is used less deflection due to self weight:

$$\frac{L_s}{250}$$

# 5 ANALYSIS AND DESIGN OF PROFILED STEEL SHEET-CONCRETE SLAB

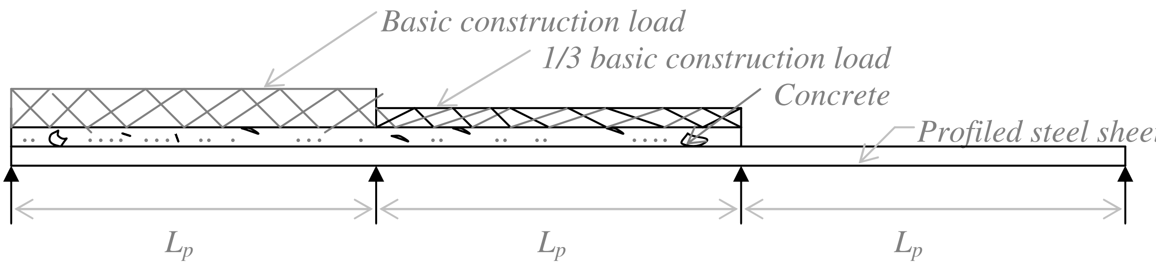
## 5.1 LOADING

All relevant design actions (loads) shall be considered separately and any combination as to cause the most critical effects on the steel profile and on the composite slab as a whole are also considered.

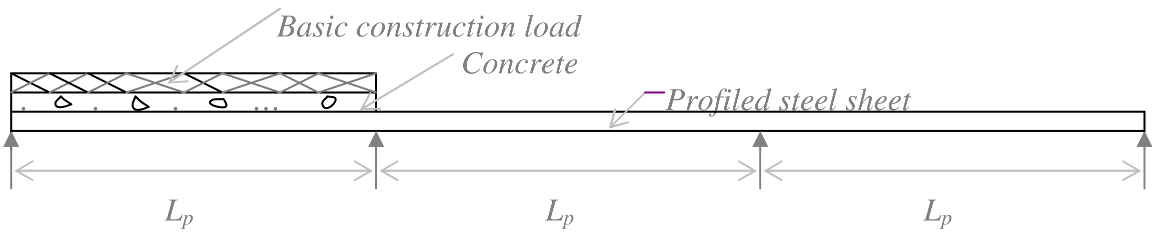
### *5.1.1 Construction Loads*

#### *5.1.1.1 Basic construction loads*

Construction loads shall be considered in addition to the weight of wet concrete and steel profiled sheet during construction stage. For general purpose working areas, the basic construction load on one span of the sheeting shall be taken as not less than 1.5KN/m$^2$ . The other spans shall be taken as either loaded with the weight of the wet concrete slab plus a construction load of one-third of the basic construction load, the situation for maximum negative moment see Figure 5.1(a), or unloaded apart from the self weight of the profiled steel sheets for maximum positive moment see Figure 5.1(b)[3]. Had it not be the requirement of BS-5954 Part-4 for construction loading as discussed above, maximum negative bending moment could have been when adjacent spans are loaded with full construction load. For spans of less than 3m, the basic construction load shall be increased to not less than 4.5/$L_p$ KN/m$^2$, where $L_p$ is the effective span of the profiled steel sheets in meters. Allowance is made within these values for construction operatives, impact and heaping of concrete during placing, hand tools, small items of equipment and materials for immediate use. The minimum values quoted are intended for use in general purpose working area, but will not necessarily be sufficient for excessive impact or heaping of concrete, or pipe lines or pumping loads [3]. Reference shall also be made to section 4.3.3 for possible increased loading due to ponding at the construction stage.




a) Construction load arrangement for maximum negative moment.




b) Construction load arrangement for maximum positive moment.

NOTE-1. The arrangements shown in this figure are for equal values of the effective span $L_p$. In all cases, the most critical arrangement of construction loads should be used.
NOTE-2. For propped construction $L_p$ is the prop spacing.

Figure 5.1. Arrangement of construction loads

*5.1.1.2 Storage Loads*

Where material to be stored temporarily on erected sheeting ( or on a recently formed slab before it is self-supporting ) produce equivalent distributed loads in excess of the basic construction loads, provision should be made in the design for the additional temporary storage loads. For instance, when bricks or cement is stored for sometimes to be used for later construction, such loads are considered as storage loads.

*5.1.2 Self Weight and Imposed Load Assessment*

As per Ethiopian Building Code Standard (EBCS-1, 1995), self weight and imposed loads on the composite slab can be assessed.

*5.1.2.1 Density of construction materials*

Section 2.4 of EBCS-1 1995, provides characteristics values for densities of specific building materials, and stored materials. Material densities required for the assessment of loading for steel profiled sheet and concrete composite slab shall be taken from the specified section of EBCS-1 1995.

*5.1.2.2 Assessment of Self-Weight*

(1) The weight of parts of structures and of non-structural elements shall be determined from the weights of elements of which they are composed.

(2) The characteristics value of the weight of individual elements shall be estimated from nominal densities and nominal dimensions of their constituent materials.

(3) For determining the effect of the self weight due to partition, an equivalent uniformly distributed load may be used.

5.1.2.3 Imposed loads on Buildings and Characteristics Values

***5.1.3 Service Load***

*5.1.4 Ultimate Load*

Imposed loads on buildings are those arising from occupancy and are in accordance with EBCS-1, 1995 as provided in section . furniture and movable

## 5.2 ANALYSIS

### *5.2.1 Analysis for Construction Loads*

For loading condition at the construction stage (*Section 5.1.1.1*) the maximum values for shear forces, bending moments and deflection can be computed as follows.

$$V_c = \frac{17}{30} wL_P \tag{5.1}$$

$$M_c = \frac{10}{107} wL_p^2 \tag{5.2}$$

$$\delta = \frac{1}{106} \frac{wL_p^4}{EI_x} \tag{5.3}$$

Where:

$w$ and $L_p$ are as defined in section 4.3.2

$V_c$ is the maximum shear force at the face of the support.

$M_c$ is maximum bending moment at mid span of loaded profiled steel sheet

$\delta$ is maximum deflection at the mid point of loaded profile sheet

$E$ is modulus of elasticity of profiled steel sheet

$I_x$ is as defined in section 3.1

Any elastic method of continuous beam analysis such as moment distribution method can be applied to determine the maximum negative bending moment and shear force which are expected to be at the middle support (Figure 5.1(a)).

### *5.2.2 Analysis for Service Loads*

Serviceability analysis requires determination of deflection and cracked width of concrete for the service loading suggested in *Section 5.1.3*. Short-term deflection and long-term deflection discussed in *Section 4.3.4* shall be used to assess the deflection of the steel profile concrete composite slab. The short-term deflection equations given are applicable for span ratios not greater than 20%[3], for general case, the deflection equation for each spans of the composite slab, treating as continuous beam can be computed as in *Section 5.2.2.1* below.

#### *5.2.2.1 Deflection*

The actions (moment and shear) at each support of the composite slab can be computed by the analysis procedure discussed in *Section 5.2.3*. The deflection for each of the spans of a continuous one-way composite slab may therefore be determined by the application of strain energy method.

For instance Castigliano's first theorem can be applied as:

$$\delta = \int_0^l \frac{M}{EI}\frac{\partial M}{\partial P}dx \tag{5.4}$$

Detail explanation to account for variable flexural stiffness for one of the spans is discussed in Appendix B.

### *5.2.3 Analysis For Ultimate Limit Load*

Moments and internal shear forces in the composite slab can be determined based on elastic analysis, plastic analysis or non-linear analysis. In this particular study elastic analysis is adopted. For one-way composite slab, where the bending of the slab is considerably in one direction, analysis procedure for continuous beam will be applicable. Bending takes place in the direction of the ridges for the reason that flexural resistance of the composite slab is more along the direction parallel to the ridges than in the transverse direction. In fact, the analysis for steel profile concrete composite slab is usually made for one-way action. Analysis procedure for non-prismatic member (member having variable cross-sectional properties along its length) shall be employed to account for the variation of sectional properties of the composite slab for positive and negative bending moments as modeled in Section 3.3.

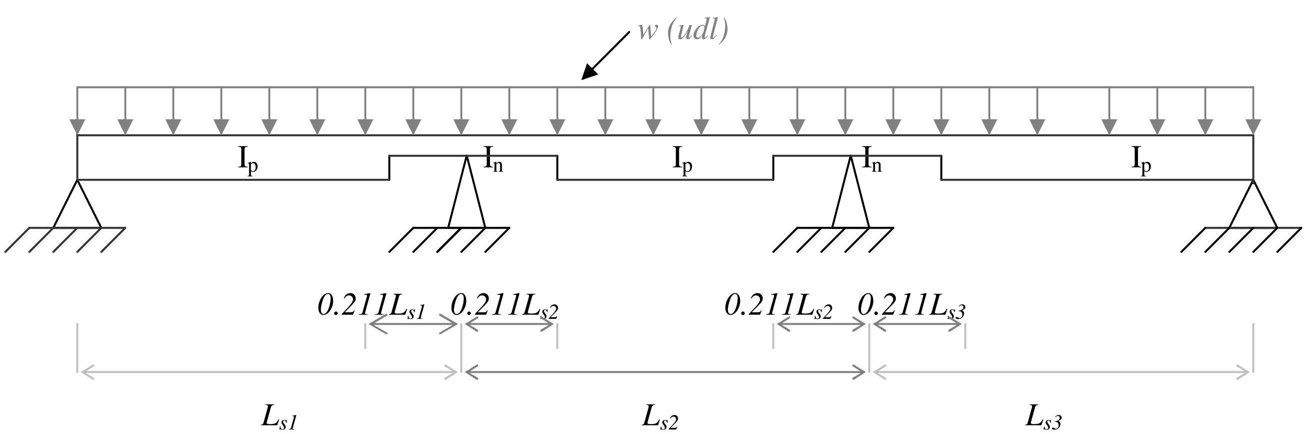



Figure 5.3 Continuous one-way composite slab loaded with uniformly distributed load

One feasible approach for analyzing a structure with non-prismatic members through the aid of stiffness matrix method consists of assuming joints at intermediate points along the length of each non-prismatic member [17]. For the composite slab under consideration, the inflection points are assumed at intermediate joints. This approach will cause the joint stiffness matrix to become very large, and is very difficult for hand manipulation. Fortran Computer program for analyzing continuous beam by the direct stiffness method can be used to determine the actions or effects such as shear force and moment at each of the supports and intermediate joints.

## 5.3 DESIGN

### *5.3.1 Design for Construction Load Action*

The initial step of the design is to choose the type of profiled steel sheet to be used for the construction of the composite slab. Cross-sectional and material properties such as thickness of the steel sheet used for manufacturing the profiled sheet, second moment area, depth of the neutral axis, allowable tensile strength and modulus of elasticity of the profiled steel sheet have to be specified. At the construction stage, the aim of the design is to limit the spacing of the temporary shoring based on flexural resistance and allowable deflection limit of the profiled steel sheet as per the specified material property and construction load arrangement. The choice of the profiled steel sheet may be from the availability of the material and/or economic consideration.

*5.3.1.1 Spacing of shoring based on flexural resistance*

Equating the flexural resistance capacity of profiled steel sheet and applied external moment at the construction stage of loading of equation 4.1 and equation 5.2 respectively, the spacing of shoring *($L_P$)* can be computed as,

$$L_p = \left( \frac{107}{10} \frac{f_{al} I_x}{wy} \right)^{\frac{1}{2}} \tag{5.5}$$

where in this particular situation $L_P$ is assumed to be only center-to-center distance of the shoring.

