STRUCTURAL PERFORMANCE OF TWO WAY

NODULAR STEEL DECK-CONCRETE COMPOSITE SLAB

MSc. THESIS

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HAWASSA UNIVERSITY, HAWASSA, ETHIOPIA

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STRUCTURAL PERFORMANCE OF TWO WAY NODULAR STEEL DECK-CONCRETE COMPOSITE SLAB

MENALE ALAYE

A THESIS SUBMITTED TO THE DEPARTMENT OF CIVIL ENGINEERING FOR THE PARTIAL FULFILLMENT OF THE REQUIREMENTS ON THE DEGREE OF MASTER OF SCIENCE IN STRUCTURAL ENGINEERING

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DEPARTMENT OF CIVIL ENGINEERING

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I hereby declare that this MSc Thesis "**Structural Performance of Two Way Nodular Steel Deck-Concrete Composite Slab**" is my original work and has not been presented for a degree in any other university, and all sources of material used for this work are clearly acknowledged.

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DECLARATION

I hereby declare that this MSc thesis is my original work and has not been presented for a degree in any other university, and all sources of material used for this thesis have been duly acknowledged.

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ABBREVATTIONS

$\bar{\varepsilon}_c^{in}$	Crushing or inelastic strain
$\bar{\varepsilon}_c^{pl}$	Plastic strain
$\bar{\varepsilon}_t^{ck}$	Cracking strain
$\bar{\varepsilon}_t^{pl}$	Plastic strain
D_i	Flexural rigidity of slab
E_o	Initial elastic stiffness of concrete
F_r	Ultimate shear resistance of single nodule
F_{vy}	Allowable yielding shear stress
P_o	Uniform load for Plate theory analysis
P_t	Maximum concentrated load for plate theory
c_p	Perimeter around base of nodule
d_c	Damage parameter in compression
d_t	Damage parameter in tension
t_d	Thickness of deck
v_b	Bond strength between steel and concrete
v_{sc}	Shear resistance of the nodule as individual connector
ε_{nom}	Nominal strain
ε_t	Total strain
σ_{co}	Initial yield stress
σ_{cu}	Ultimate stress

σ_{max}	Maximum principal stress
σ_{nom}	Nominal stress
σ_t	Tensile stress
σ_{true}	True stress
σ_{bo}	Initial equi-biaxial compressive yield stress
σ_{co}	Initial uniaxial compression yield stress
I_{cr}	Cracked moment of inertia
I_e	Effective moment of inertia
I	Moment of inertia in the transformed section
Q	First moment of area about the neutral axis
V	Kirchhoff's shear on slab due to load
a	Dimension of square slab
p	Pressure invariant
q	Shear flow produced by load
w	Maximum deflection from plate theory
μ	Viscous parameter
ϵ	Eccentricity
Ψ	Dilation angle

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ABSTRACT

Composite slab is becoming more attractive with the advancement of building industrialization. Researches has shown that it has a superior advantages over usual types of slabs. But the technology was limited only for one way composite slab. The analysis and design concept of two way structural action of composite slab is not matured for design and construction. In this study, the analytical based performance feasibility study of (Wong, 1987) on two way composite slab with nodular steel deck is verified by finite element software using ABAQUS/CAE/Explicit. The plate theory results has an excellent agreement with the numerical results. The structural performance of the nodular steel deck was compared with traditional two way composite slab having one way corrugated steel sheet. Flexural test with concentrated load is developed by the concept of yield line theory and load-deflection, shear slip and load slip behavior are studied to compare the performance. The results revel that the nodular deck is superior in shear and load capacity. However it exhibit brittle behavior.

1. INTRODUCTION

The concept of “composite construction” refers to the use of two or more structural materials to act compositely as a single structural element or a system in order to exploit their best performance. Composite slab construction is composed of concrete and steel deck. In construction stage, steel deck used as a formwork and after construction it is used as reinforcement permanently.

Composite slab construction had gained its popularity due to its advantage of eliminating the need for formwork installation, minimum use of reinforcement bar, ease to be placed and handled, containing pre-engineered ducting for electrification, communication and air conditioning, providing safe platform, reduced time of construction and reduced dead load.