*5.3.1.2 Spacing of shoring based on allowable deflection*

The deflection equation for continuous beam can be applied to determine the maximum deflection at the middle span during the construction stage.

The allowable deflection of the profiled steel sheet is as specified in *Section 4.3.3*. Equating the deflection equation given in Section 5.2.1 (Equation (5.4)) with the permissible deflection for the case where pounding effect of the wet concrete is not accounted (*Section 4.3.3*) can be calculated as:

$$L_p = \left( \frac{106EI_x}{180w} \right)^{\frac{1}{3}} \tag{5.6}$$

or

$$L_p = \left( \frac{106 * 20 E I_x}{w} \right)^{\frac{1}{4}} \tag{5.7}$$

Thus, the minimum spacing from flexural resistance and permissible deflection requirement will govern the shoring space during construction.

### *5.3.2 Design for Service Load Action*

The sum of the long term and short term deflection of the composite slab computed from Section 4.3.4.1 and Section 4.3.4.2 respectively should be below the limiting values stated in Section 4.3.3.3. Serviceability requirement for crack control is discussed in the sub-section below.

#### *5.3.2.1 Minimum Reinforcement Area for Crack Control*

The required minimum areas of negative reinforcement may be calculated from the relation given by [7]:

$$A_s = \frac{k_c k f_{ct.ef} A_{ct}}{\sigma_s} \tag{5.14}$$

Where

$A_s$ is the area of negative reinforcement

$A_{ct}$ is the are of concrete within tensile zone. The tensile zone is that part of the section which is calculated to be in tension just before formation of the first crack

$\sigma_s$ is the maximum stress permitted in the reinforcement immediately after formation the crack. This may be taken as 100% of the yield strength of the reinforcement, $f_{yk}$ . A lower value may, however, be needed to satisfy the crack width limits

$f_{ct.ef}$ is the tensile strength of the concrete effective at the time when the cracks may first be expected to occure. In many cases, such as where the dominant imposed deformation arises from dissipation of the heat of hydration, this may be within 3-5 days from casting depending on the environmental conditions, the shape of the member and the nature of the form work. When the time of cracking cannot be established with confidence as being less than 28 days, it is suggested that a minimum tensile strength of 3Mpa. be adopted

$k_c$ is a coefficient which takes account of the nature of the stress distribution within the section immediately prior to cracking. The stress distribution is that resulting from the combination of effects of loading and restrained imposed deformation.

= 1.0 for pure tension

= 0.4 for bending without normal compressive force.

$k$ is a coefficient, which allows for the effect of non-uniform self-equilibrating stresses value of *k* for various situations are given below:

(a) tensile stresses due to restraint of intrinsic deformations generally $k$ = 0.8

for rectangular sections when $h \leq 300mm, k = 0.8$

$h > 800mm, k=0.5$

(b) tensile stresses due to restraint of extrinsic deformations $k$=1.0

### *5.3.3 Design for Ultimate Limit Load Stage*

*5.3.3.1 Flexural Resistance Design*

The moment capacity either for under reinforced or over reinforced in positive moment regions shall be calculated assuming rectangular stress blocks for both concrete and profiled steel sheet as per *Section 4.1.2*. The moment capacity in negative moment regions shall be determined by considering the compressive resistance to be provided by the concrete only and the tensile force by the provision of reinforcement bars.

The compression resistance offered by the profiled steel sheet shall be neglected, the reason being explained in Section 3.2.1. Bending of the section around the supports will induce a resultant tensile force $F_{st}$ in the top reinforcement steel, and a resultant compressive force in the concrete $F_{cc}$ at the bottom which acts through the centroid of the effective area of concrete in compression. For design purpose, the composite slab is assumed to be a rectangular cross-section with overall depth ($H$) equal to the depth from center of the profile sheet to the top of the slab. The slab cross-section and the stress block are shown in Figure 5.5 below accompanied with the derivation for flexural reinforcement bar design for negative moment near the supports [15].

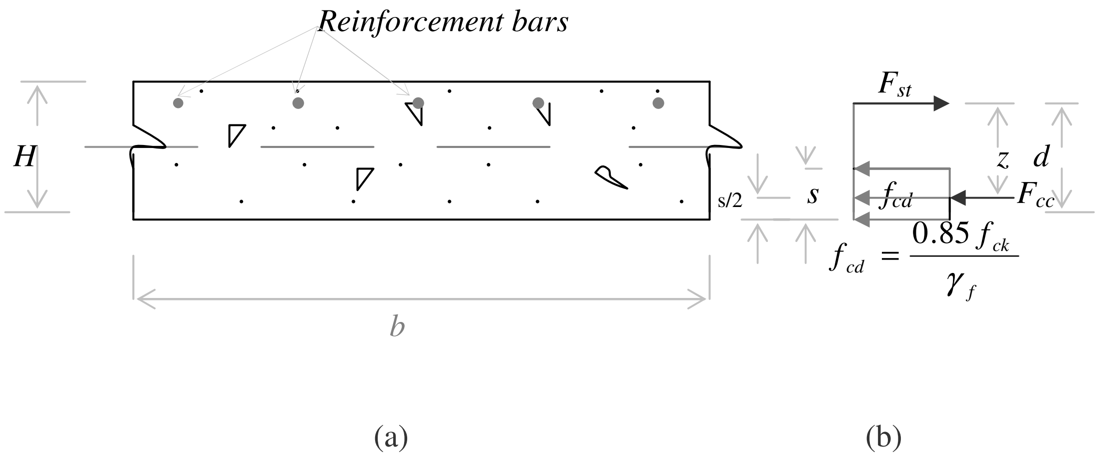



Figure 5.5 (a) assumed cross-section of the slab with negative reinforcement
(b) stress block

From equilibrium, the ultimate design moment $M$ must be balanced by the moment of resistance of the section so that:-

$$M = F_{cc} z = F_{st} z \tag{5.15}$$

Where $z$ is the lever arm between $F_{cc}$ and $F_{st}$.

$$F_{cc} = Stress * Area\ of\ action$$

$$= f_{cd} bs$$

$$F_{cc} = \frac{0.85 f_{ck} bs}{\gamma_f} \tag{5.16}$$

for $\gamma_f = 1.5$

$$F_{cc} = 0.567 f_{ck} bs$$

Therefore,

$$M = F_{cc} z = 0.567 f_{ck} bsz$$

Substituting for $z = d - s/2$

$$M = 1.34 f_{ck} b(d - z)z \tag{5.17}$$

Rearranging and substituting, $K = \dfrac{M}{bd^2 f_{ck}}$

$$\left(\frac{z}{d}\right)^2 - \left(\frac{z}{d}\right) + \frac{K}{1.134} = 0$$

Solving the quadratic equation,

$$z = d\left[0.5 \pm \sqrt{0.25 - \frac{K}{1.134}}\right]$$

Out of the two solution only one is the practical solution, i.e.,

$$z = d\left[0.5 + \sqrt{0.25 - \frac{K}{1.134}}\right] \tag{5.18}$$

The second solution with minus sign before the radical will bring about $s > d$,, which is not practically acceptable solution.

In the resisting moment equation,

$$M = F_{st} z = \left(\frac{f_{sk}}{\gamma_s}\right) A_s z$$

With $\gamma_s = 1.15$

$$A_s = \left(\frac{M}{0.87 f_{sk} z}\right) \tag{5.19}$$

Where $f_{ck}$ is as defined in *Section 4.1.2* and $f_{sk}$ is the characteristics yield strength of the reinforcement bar.

*5.3.3.2 Vertical Shear Resistance*

In the absence of a more accurate calculation, the design resistance of any surface of potential shear failure may be determined from [9].

$$V_{Rd} = 2.5 A_{cv} \eta \tau_{Rd} + A_s f_{sk} / \gamma_s + V_{Pd} \tag{5.20a}$$

or

$$V_{Rd} = 0.2 A_{cv} \eta f_{sk} / \gamma_s + V_{Pd} \tag{5.20b}$$

Which ever is smaller.

Where $\tau_{Rd}$ is the basic shear strength to be taken as $0.25 f_{ctk0.05} / \gamma_o$ ;

$f_{ck}$ and $f_{sk}$ are as defined in Section 5.3.3.1;

$\eta = 1$ for normal weight concrete;

$\eta = 0.3 + 0.7(\rho_c / 25)$ for light weight aggregate concrete of unit weight $\rho_c$ in KN/m$^3$;

$A_{cv}$ is the mean cross-sectional area per unit length of beam of the concrete shear surface under consideration;

$A_s$ is the sum of the cross-sectional area of transverse reinforcement (assumed to be perpendicular to the beam) per unit length of beam crossing the near surface under consideration;

$V_{Pd}$ is the contribution of the sheeting, determined as:

(1) Where the profiled steel sheets are continuous across the top beam, the contribution of profiled steel sheeting with ribs transverse to the beam taken as:

$$V_{Pd} = \frac{A_{s\,p} f_{yk}}{\gamma_s} \tag{5.21a}$$

Where

$V_{Pd}$ is per unit length of the beam for each interaction of the shear surface by the sheeating,

$A_{sp}$ is the cross-sectional area of the profiled steel sheeting per unit length of the beam, and $f_{yk}$ is its yielding strength.

(2) Where the profiled steel sheeting with ribs transverse the beam is discontinuous across the top flange of the steel beam and a stud shear connectors are welded to the sheet beam directly through the profiled steel sheets, the contribution of the steel sheeting shall be taken as:

$$V_{Pd} = \frac{P_{pb,Rd}}{s_1} \qquad \text{but} \qquad \leq \frac{A_{sp} f_{yk}}{\gamma_s} \tag{5.21b}$$

Where $P_{pb,Rd}$ is the design bearing resistance of a headed stud welded through the sheeting, given by Equ. ,and $s_1$ is the longitudinal spacing center to center of the studs.

$$P_{pb,Rd} = k_{\varphi} d_{d_o} t f_{sd} \tag{5.22}$$

Where $k_{\varphi} = 1 + a/d_{d_o} \leq 4.0$

$d_{d_o}$ is the diameter of weld collar which may be taken as 1.1 times the diameter of the shank of the stud;

$a$ is the distance from the center of the stud to the end of the sheeting to be not less than $2d_{d_o}$;

$t$ is the thickness of the sheeting;

$f_{sd}$ is as defined in Section 4.1.2.

# 6 LOCALLY PROFILED STEEL SHEET

## 6.1 INTRODUCTION

As discuss in the previous sections, locally manufactured profiled steel sheet forms have mainly been used for roof cover and cladding purpose only. Attempt has hardly been made by the local construction industry to incorporate profiled steel sheets as a composite system with concrete for the construction of steel deck concrete composite slabs. The results of the analysis and design from the procedures so discussed in the previous sections can be considered reliable, if the mechanical properties of the components of the composite slab independently and the composite slab as a single unit are precisely determined in a standard laboratory test procedures. Tensile strength of the steel sheet and shear bond strength at the interface of profile steel sheet and concrete are the two basic mechanical properties intended to be tested at the capacity of local laboratory facility. Detail of the test procedure, test set up and analysis of the data are incorporated in *Section 6.3*. The tests were conducted in the Faculty of Technology Material Testing Laboratory of Addis Ababa University.

## 6.2 TEST PROGRAM

**Table 6.1 Bar Chart showing the Program for the tests**

| *Item No.* | Description | *August,2002* *Weeks* | | | | *September,2002* *Weeks* | | | | *October,2002* *Weeks* | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1st | 2nd | 3rd | 4th | 1st | 2nd | 3rd | 4th | 1st | 2nd | 3rd | 4th |
| *1* | Tensile Strength Test | | | | | | | | | | | | |
| 1.1 | Test sample preparation | | | | | | | | | | | | |
| 1.2 | Conducting test | | | | | | | | | | | | |
| ***2*** | ***Shear Bond Strength Test*** | | | | | | | | | | | | |
| 2.1 | Test specimens preparation | | | | | | | | | | | | |
| 2.2 | Test set up | | | | | | | | | | | | |
| 2.3 | Concrete casting inside the test specimens and keep for 28 days with proper curing | | | | | | | | | | | | |
| 2.4 | Concrete casting inside concrete cube mold of 150*150*150mm and keep ----<br>-for an age of 14 days (two samples) and<br>-for an age of 28 days (three samples) | | | | | | | | | | | | |
| 2.5 | Conduct shear bond strength test | | | | | | | | | | | | |
| 2.6 | Conduct concrete compressive cube strength test | | | | | | | | | | | | |

## 6.3 PROPERTIES AND TEST FOR LOCAL STEEL SHEET

### *6.3.1 Engineering Properties*

The principal quantities defining the mechanical properties of metal under non repeting loading are as discussed below.

- *Young's Modulus (E)* : The slope of the stress-strain curve in the elastic region where stress is proportional to strain and is defined as:

$$E = \frac{Stress}{Strain} \tag{6.1}$$

- *Poisson's Ratio ( $\nu$ )* : During elastic deformation there are changes in width as well as in length. These are related by Poisson's ratio ( $\nu$ ), given by:

$$\nu = \frac{Width\ Strain}{Length\ Strain} \tag{6.2}$$

- *Nominal Stress*: The nominal or engineering stress is the load at any instant divided by the original cross-sectional area.

- *Nominal Yield Stress*: It is the nominal stress at which elastic deformation begins to change in to plastic deformation.

- *Proof Stress*: For many metals the transition from elastic to plastic deformation is not clearly evident but occurs progressively. Then the stress at which the metal is said to yield, or flow plastically, is defined in terms of a proof stress. This is the stress at which a permanent elongation, or permanent set of a specified percentage of the initial gauge length (usually 0.2%) occurs. The method of determining the proof stress is given in Figure 6.1 below.

- *Ultimate Strength*: When the transition from uniform to non-uniform plastic deformation occurs and necking begins, the load elongation curve goes through a maximum tensile stress ($f_u$) given by:

$$f_u = \frac{Maximum\ Load}{Initial\ Cross\ Sectional\ Area} \tag{6.3}$$

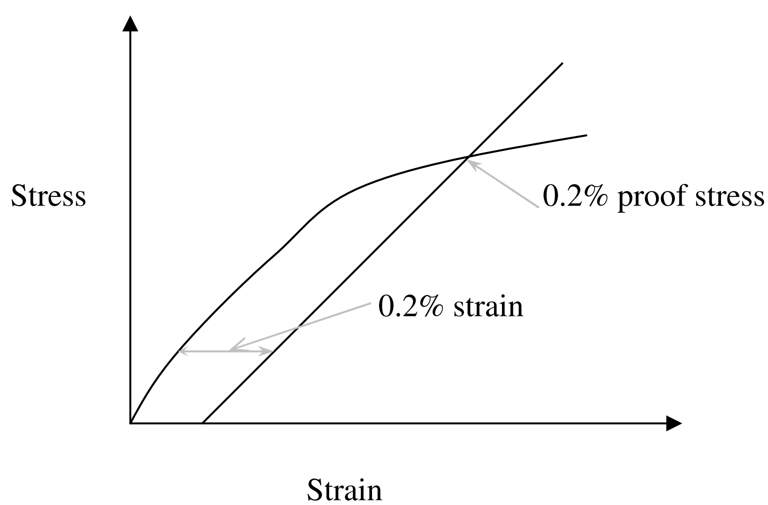



Figure 6.1 Determination of the proof stress from a tensile stress strain curve.

- *Hardness*: It is a measure of resistance to deformation under conditions in which a load indenter is penetrating the surface of the metal under test. The different methods of indentation hardness testing differ in the form of indenter which is forced in to the surface.

(i) The Brinell hardness test used a hardened steel ball as the indenter.

(ii) The vickers hardness test is based up on the use of a square based diamond pyramid of $136^0$ included angle.

(iii) The rock well hardness test involves a diamond cone indenter with $120^0$ included angle and a slightly rounded point.

In all cases the hardness number recorded is related to the ratio of the applied load to the surface area of the indentation formed.

### *6.3.2 Tension Test*

Three test pieces of steel sheet had been brought from Kality Metal Industry to determine some of the basic engineering properties i.e. yield stress, Ultimate strength, percentage elongation and Young's modulus as per the test procedure recommended by the Ethiopian Standard, ES ISO 6892:2002 [20].

For the purposes of Ethiopian Standard, the following definitions apply.

- *Gauge length (L)* : Length of the cylindrical or prismatic portion of the test piece on which elongation shall be measured. In particular a distinct is made between:

  - *Original gauge length* ($L_o$) : Gauge length before application of force.
  - *Final gauge length* ($L_u$) : Gauge length after rupture of the test piece.
  - *Parallel length* ($L_c$) : Parallel portion of the reduced section of the test piece.

- *Elongation*: Increase in the original gauge length ($L_o$) at any moment during the test.
- *Percentage elongation*: Elongation expressed as a percentage of the original gauge length.
- *Test piece width (b)* : Width of the parallel length of a flat test piece.

*6.3.2.1 Types of test piece to be used for thin products; sheets, strips and flats between 0.1mm and 3mm thick [20].*

Generally, the test piece has griped ends, which are wider than the parallel length. The parallel length ($L_c$) shall be connected to the ends by means of transition curves with a radius of at least 20mm. The width of these ends shall be at least 20mm and not more than 40mm. The parallel length can be taken as $L_o + 2b$.

There are two test pieces, with dimensions as given in Table 6.2.

**Table 6.2 Dimensions of Test piece for Local Sheet**

Dimensions in millimeters

| Test Piece type | Width $b$ | Original gauge length $L_o$ | Parallel length $L_c$ | Free length between the grips for parallel sided test piece |
|---|---|---|---|---|
| 1 | $12.5 \pm 1$ | 50 | 75 | 87.5 |
| 2 | $20 \pm 1$ | 80 | 120 | 140 |

The shape and dimensions of the three test pieces used for tensile test of the steel sheet samples are designed to meet the requirements specified for the second type of test piece given in the above table. Figure 6.2 shows the shape and dimensions of the test pieces.

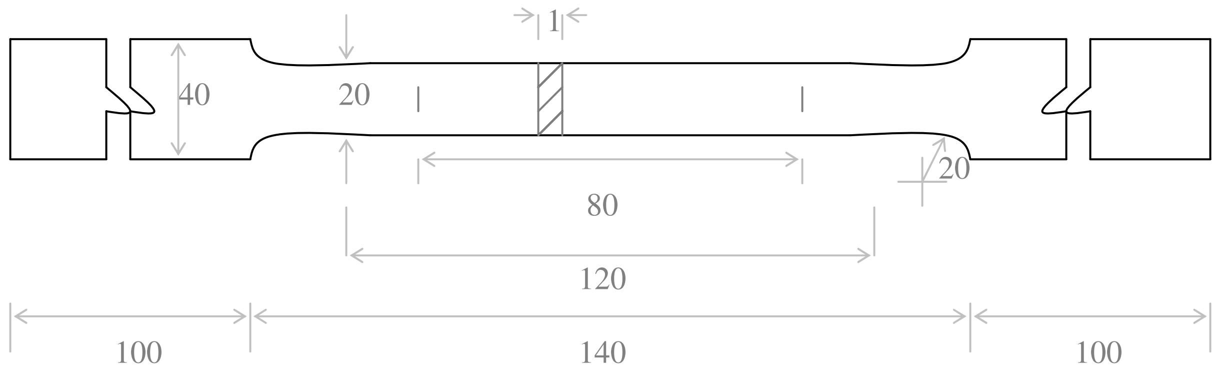



Figure 6.2 Geometrical Shape and dimensions of the test pieces all dimensions in millimeter.

Universal Testing Machine with an accuracy of 0.1KN force and with rate of stressing maintained within the range of 6 to 60 $N/mm^2.s^{-1}$[20] had been in use during the application of the load. The load was applied through the ends of the test piece gripped in the upper and lower clamps of the testing machine. The elongation of the test piece, at an interval of loading equal to 0.5KN, was measured by elongation measuring device with an accuracy of 0.01mm attached to the test sample during the test operation. The record of applied tensile loads and the corresponding elongations gauged are presented in Table C-1 of Appendix C. The stresses and strains calculated as the applied load divided by the original cross sectional area and elongation divided by original length respectively are also incorporated in the same table.

*6.3.2.2 Test Result*

The stress- strain curves drawn from the tensile test result of the three test specimens are shown in Figures 6.3(a),(b) and (c) below.

.

The tensile stresses obtained from the test are all below the minimum recommended values specified in the manuals of Kality Metal factory (Appendix A, Table-2).

It is therefore recommended that appropriate quality control shall be taken with respect to the quality of steel sheets imported from abroad at least to meet the minimum requirement specified in Section A-2 of Appendix A.

The values of Modulus of Elasticity obtained from the tests are in the range of

58 $KN/mm^2$ to 86 $KN/mm^2$, however, as far as steel is concerned Modulus of Elasticity has to be within 200 to 210 $KN/mm^2$. Such discrepancy is expected to be from the error encountered in the laboratory set up of gauging instrument used for measuring elongation. Even though, the universal testing machine, in its brand, had such accurate strain gauging device, at present the system is not operational. Automatic strain gauge measuring instrument suitable for this particular purpose would be expected to give reliable Young's Modulus value.