In 1950 Steel deck construction in concrete composite slab was first introduced by Granco Steel Products Company. By the same year in Europe “Provisional Regulations for Design of girder in composite construction” was published. In 1960 the first through deck stud welding was recorded on the Federal court house of Brooklyn (Inas Mohamood Ahmed, 2019). In 1967 an extensive theoretical and experimental studies was initiated at Iowa State University. Until 1974, three hundred forty-one (341) experimental specimens of various types were tested at Iowa state university.

Performance and behavior of composite slab is majorly depending on shear bond capacity of the interface between steel deck and concrete and composite action. So many researchers were doing research focusing on means of enhancing the shear bond and development of design method. Specifically for composite slab spanning in one direction provision for experimental set up, analysis and design is available. However, many researches has been publishing on shear transferring mechanisms (shape of steel decking, embossment, shear studs and end restraints), failure behaviors (slab with larger deflection, bond slip failure) also researchers has developed new analytical models. Despite its attractive advantages noticed in conventional type of slabs, there is limited number of researches on two way spanning composite slab. That is why it is didn't have provision for analysis and design until now.

1.1. Statement of problem

Because of the technological advancement of steel production industries and improvement of sophisticated finite element software, composite slab is becoming attractive research area due to its inherited advantages. However, limited number of studies has been done on two way spanning composite slab. Construction and performance feasibility studies showed that it has similar advantage over one-way composite slab like the conventional (Wong, 1987). Currently there is no any provision for analysis and design of two-way composite slab. Yet it is important to verify if the analytical based feasibility study is correct or not. Studies need to be done to develop analytical models and design method if the blended advantage of the two-way configuration and composite action is possibly achievable.

1.2. Significance of the study

An extensive experimentally supported analytical and parametric studies need to be done to settle the knowledge and harvest the advantage of the system. This work tried to put some effort to evaluate the ultimate capacity of simply supported waffle type two way composite slab considering possible shear bond mechanism using numerical method and adapt plate theories to compare analytical responses to step forward and opening a room for the development of experimental studies and developing analytical models.

1.3. Research Objective

1.3.1. General Objective

The general objective of the study is to reveal potential structural advantages of two way nodular steel deck concrete composite slab and opening a room for further development of the knowledge.

1.3.1.1 Specific objective

- Validate the analytical based performance feasibility of two way composite floor by finite element model
- Comparing the structural performance of two way composite slab of trapezoidal steel deck with proposed nodular steel profile.
- Studying the deflection contribution of secondary reinforcements

2. LITRATURE REVIEW

Composite action between steel and concrete is ensured by shear bond capacity between steel and concrete. Bonds are created for composite actions by means of mechanical interlock, frictional interlock by profiles shape, end anchorage provided by welded studs and end anchorage by deformation of the ribs at the ends of sheeting combination with frictional interlock (Eurocode 4: Design of composite steel and concrete structures, 2004).

Experimental and parametric studies were conducted to study the contribution of shear bond mechanisms in one-way composite slab. But there are limited number of studies are to understand the flexural capacity, shear bond capacity and failure behavior of two-way composite slab.

2.1. One-way composite slab without end anchorage

2.1.1. Mechanism of shear Connection

There are three types of shear connection between a profiled steel sheet and a concrete slab. At first reliance was placed on the natural bond between the two. This is unreliable unless separation at the interface is (slip and vertical separation) prevented, so sheets with reentrant profiles was developed. This type of shear connection is known as ‘frictional interlock’.

The second type is ‘mechanical interlock’, provided by pressing dimples or ribs into the sheet. The effectiveness of this embossment depends entirely on their depth, which must be accurately controlled during manufacture. The third type of shear connection is ‘end anchorage’. This may be provided where the end of the sheets rests on a steel beam, by means of shot-fired pins or by welding studs through the sheeting to the steel flange.

2.1.2 Shear Behavior of profiled steel decks and research updates

Many researches have been done to study the shear behavior of new profiles before construction. New steel decks are proposed by changing the shape, rib dimension and embossment type with the objective of improving shear bond strength between steel and concrete.

(Shen, 2001) evaluate the performance of new corrugated steel deck profiles of four different embossment types. The three proposed embossment types were compared with the existing. Initially push out test was conducted for all embossment types then twenty full scale slab bending test is performed.