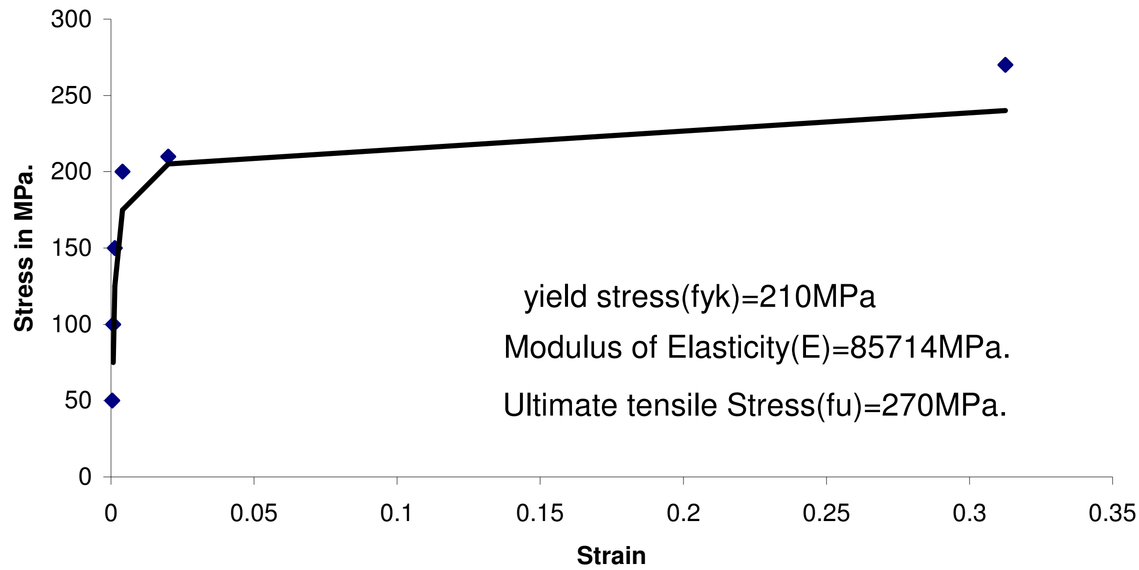



***Figure 6.3(a) Stress Strain Curve for Local Steel Sheet(Specimen-1)***

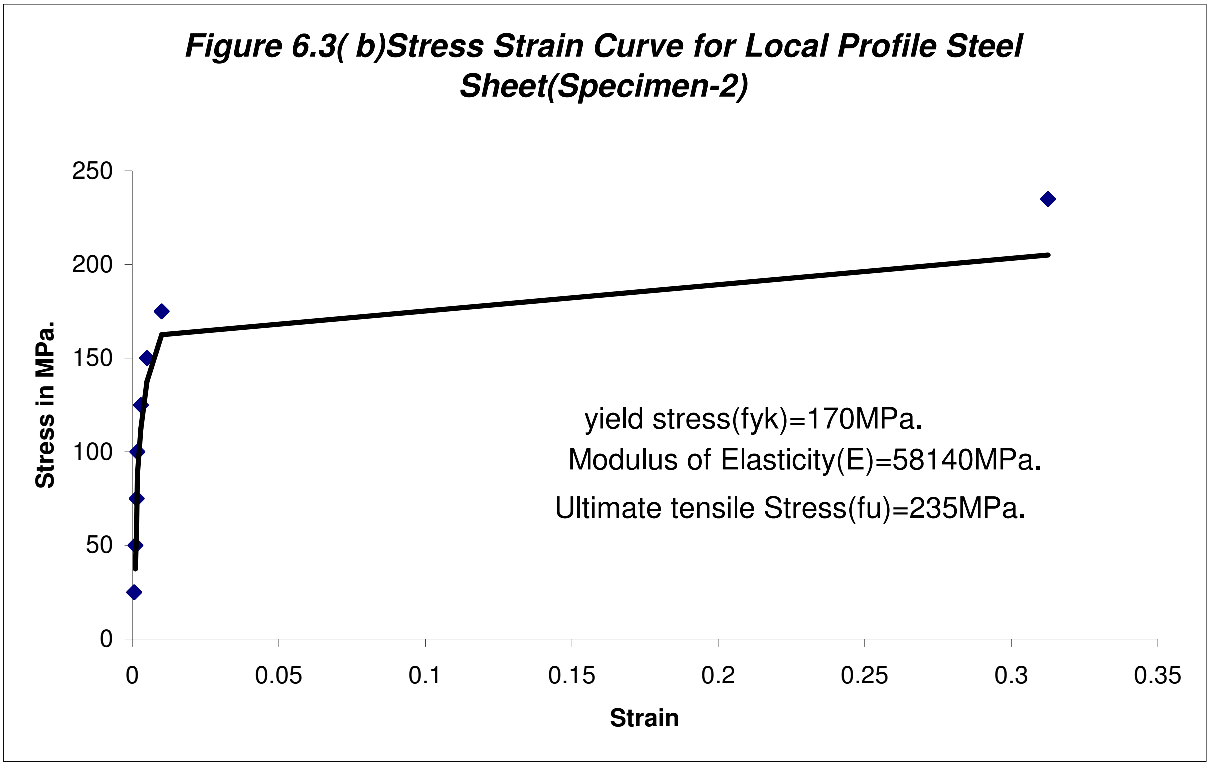


Figure 6.3( b)Stress Strain Curve for Local Profile Steel Sheet(Specimen-2)
250
200
150
100
50
0
Stress in MPa.
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
Strain
yield stress(fyk)=170MPa.
Modulus of Elasticity(E)=58140MPa.
Ultimate tensile Stress(fu)=235MPa.

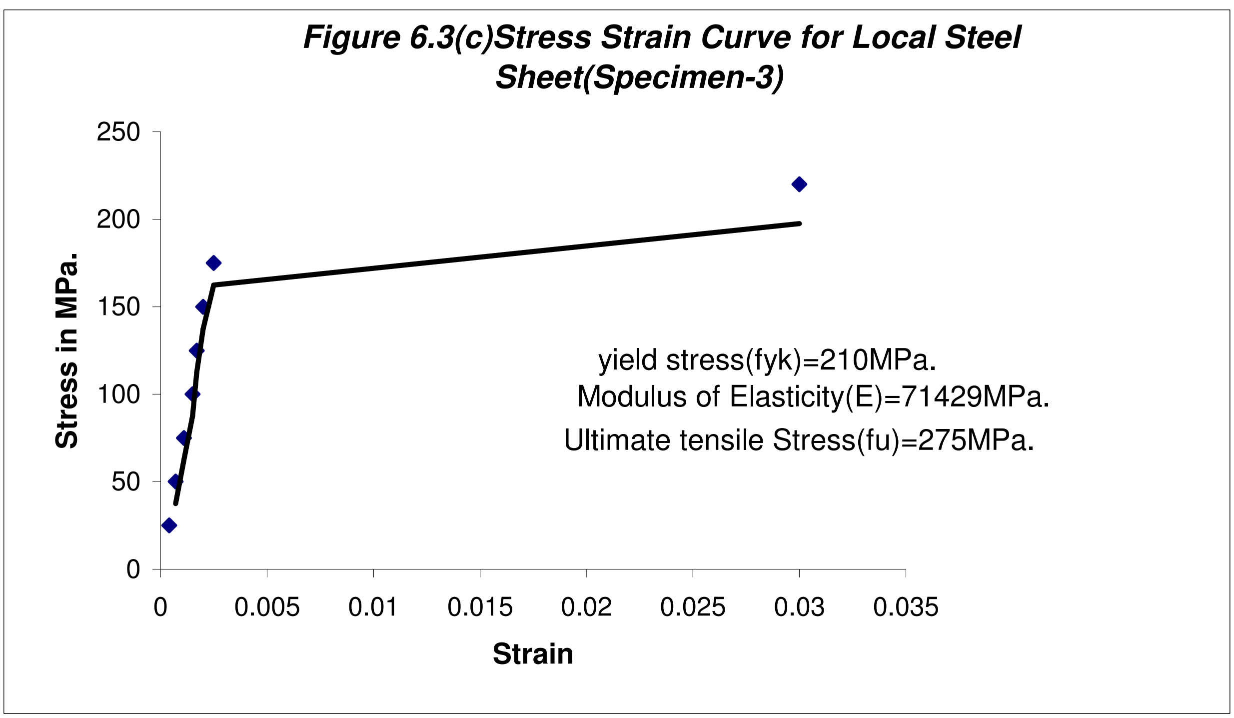


Figure 6.3(c)Stress Strain Curve for Local Steel Sheet(Specimen-3)
250
200
150
100
50
0
Stress in MPa.
0
0.005
0.01
0.015
0.02
0.025
0.03
0.035
Strain
yield stress(fyk)=210MPa.
Modulus of Elasticity(E)=71429MPa.
Ultimate tensile Stress(fu)=275MPa.

### *6.3.3 Shear Bond Strength*

Flexural and vertical shear resistance offered by either the profiled steel sheet or the composite slab can be assessed as per the detailed discussions covered in Sections 4 and 5. However, for longitudinal shear resistance developed at the interface of concrete and profiled steel sheet during the composite action is not as simple as the way bending and vertical shear resistances are determined. As it is noted in *Section 4.2.3* only chemical bond is not usually sufficient to provide adequate longitudinal shear resistance.

Small embossments with various patterns are the usual provision through indenting the flange and or the web of the sheeting to develop the required shear interlock. It was therefore found necessary to assess the contribution of the chemical and mechanical shear resistance offered by the embossments through laboratory test.

#### *6.3.3.1 Specimens for Shear Bond Strength Test*

There were two basic issues in the preparation of the shear bond specimen. The first issue was to search for well-organized local workshop, which was capable of preparing the specimen. Fortunately, Kality Metal Industry, Governmental owned Factory, had agreed to assist the research work through the provision of material and workshop facility to be accounted at the research and development budget of the Industry. The second important issue was to design a specimen for the shear bond test, which will suit the local available laboratory equipment.

Considerable effort was made in compromising the laboratory equipment in the aforementioned Laboratory and the reliability of the model to result in the expected mode of failure. Finally, the specimen was designed to suit the Universal Testing Machine due considering the geometrical parameters and operation mechanisms of the Machine for the type of test commonly known as Push out Test. Although the push-out test was developed in the 1930s, to this day there is no generally accepted or standard procedure for fabricating and testing push out specimens. Consequently, researchers that have investigated shear resistance of embossments have often used similar but yet different procedures for conducting the tests [19].

The specimen for this study is assembled out of two pieces of EGA-700 sheet with metal sheet thickness of 1mm, four anchorage bolts and two side plates of 4mm thick welded together with the profile sheet at the outer face. In addition to the metal components mentioned above, side and bottom shattering forms made up of plywood are also used as formwork.

Embossments, which are the important element of the model are embossed on the top and bottom of the profile sheet by molds of female and male dies fitted in hand operating pressing machine. The size of the embossment is 30*20*4mm, this typical embossment dimension is chosen so that the indentation shall be well accommodated on the ridge and trough of the locally manufactured profile sheets. Figure 6.4 shows the embossments made on one of the profile sheet piece.

Due to various reasons, such as unexpected failure of the specimen before and after the test and detail study requirement of the trend of shear resisting capacity for various distributions of the embossments, it was found essential to prepare as many specimens as possible. The specimens for this particular study are of two sets according to their geometrical dimensions.

The first set consisting of four specimens with external geometrical dimension equal to 330*220*110mm, differs from each other depending on the total number of embossments provided on the two side profile sheets. One of the specimens is free of embossments (plane side profiles) and for the rest three specimens, the number of embossments provided are four for the second, eight for the third and twelve for the fourth specimen.

Likewise, the second set is consisting of four specimens with exterior dimension of 330*220*160mm, differs from each other depending on the number of embossments. The first specimen is being with out embossments, and the rest having six, twelve and eighteen embossments. The photographic view of the eight specimens of the two sets discussed above before and after concrete caste are shown in Figure 6.5(a) and (b).

Figure 6.4 embossments on profile sheet

(a)

(b)

Figure 6.5(a) and (b), push out specimens, before and after concrete pouring respectively

6.3.3.2 Push out Test

*Assumptions*

- The plastic concrete setting, curing, and bond force between the profile sheet and concrete when poured in the vertical direction inside the test specimens are considered to result the same effect compared to the actual concrete on horizontally laid profiled steel sheet.
- Load normal to the composite slab is assumed to be applied through two pairs of side anchorage bolts.

The plan and sectional view of the specimen model are shown in Figure 6.6. From the sectional view, it is seen that the side profile sheets are a bit higher than the concrete block in between the profiles. On the other hand the concrete block sandwiched between the profile sheets are at lower level than the bottom of the sheets. Two equal and opposite forces are applied at the top of the profile sheets and at the bottom of the concrete block. The test was carried  after the concrete had attained its full strength at an age of 28 days.

Each specimen was then placed in the universal testing machine at the lower bearing pad and lifted up till the specimen was in contact with the top bearing pad. Shear load was applied with the testing machine to the specimen in the rate of loading ranging from 0.4 KN/sec. to 0.6KN/sec. [19]. Displacement control device with an accuracy of 0.01mm was also attached to the specimens to measure the slip occurred during the application of the shear load. The test set up and simultaneous recording of shear load and slip during one of the push out tests are shown in Figure 6.7.

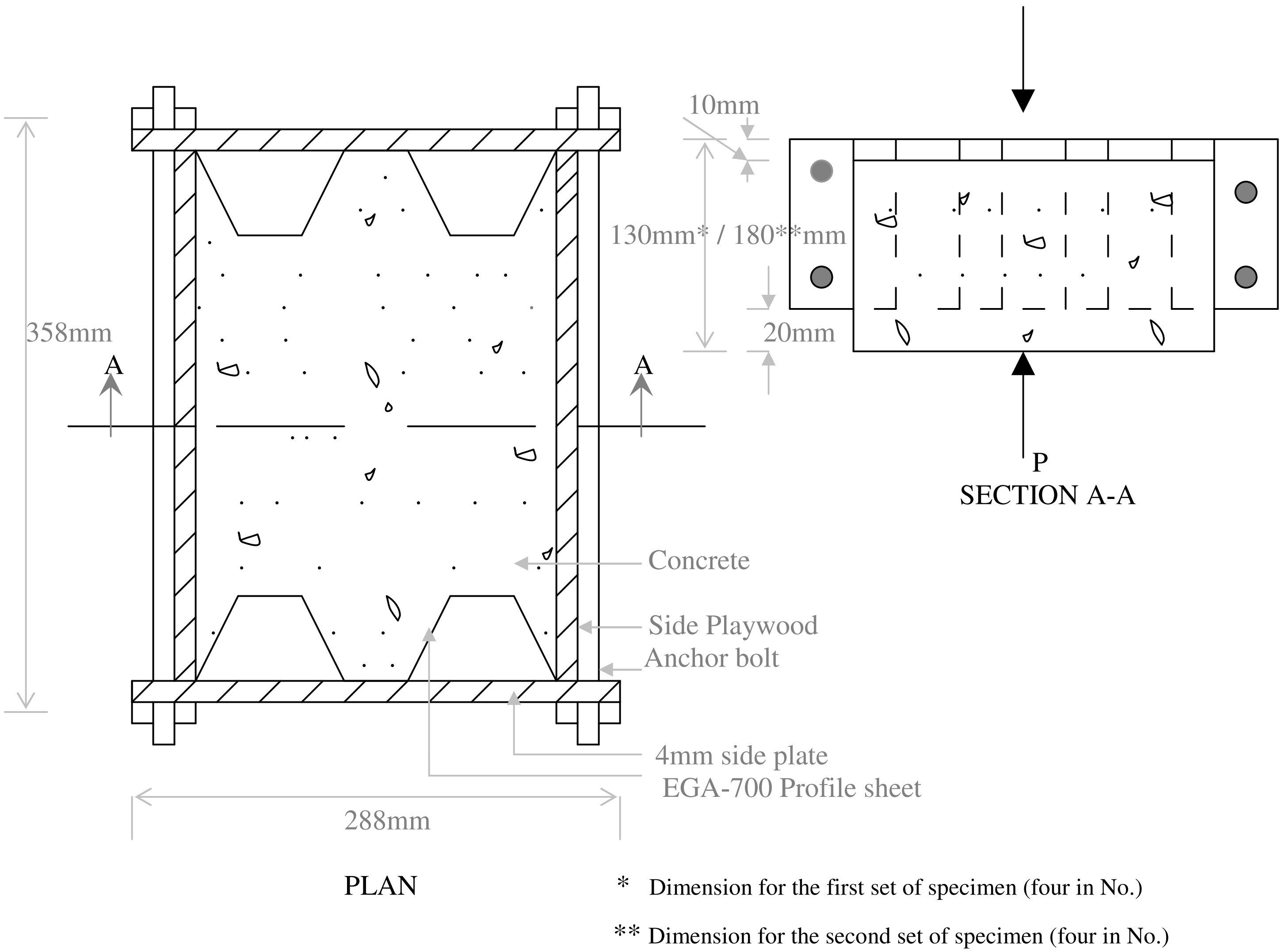



Figure 6.6 Specimens Model for Push Out Test

Table C-2 of Appendix C contains geometrical data for the specimens, applied shear load and corresponding slip (displacement) gauged during the tests. Shear resistance computed from the shear load and concrete–profile sheet contact area are also incorporated in the same table. The shear resistance of embossment is governed by either compressive strength of concrete or tensile strength of profile sheet. The shear strength increases with increasing concrete compressive strength up to the maximum value equal to the tensile strength of the profile sheet]. Failure in push out tests would be controlled by concrete failure for relatively low values of concrete compressive strength and steel failure for relatively high values of concrete strength [19]. It is therefore found essential to incorporate the compressive strength of the concrete used for the test. Concrete cube samples taken from the mix batches for the push out tests and the concrete cube compressive strength obtained for an age of 14 and 28 days are shown in Table C-3 of Appendix C.

Figure 6.7 Test operation (during data recording)

### 6.3.3.3 Stress Vs Slip Relation

The relation ship between strength and slip for push out test for headed stud shear connector results similar to those shown in Figure 6.8 [19]. The test is generally run monolithically, but an unloading curve is shown to illustrate the unloading behavior after significant slip has occurred as in the case of a major overload event [19]. Generally the basic relationship between strength and slip has an exponential form.

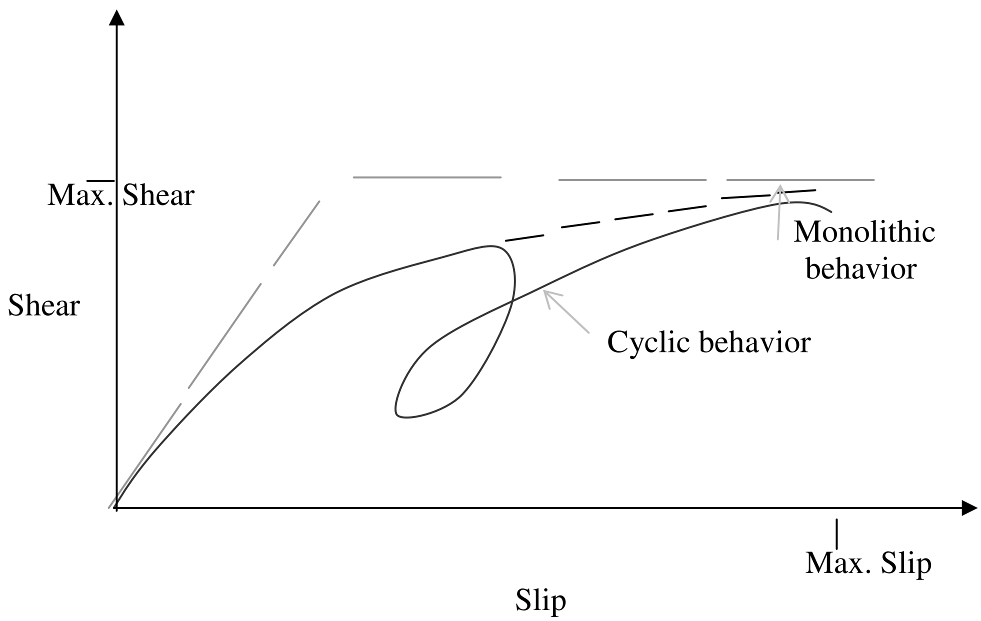


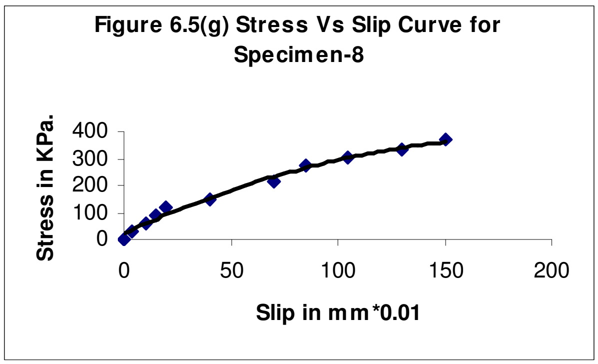

*Figure 6.8 Shear – Slip Curve for headed stud connector*

On the other hand, the Shear Vs Slip graphs resulted from the specimen models so designed for shear bond strength test detailed in the previous sections, showed somehow different distribution compared to the Shear – Slip curve for headed stud connector of Figure 6.5. Some of the differences are:

- The relation of Shear and Slip better fit polynomial distribution as shown unlike the exponential relation stated for push out test for headed stud connector.

- Cyclic behavior experienced for headed stud shear connector is not seen for the particular shear model test under consideration.
- The mode of failure observed for all push out test specimens used for this study was brittle in nature.

A typical shear verses slip graph drawn from the test result of specimen-8 is shown in Figure 6.9.




*6.3.3.4 Shear resistance offered by a single embossment*

The shear resistance offered by chemical bond and mechanical inter lock due to the embossments as obtained from the push out test result, from Table C-2 are summarized in Table 6.3 below. The distribution of embossments used in the test are projected for per unit square meter of profile sheet which is calculated as the number of embossments within the side profile sheet of the specimen divided by the area of the side profile sheet of the same specimen in meter square. The average resistance capacity of a single embossment is also computed from the shear resistance divided by the number of embossments as shown in the last column of Table 6.3.

**Table 6.3 Shear resistances for various distributions of embossments computed from the test data of the push-out test**

| No. Embossments Per unit Square meter | Shear Resistance in( Kpa) | | | Resistance by A single embossment in (KN) |
|---|---|---|---|---|
| | 1st Set of Specimen | 2nd Set of Specimen | Average of 1st and 2nd Set (*q*) | |
| 0 | 90.91 | 69.7 | 80.31 | - |
| 91 | 143.18 | 163.64 | 153.41 | 1.7 |
| 182 | 386.36 | 233.33 | 309.85 | 1.7 |
| 273 | 420.45 | 366.67 | 393.59 | 1.44 |

From the experimental result of embossments shear resistance capacity obtained by Makelainen as discussed in *Section 4.2.3.2* can also be used to determine the resisting capacity of a single embossment as shown in Table 6.4 . The shear resisting capacity is determined for one raw of embossments and as a result the number of embossments for each specimen, for one-meter length and 40mm spacing, is calculated out to be 25.

**Table 6.4 Makelainen Test result of Embossments Shear Resistance**

| Notation | Shear Resistance of embossments $V_{TEST}$(KN/m) | No. Embossments per meter length | Resistance by A single embossment in (KN) |
|---|---|---|---|
| HD5 | 22.5 | 25 | 0.9 |
| HD6 | 45.9 | 25 | 1.84 |
| HD07 | 33.8 | 25 | 1.35 |
| HDV7 | 43.9 | 25 | 1.76 |
| HD2S | 39.0 | 25 | 1.56 |
| HD20 | 36.8 | 25 | 1.47 |

Since the geometrical dimensions of the embossments used for the two tests are not identical, it is not possible to make direct comparison between test results of Table 6.3 and Table 6.4. Hence, the geometrical variables of each embossment, important parameters for shear interlock, used in the tests can be compared as in Table 4.5 below to relate the shear resistance of a single embossment obtained from the two tests. The lateral dimensions in the direction of shear flow are the important geometrical dimensions used for the comparison.