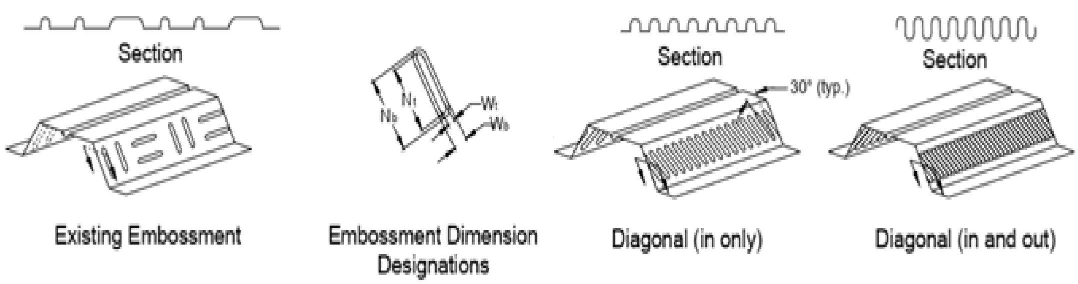

Figure 2.1 Type of sheet profile used by shen

First Yield Method (Heagler 1992), the ASCE Appendix D Alternate Method (Standard 1992), and Widjaja's(1997) iterative methods were used for conservative prediction of strength. Embossment depth, deck depth, concrete thickness, span length, concrete strength and deck yield strength were taken as parameters. Among those analytical models the first yield method and ASCE's Appendix D method found to be advantageous because they are easy to calculate and give reliable lower bound strength without experimental tests.

The behavior of the existing embossment type showed less flexural cracks after shear bond failure than the proposed one. ‘In and out’ and ‘in only’ embossment types have similar strength and behavior so it was recommending to manufacture the ‘in only’ type due to the difficulty in forming ‘in and out’ type. The new embossment type generally has 20-30% more strength with the deepest embossment depth.

M-k and partial interaction method were used to determine the shear behavior of trapezoidal steel sheet with rectangular dish embossment (V. Marimuthu, 2006). 18 specimens were experimented in 6 sets, 3 with short span and long span. Each set contain 3 specimens of which 1 with monotonic loading and 2 for cyclic loading. Similar studies were done in trapezoidal steel sheet profiled with chevron embossment at 90 degrees and zero degree by H.D Wright et al (1987) and S. Chen (2003) respectively.

Table 2.1 Comparison of M-k values of different Trapezoidal sheet profiles

Author	Embossment type	m	k
H.D wright et al (1987)	Chevron embossment 90^{o}	107.527	0.0401
S. Chen (2001)	Chevron embossment 0^{o}	84.665	0.0221
V.Marimuthu (2006)	Rectangular dish	87.956	0.0322

The shear capacity calculated by m-k method shows 26% higher value. The cyclic load doesn’t show any effect in load carrying capacity of the slabs. It was observed that rectangular dish embossment has lesser shear performance than chevron 90 and 0-degree embossment.

Slenderness first considered as strength parameter that affect accuracy of modeling longitudinal shear using both experimental and numerical method (Redzuan Abdullah, 2008). Load carrying capacity of composite slab was able to express as a function of slenderness parameter and shear bond property is unique in accordance with slenderness parameter. One compact and other slender specimen are required to provide good shear bond modeling properties for various slenderness ratios by linear interpolation or extrapolation. Force equilibrium method is used to relate shear bond stress and end slip.