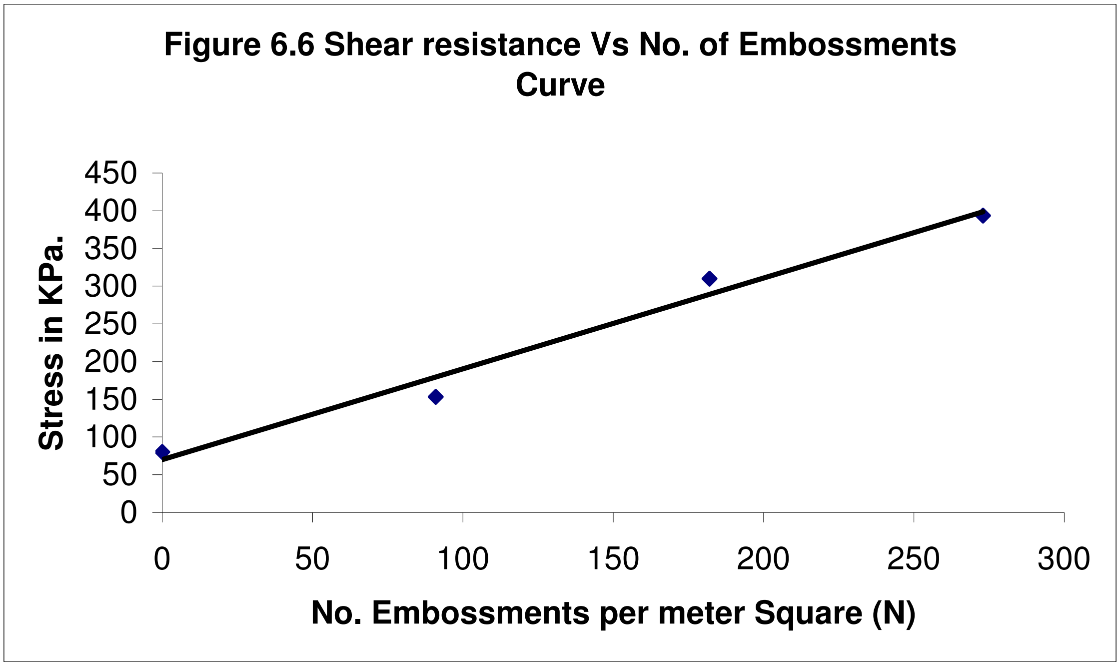
**Table 6.5 Lateral dimensions and embossment resistance for embossments used in the Two tests ( Makelainen Test and Push out test in this study)**

| Makelainen Test | | | | Ratio of Lateral dimension of embossments from Makelainen (col.(c)) to lateral dimension for this study($30*4mm^2$) | Ratio of embossment resistance from Makelainen Test to Average embossment resistance for this study (1.7KN) |
|---|---|---|---|---|---|
| Notation | Embossment height in (mm) (a) | Embossment lateral dimension in (mm) (b) | col. (a)*col.(b) (c) | | |
| HD5 | 1.43 | 45 | 64.35 | 0.54 | 0.53 |
| HD6 | 2.31 | 45 | 103.95 | 0.87 | 1.08 |
| HD07 | 1.62 | 52 | 84.24 | 0.70 | 0.79 |
| HDV7 | 1.92 | 52 | 99.84 | 0.83 | 1.03 |
| HD2S | 1.76 | 45 | 79.2 | 0.66 | 0.92 |
| HD20 | 1.98 | 45 | 89.1 | 0.743 | 0.86 |

From comparison of the last two columns of the above Table 6.5, the trend of ratio of embossment resistance capacity is generally greater than the corresponding geometrical ratios. Hence, it can be concluded that embossment dimensions used by Makelainen are more efficient than embossment dimensions used in this study with respect to shear interlock.

*6.3.3.5 Relation between Shear Resistance and Embossment distribution*

A plot of graph between the average shear resistance and number of embossments per meter square from the summery of the test result as in Table 6.3 exhibit a linear relation as shown in Figure 6.6 below.




The linear equation for the  graph in Figure 6.6 can be expressed as:

$$\tau_{re} = C_e + K_e N \tag{6.4}$$

Where $\tau_{re}$ is shear resistance for a given embossment distribution for the type of embossment used in the test in KPa,

$C_e$ is a constant which is the intersection of the graph at the  ordinate for the particular test equal to 80.3,

$K_e$ is the slope of the graph equal to 1.15,

$N$ is the number of embossments per unit square meter.

Equation (6.1) can be written as,

$$\tau_{re} = 80.3 + 1.15N \tag{6.5}$$

*6.3.3.6 Parametric Constant determination ($k_d$ and $m_d$) from push out test result*

The parametric constants ( $k_d$ and $m_d$ ) are usually determined from the proto-type test procedure just discussed in section 4.2.3.1. However, in this study unlike the usual procedure, the shear resistance obtained from the push out test and equating shear flow equation (Equation 4.14) to the shear resistance equation for parametric test (Equation 4.18) is the technique adapted to determined shear parameters for the profile sheets used in the push out test as discussed below.

The parametric test requires at least two composite slabs with different span length (L) and over all slab depth (D) to determent the two basic parameters. Likewise, in this particular technique, two slabs of different span length and depth within the practical geometrical dimensions are assumed too.

- Assumed Geometrical dimensions of the composite slabs

  - Composite slab 1

    $L_1$=5000mm

    $D_1$=150mm

  - Composite Slab 2

    $L_2$=3000mm

$D_2$=130mm

- Sectional Properties

$I_x$ (for the profile sheet used in the test EGA-700)

per meter width as defined in section 3.1 = 260378.4mm$^4$

$I_{p1}$ ( as per equation 3.7,3.11, and 3.14 )

per meter width of slab for slab no.1 =178,337,804.9mm$^4$

$Q_1$ ( as defined in section 4.2.2 ) =1,284,285.7mm$^3$

per meter width of slab for slab no.1

$I_{p2}$ ( as per equation 3.7,3.11, and 3.14 )

per meter width of slab for slab no.2 = 117,548,474.1mm$^4$

$Q_2$ ( as defined in section 4.2.2 )

per meter width of slab for slab no.2 = 924,190.5mm$^3$

Shear flow equation for positive moment is given as ( Equation 4.14),

$$F_s = \frac{MQ}{I}$$

Shear resistance ( $F_r$ ) for the shear span from the resistance obtained from the test data,

$$F_r = \frac{qL_v b}{1.25} \tag{6.6}$$

Where $q$ is the shear resistance obtained for push out test as in Table 6.3,

$L_v$ is the shear span,

$b$ is unit width of slab

1.25 in the equation is partial safety factor

Equating the shear flow and shear resistance and substituting $M = V_u L_v$,

$$\frac{V_u L_v Q}{I} = qL_v b$$

=> $$V_u = \frac{qIb}{1.25Q} \tag{6.7}$$

Where, $V_u$ is the ultimate shear at the support for two point loads at a distance of the shear span from the support of simply supported composite slab as shown in Figure 4.9.

Shear resistance equation (Equation 4.18) from parametric test is give as,

$$V_u = \frac{bd}{1.25}\left(k_d \sqrt{f'_{ck}} + m_d \frac{\rho d}{L_v}\right) \tag{4.18}$$

Equating Equations (6.7) and (4.16),

$$\frac{qI}{Q} = d\left(k_d\sqrt{f_{ck}^{'}} + m_d\frac{\rho d}{L_v}\right) \tag{4.19}$$

From Equation (4.19) two equation can be developed for the assumed composite slabs, slab no. 1 and slab no.2. The equations after simplification are as shown below,

$$139q = 706k_d + 0.1533m_d \tag{4.20}$$

$$127q = 599k_d + 0.2174m_d \tag{4.21}$$

Where,

$f_{ck}^{'} = 28.6$ N/mm$^2$ from averaging the concrete cube compressive strength obtained at the age of 28 days ( Appendix C, Table C ).

Solving the above equations for $k_{d}$ and $m_d$ in terms of $q$ can be given as,

$$m_d = 107q$$

$$k_d = 0.1737q$$

The coefficients are thus determined for various average shear resistance ($q$) recorded from the push out test result. Table 6.6 below summaries the values of the coefficients.

**Table 6.6 Values of shear parameters for the profile sheets used in the push out test.**

| Shear resistance from push out test in N/mm$^2$ (Mpa.) $*10^{-3}$ | $k_d$ | $m_d$ |
|---|---|---|
| 80.31 | 0.014 | 8.6 |
| 153.41 | 0.027 | 16.4 |
| 309.85 | 0.054 | 33.2 |
| 393.59 | 0.070 | 42.11 |

A manufacturer of steel deck has published $k_d$ and $m_d$ values as 0.0202 and 121 respectively for steel deck property detailed below[5].

Deck height (h) = 38mm

Gauge of the steel deck = 0.91

$I_x$ = 300,441.17mm$^4$

$f_{ck}^{'}$ = 26 N/mm$^2$

$f_{yk}$ =231N/mm$^2$

Even if it is not possible to compare directly the values of the coefficients obtained from the test and the values provided by the manufacturer for the deck specified above, it is possible to draw the following comment.

The coefficient $k_d$ provided by the manufacturer do have similarity with the test result laying between the first two profile sheets. However, the value of $m_d$, which is the function of the roughness nature of the surface at the interface of concrete and profile sheet, provided by the manufacturer is too high compared with the values obtained from the test result. The reason for this variation may be due to the interlocking mechanism used by the manufacturer is much better than the interlocking technique used for this particular study (embossments only).

Even if the method so far discussed for derivation of the shear parameters looks sound with respect to its simplicity and reasonable acceptable values compared to the values obtained from the proto type test, it is advised here to confirm the validity of the procedure through proto type test result for future study.

## 6.4 DESIGN EXAMPLE

### *6.4.1 Office Building Floor Slab*

Slab floor of an office building, the plan drawing shown in Figure 6.7, is used for the demonstration of design procedure discussed in the previous sections and also the same floor slab is employed for cost comparison between profile steel sheet concrete composite slab construction and conventional reinforced slab construction at the end of the example. Anything associated with cost is calculated based on the current material, labor, and rental unit prices used for construction cost estimation of building in Addis Ababa City[21].

- Design Data

- *Profiled Steel Sheet*

EGA-700, Kality metal factory sheet profile product with sheet metal thickness of 1mm, is used as decking.

Yield strength , $f_y$ = 280MPa.

Allowable tensile strength, $f_{al}$=160MPa.

Modulus of Elasticity, $E_{sp}$=210KN/mm$^2$

Density, 7850 Kg/m$^3$

From Equation (3.1), $y_c$ =16.02mm

From Equation (3.2), $I_x$ = 260378.4mm$^4$ (for per meter width of profile steel sheet)

- *Reinforcement Bar*

Yield Strength, $f_{ys}$=300MPa.

Figure 6.7 Floor Plan of Office Building.

Modulus of Elasticity, $E_s$=210KN/mm$^2$

➢ *Concrete*

Density of Wet Concrete = 2400Kg/m$^3$

Density of dry Concrete = 2350Kg/m$^3$

Concrete Cylindrical Characteristics Strength, $f_{ck}$=25MPa.

Modulus of Elasticity, $E_c = 9.5(8 + f_{ck})^{1/3}$ =30.5GPa. [3]

❑ Design For Construction Load

Considering overall depth of the composite slab ($D$) equal to 150mm, effective depth of the slab, $d$ ( i.e from neutral axis of the profile sheet up to top of slab) is calculated as,

$d$ = 150 – (34 – 16.02) = 132.02mm.

➢ Weight of wet concrete and steel deck for unit width of deck,

= 1*0.13202*2400*9.81 + (1/.70)*10$^{-2}$*7850*9.81

= 3.22KN/m

➢ Construction Load =1.5KN/m

Total Load = 4.72KN/m

➢ Moment carrying capacity ($M_c$) of the profiled sheet for $f_{al}$=160MPa (allowable stress for profile sheet)

$$M_c = \frac{f_{al} I_x}{y_t} = \frac{f_{al} I_x}{h - y_c} = \frac{160 * 260378.4}{(34 - 16.02)} = 2.32 KNm$$

Equation 5.11, 5.12, and 5.13 are used for deterring the spacing of props depending on flexural and deflection requirements.

- Spacing of props based on flexural requirement,

$$L_p = \sqrt{\frac{107 M_c}{10w}} = \sqrt{\frac{107 * 2.32}{10 * 4.72}} = 2.3m$$

- Spacing of props based on deflection requirement,

$$L_p = \left( \frac{106 * EI_x}{180w} \right)^{\frac{1}{3}}$$

$$= \left( \frac{106 * 210 * 260378.4}{180 * 4.65/1000} \right)^{\frac{1}{3}} = 1.91\text{m}$$

or

$$L_p = \left( \frac{106 * 20 * 260378.4 * 210}{4.65/1000} \right)^{\frac{1}{4}} = 2.23\text{m}$$

The minimum prop spacing is obtained from deflection requirement.

Design prop spacing is thus equal to 1.9m.

- Design for Service Load

Equation 4.19 is used for assessing the short-term deflection due to imposed load

$$\delta = \frac{1}{100}\frac{W_{ser}L_s^4}{E_c I_p}$$

The sectional property $I_p$ is calculated from Equations 3.7, 3.11 and 3.14 and is equal to

$$I_p = \frac{I_{cp} + I_{up}}{2} = \frac{107{,}593{,}141.5 + 249{,}082468.2}{2} = 178{,}337{,}804.9mm^4$$

- Imposed load for an office building as in in section of EBCS-1,1995 is 3KN/m$^2$.

$$\delta = \frac{1}{100}\frac{3 * 5000^4}{30500 * 178{,}337{,}804.9} = 3.5mm$$

Long term deflection can be computed from Equation (2.21) assuming no negative reinforcement at the mid span is 2*4.42mm. Total deflection ($\delta_t$), which is the sum of long term and short-term deflections are then becomes,

$$\delta_t = 3.5 + 2 * 3.5 = 10.5mm$$

Allowable deflection limit as given in Section 4.3.3.3

$$\frac{L_s}{350} or 20mm$$

$$\frac{5000}{350} = 14.3mm \quad > 10.5 \quad ok!$$

- Analysis for Ultimate Limit Load

- Loading

- Dead Load ($G_k$) = Weight of concrete slab, profile sheet, 30mm thick mortar and

25mm thick, Terrazzo tile flooring.

$= (0.150-0.034/2)*23.5 + 1/0.7*1*10^{-3}*78.5 + 0.03*23 + 0.025*23$

$= 4.48KN/m$

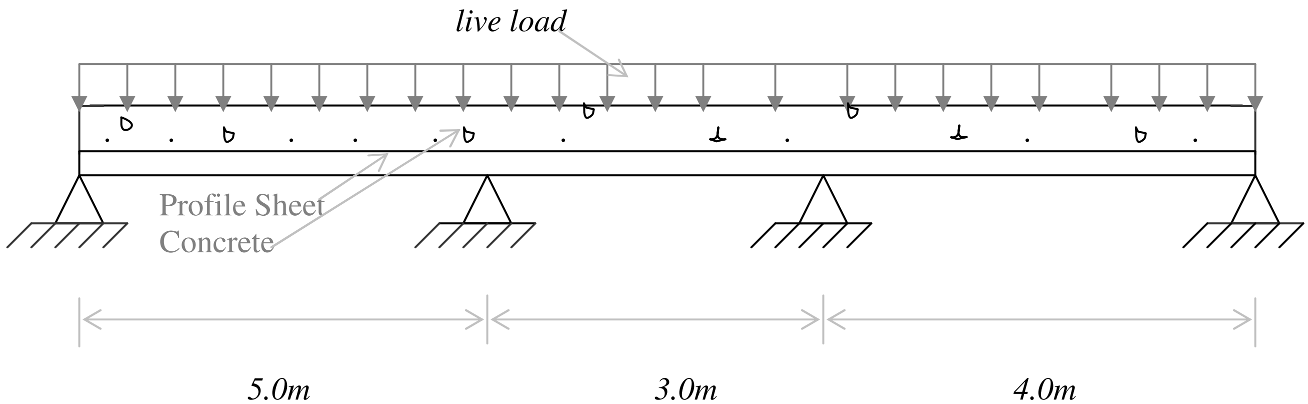
- Live Load ($q_k$) = 3KN/m

- Factor dead load = 1.3*$G_k$ = 5.58KN/m

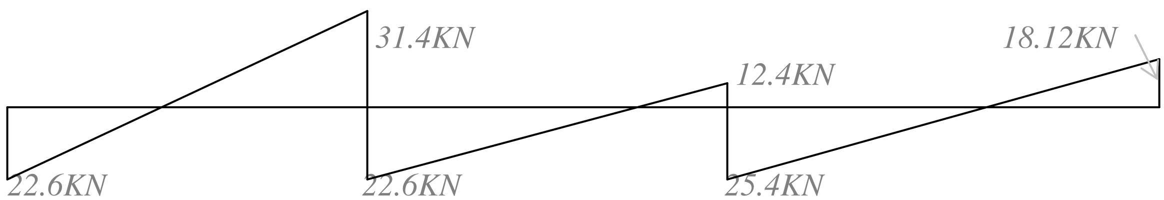
- Factored live load = 1.6*$G_k$= 4.8KN/m

➢ Loading Conditions, Moment and Shear Envelops

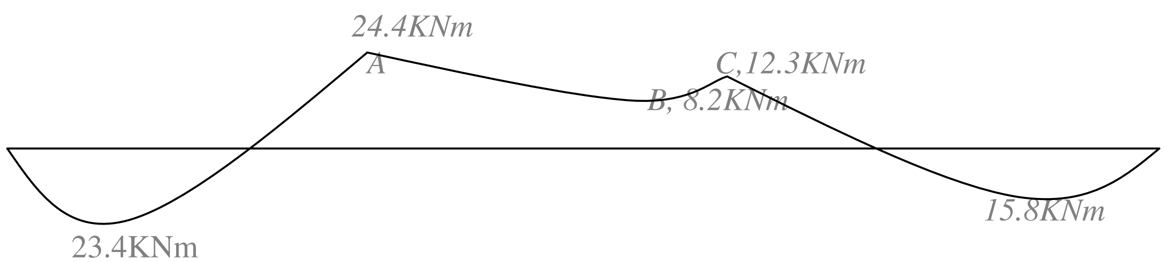
Elastic analysis procedure as discussed in section 5.2.3 were used for various possible positions of the live load to drive the composite bending moment and shear force diagram for Section A-A and Section B-B of Figure 6.7. The composite bending moments and shear forces for the two sections are shown in Figure 6.8 and 6.9 respectively.




(a) Composite slab with live load

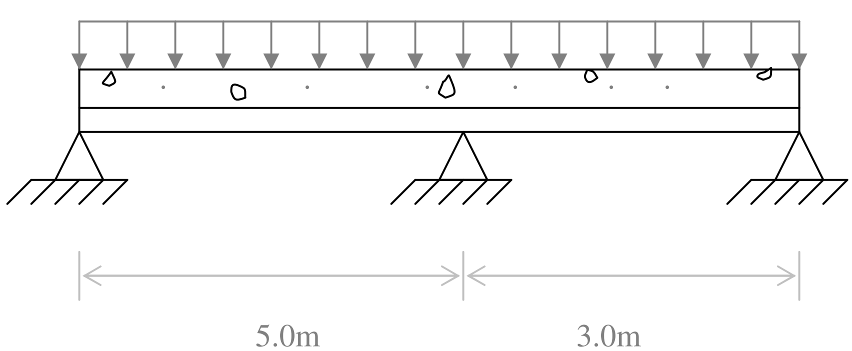



(b) Composite Shear Force Diagram

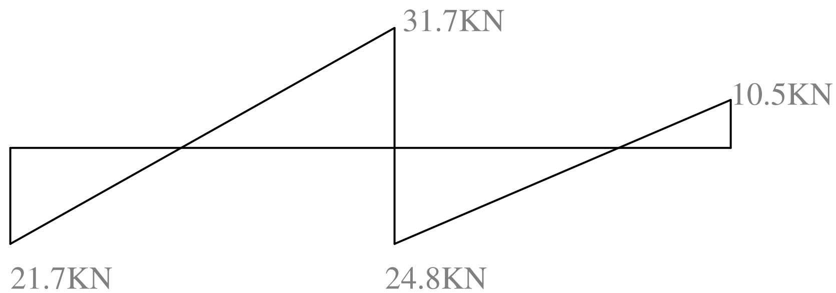



( c) Composite Bending Moment Diagram

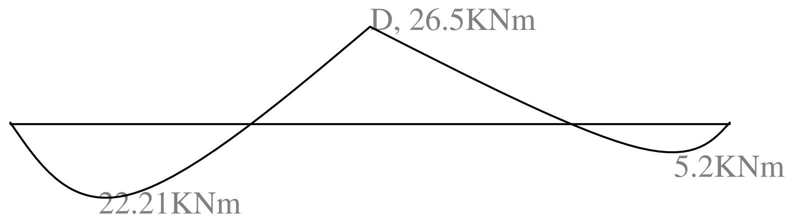
Figure 6.8 Composite slab at Section A-A




(a) Composite slab with live load




(b) Composite Shear Force Diagram




( c) Composite Bending Moment Diagram

Figure 6.9 Composite slab at Section B-B

- Design for Flexure

- Positive Reinforcement

As it is discussed in the previous sections, one of the main purposes of the profile steel decking is to provide positive moment resistance. Equations developed in Section 4.1.2 for flexural resistance of profiled steel sheet and concrete composite slab can be used for checking the moment resisting capacity of the profiled steel sheeting.

From Equation (4.3), $\rho_b$ can be calculated as;

$$\rho_b = \beta_1 \frac{f_{cd}}{f_{yd}} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_y} \frac{D-h}{d} \tag{4.3}$$

$\beta_1 = 0.8$

$$f_{cd} = \frac{0.85 * f_{ck}}{\gamma_s} = \frac{0.85*25}{1.5} = 14.17MPa$$

$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{280}{1.15} = 243.5MPa.$$

$$\varepsilon_u = 0.0035$$

$$\varepsilon_y = \frac{f_{yd}}{E} = \frac{243.5}{210{,}000} = 0.00116$$

$$\rho_b = 0.8\frac{14.17}{243.5} * \frac{0.0035}{0.0035 + 0.0016}\frac{150-34}{132}$$

$$= 2.8\ \%$$

$$Actual\ Steel\ Ratio\ (\ \rho\ ) = \frac{A_s}{bd} = \frac{1000}{0.7*1000*132} = 1.1\%$$

$\rho_b > \rho,$ The composite slab is under reinforced.

For under reinforced composite slab, the moment resisting capacity of the profiled steel sheet is calculated from equation (4.2) as,

$$M_{psc} = A_{sp} f_{yd} (d - a/2)$$

$$a = \frac{A_s f_{yd}}{f_{cd} b} = \frac{1000*243.5}{0.7*14*1000} = 24.85mm$$

$$M_{psc} = \frac{1000}{0.7} * 243.5(132 - 24.85/2) = 41.6\text{KNm}$$

From the moment envelops the Maximum positive moment is 26.5KNm < 41.6KNm, the steel deck is safe against flexure.