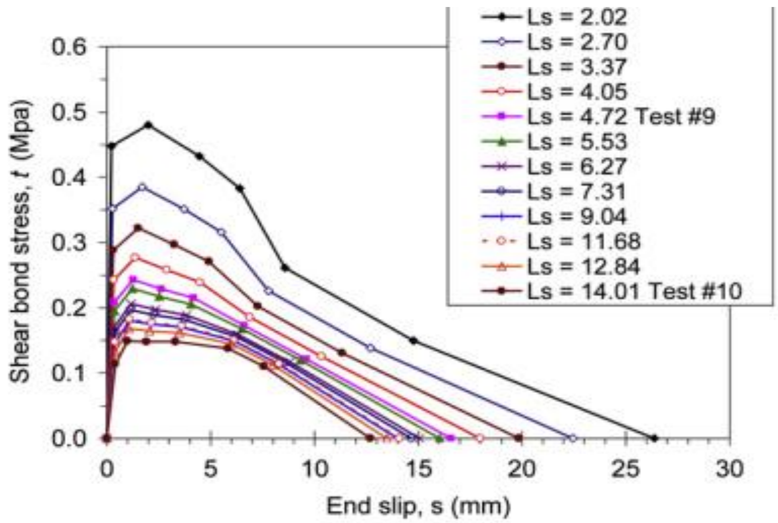

Figure 2.2 Interpolation of shear bond property

A new shear bond mechanism was found from in Barcelona (Miquel Ferrer, 2018). The new bonding system consists of producing band of many small crown-shaped cuttings by punching before profiling. The experimental study includes the conventional galvanized steel and concrete as well as ferric stainless steel with light weight concrete. Steel profile with traditional embossments was used for comparison. The longitudinal shear strength found 1.4 to 7 times higher. Yielding of both materials occur without slippage.

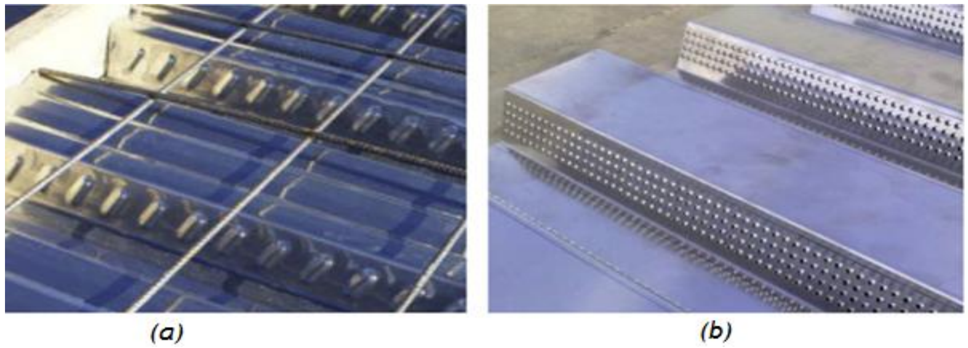

Figure 2.3 a) Traditional Embossment b) Punched profile

Table 2.2 Summary of new perfect bond technology

			F load at slip			Disp at Fu (mm)	
Specimen code	Contact	Test type	0.1mm	0.5mm	Fu(kN)	Max.Def	Slip
A80-E-n1	Embossed	Static	60.1	68.6	75.9	13.7	2.1
A80-E-n2	Embossed	Cyclic+Static	65.6	77.7	85	10.2	0.9
A80-E-n3	Embossed	Static	77.7	83.8	84.2	11.2	0.7
A80-HD	Punched	Static			141.5	33.6	no slip
C60-E-Csteel	Embossed	Static	23.5	20	34.1	36.1	75.2
C6-E-Inbox-n2	Embossed	Cyclic+Static	3.5	9.9	36.7	39.5	70.7
C6-E-Inbox-n3	Embossed	Cyclic+Static	4.4	7.9	34.9	36.2	66.8
C60-HD	Punched	Static			83.1	90	no slip
W60-E-1.2	Embossed	Static	17.6	17.6	25	134	15.6
W60-MD-1.2-n1	Punched	Static			179.1	83.9	no slip
W60-MD-1.2-n2	Punched	Cyclic+Static			176.8	68.1	no slip