- Negative Reinforcement

Top reinforcement bars for negative moments are provided as per the critical bending moments at points shown on the bending moment diagram (A,B,C and D). Negative Reinforcement calculation based on the equations derived in Section 5.3.3.1 are shown in Table 6.6.

**Table 6.7 Negative Reinforcement for the composite slab.**

| Critical bending Moment location | Max. Bending Moment in (KNm) | *Effective depth of slab ($d$) in (mm) | Value of $K$ $k=M/bd^2f_{ck}$ | Value of $z$ in (mm) $=d\left[0.5+\sqrt{0.25-\frac{k}{1.134}}\right]$ | Reinforcement Area ($A_s$) in (mm$^2$) $=\frac{M}{0.87f_{sk}z}$ |
|---|---|---|---|---|---|
| A | 24.4 | 102.02 | 0.094 | 92.7 | 1008.5 |
| B | 8.2 | 102.02 | 0.032 | 99.05 | 317.2 |
| C | 12.3 | 102.02 | 0.047 | 97.57 | 483.0 |
| D | 26.5 | 102.02 | 0.102 | 91.8 | 1107.7 |

**Effective depth = 132.02 – (25 + ϕ/2) =102.02mm*

*ϕ( reinforcement bar diameter )= 10mm*

*b=1000mm,*

$f_{ck}$*=25Mpa,*

$f_{sk}$*=300Mpa.*

*6.4.9 Shear Bond Strength*

The maximum ultimate vertical shear force from the shear diagram is 31.7KN.

To balance this maximum shear flow, profile sheet with shear resisting capacity of 309.85Kpa, and corresponding $k_d$ and $m_d$ values computed in Section equal to 0.054 and

32.2 are found appropriate. Inserting the shear parameters in equation 4.18, the ultimate shear resisting capacity of the composite slab would be computed as,

$$V_u = \frac{bd}{1.25}\left( k_d \sqrt{f_{ck}^{'}} + m_d \frac{\rho d}{L_v} \right)$$

$$= \frac{1000*132}{1.25}\left( 0.054*\sqrt{28.6} + 32.2\frac{0.01*132}{1250} \right) = 34.6KN > 31.7\text{KN} \qquad \textit{ok!}$$

### *6.4.2 Construction Cost Comparison between Steel Sheet Profile-Concrete Composite Slab and Conventional Reinforced Slab*

- Material and Labor Cost

Bill of quantity sheets for the office building floor slab construction in two alternatives, i.e. when the slab is constructed out of steel sheet profile –concrete composite slab on the one hand and on the other hand the construction is out of conventional reinforced slab system are given below in Table 6.6 and Table 6.7 respectively for construction cost comparison purpose.

**Table 6.8 Bill of quantity sheet for the case when the building floor slab is constructed from steel sheet profile-concrete composite slab**

| Item No. | Description | Unit | Quantity | Unit Price in Birr | Amount in Birr |
|---|---|---|---|---|---|
| 1 | Steel sheet profile of 1mm thickness cost including forming in to EGA-700 and indenting embossments on the surface of the sheets. | Kg | 2030.32 | 11.20 | 22,739.58 |
| 2 | Erecting of props and joist beams for profile steel sheet support and laying the profile sheet on the leveled joist beam | $m^2$ | 19.48 | 50.00 | 974.00 |
| 3 | Fixing and joining of profile steel sheets to each other and to crossing beams | $m^2$ | 224.2 | 2.00 | 248.4 |
| 4 | Top reinforcement cost including cutting, laying in position and tying with black wire | kg | 869.4 | 6.60 | 5738.04 |
| 5 | Concrete Class C-25 poured on the profiled sheet as per the design depth. | $m^3$ | 35 | 650.00 | 22,750.00 |
| 6 | Removing the false work | $m^2$ | 19.48 | 0.90 | 17.53 |
| 7 | Painting one coat of anti-rust and three coats of enamel paint for the slab soffit (profiled steel Sheet) | $m^2$ | 258.64 | 10.00 | 2,586.40 |
| | **Total** | | | | **55,053.95** |

**Table 6.9 Bill of quantity sheet for the case when the building floor slab is constructed from conventional reinforced slab**

| Item No. | Description | Unit | Quantity | Unit Price in Birr | Amount in Birr |
|---|---|---|---|---|---|
| 1 | Slab false work including Erection of props, joist beams and laying of flat panels over the joist beams. | $m^2$ | 224.20 | 45.00 | 10089.00 |
| 2 | Top and bottom reinforcement bars cost including cutting, laying in position and tying with black wire | kg | 2090.72 | 6.60 | 13,798.75 |
| 3 | 150mm thick Concrete slab of Class C-25 poured on the flat panel | $m^3$ | 37.12 | 650.00 | 24,128.00 |
| 4 | Removing the false work | $m^2$ | 224.20 | 0.90 | 201.78 |
| 5 | Chiseling the slab soffit for further plastering | $m^2$ | 224.20 | 1.70 | 381.14 |
| 6 | Three coats of 3cm thick 1:3 cement mortar plastering. | $m^2$ | 224.20 | 25 | 5605.00 |
| 8 | Three coats of plastic paint for the slab soffit (profile sheet) | $m^2$ | 224.20 | 9.0 | 2017.80 |
| | **Total** | | | | **56,221.47** |

From the Bill of Quantity sheets it can be seen that *2%* cost saving is possible from using the composite slab instead of conventional slab construction. Moreover, the cost saving can be raised to about *11%* if the thickness of the profiled sheet used for the decking had been *8mm* gauge instead of *1mm* gauge sheet used for cost estimation as per the recommendation in the design example. It should be also noted that for the particular design example changing the gauge of the sheet profile would not bring about the composite slab failing to fulfill all the design requirements.

➢ Construction Time

The office building floor slab in the previous example is used for the comparison of time elapsed for construction in the two alternatives i.e. when the slab is constructed in steel sheet profile concrete composite slab in the one hand and on the other hand the slab is assumed to be built up through the conventional system. For the purpose of comparison, the same resources are assumed for the execution of the two construction options. Table 6.9 shows the assumed number of skilled and non-skilled laborers for each crew supposed to involve in the execution of the activities and their daily average production rate derived from the current construction labor cost in Addis Ababa City.

**Table 6.10 Assumed Construction crew and production rate.**

| Crew | Skilled Laborers No. | Unskilled Laborers (Assistants) No. | Daily production rate |
|---|---|---|---|
| Carpentry | 3 | 6 | $30m^2$ |
| Profile sheet fixture | 3 | 3 | $50m^2$ |
| Bar bender | 3 | 6 | 300kg |
| Chiseler | 3 | - | $44m^2$ |
| Plaster | 3 | 6 | $30m^2$ |
| Painter | 3 | - | $75m^2$ |
| False Work Removing | 3 | 6 | $167m^2$ |
| Concreting | 3 | 15 | $10m^2$ |

From the volume of work described in Table 6.7 and 6.8 and the assumed Construction crew daily production rate of Table 6.9, a total of 16 and 35 days are required to accomplish the slab construction by steel profile composite and conventional slab construction systems respectively. From the example it can be deduced that more than 200% time could be saved through the adoption of composite system over the conventional slab construction technique.

## DISCUSSION AND CONCLUSION

### 7.1 DISCUSSION

The profiled steel sheets used for composite slab construction decking produced in developed nations like the United Kingdom do have features somewhat different from the local manufactured profile sheets. The main reason for the difference in the forms of the profile sheets is the purpose of production. Profiled steel sheets produced in the local factories are not produced for slab decking but it is manufactured for roofing and cladding purposes only. To site some of the differences, the height of the corrugation for concrete slab decking purpose ranges from *38 to 75mm,* while the local sheet profiles has got profile corrugation height which is only *34mm* high. Moreover, the profile sheets for concrete slab decking purpose usually provided with longitudinal ridges, which is not the feature for the local profile sheets. It is found as an easy issue to form profile sheet of any shape with the manufacture of female and male pressing dies out of high carbon steel by local workshop machines. The dies are then can be quickly assembled and disassembled at any instant to the hydraulically operating forming machinery. Even the provision of embossments on the flange and/or web of the profile sheet can be facilitated with the help of machinery to form embossing cells on the surface of the pressing dies.

As far as the cross-sectional properties of the profile sheet is concerned, manufacturers usually provide manuals and Kality metal factory do have such manual too [13]. The expression given in Equation 3.2, the equation for second moment of area for the local profiled steel sheets, gives only 3% discrepancy with the values given in Kality Metal Factory's Manual. Cross sectional properties for the composite slabs usually defined based on positive moment resistance only [3]. However, it is also found important to account the cross sectional properties for negative moment regions, and therefore expressions for negative moment regions near the supports are discussed in Section 3.2. Due to the variation of second moment of area for the positive and negative moment regions, a model of the cross-section of the composite slab is developed to account for the cross-sectional variation in the spans of the composite slab.

Analysis and design procedures for different stage of loading for the composite slab are more or less the same as the recommendation given by British Standard, BS 5950 Part-4 [3]. However, the recommendation of British Standard for shear bond strength determination, which is based on proto type model test for fixing parametric values, is not found feasible due to the requirement of testing machineries beyond the scope of local laboratory facilities. Instead push out test was conducted to assess the shear resistance phenomena for this particular study.

From the combination of push out test results and shear flow and shear resistance equations, technique had been developed to drive parametric coefficients which was supposed to be determined through proto type test only. However, further proto type test for verification of the validity of the technique shall be under taken in future development.

Finally, a comparison between composite and conventional slab constructions is made through a typical example of an office-building floor slab designed for the two cases of construction. The composite construction has been found to be economical in account of direct cost and time saving. The direct cost saving could also be raised when lighter gauge of 8mm is used instead of 10mm gauge steel profile.

## 7.2 CONCLUSION

Even though local profiled steel sheets are manufactured for roofing and cladding purposes, it may be used for reinforced slab decking too. However, for better efficiency of steel sheet profiles in carrying design loads, the sections shall be modified to forms such as the type recommended by BS 5950 Part-4 (Figure 2.1).

Tensile strength test from three test samples, collected from Kality Metal Industry (local manufacturer), is failed to fulfill the minimum requirement specified in the manufacturer's manual. Proper quality control in the import of steel sheets shall be carried out in the future to attain at least the specified minimum mechanical properties.

Locally manufactured profiled steel sheet and concrete composite slab construction for building has been found to be economical in comparison to conventional slab construction technique practiced to date. It is therefore advantageous for local construction industries in terms of direct cost and construction time saving to incorporate this new technique for future development. For cost assessment, assumption was made that, profiled steel sheet, which at least satisfy the minimum mechanical properties, could be obtained with the same unit cost of the steel sheet in use by Kality Metal Factory by then.

In this study the push out test together with shear resistance and shear flow equations are employed to determine the important shear parameters *( $k_d$ & $m_d$)* which is in all literatures the proto type test is recommended for determination of the coefficients.

Present local building code has not provided sufficient detail on steel deck concrete composite slab design, it is hence advisable to incorporate detail treatment about the subject during future revision of the local code of practice.

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# APPENDIX A

**APPENDIX B**

**APPENIDX C**

# APPENDIX D

## DECLARATION

*"The thesis is my original work, has not been presented for a degree in any other university and that all sources of material used for the thesis have been duly acknowledged".*

**Candidate**

Name Rufael Redie

Signature ______________________