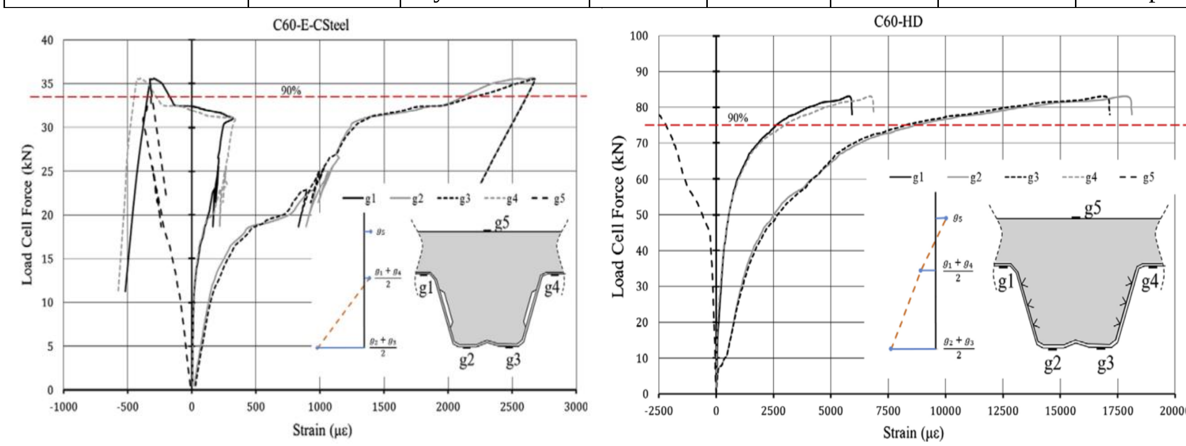

Figure 2.4 Force-strain diagram for normal and punched profile sheets

2.2. Two-way composite slab

The behavior and analysis of two-way simply supported concrete composite floor was first studied in Iowa state university by (Porter, 1974). Two-way structural system was maintained using one way corrugated steel sheet and supported the slab in all edges. Yield line analysis, orthotropic plate theory and shear bond analysis were used to estimate the performance. Five full scale two-way slabs subjected to monotonic and cyclic load and fifty-one full scale one-way experimental data were collected to evaluate the performance of the system. As the simply supported two way slab is considered for construction it is considered to be subjected to uniformly distributed load represented by equivalent four concentrated load.

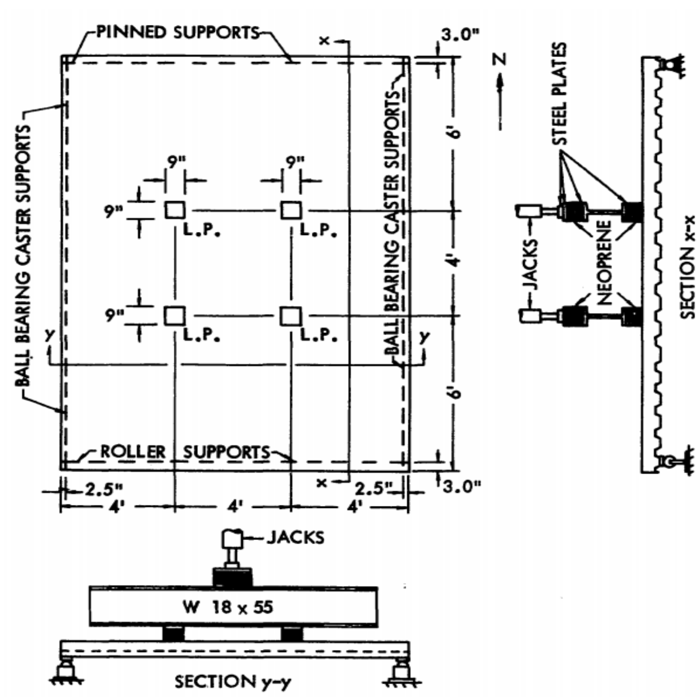

Figure 2.5 Full scale two way flexural test experimental setup

Crack propagation, end slippage, reaction distribution, strain and deflection was recorded. But there was a delay in loading due to time elapse to collect data from strain transducers. Steel sheet didn't allow seeing the real two way system because the one directional rib only carry and transfer the load in one direction and a little is contributed from the weaker direction. The reaction distribution to the strong direction was 78% keeping the span ratio to be 1.33 and the performance and failure behavior was similar to one-way composite slab.

The new structural configuration of two way layered nodular steel deck concrete composite slabs was first proposed by (Wong, 1987). This structural system is similar with structural arrangement of conventional simply supported two-way slab. Unlike corrugated ribs in one-way composite slab, new steel profiled with nodular shape was proposed. The feasibility of manufacturing of steel deck, construction and performance of the structural system was studied using analytical models.

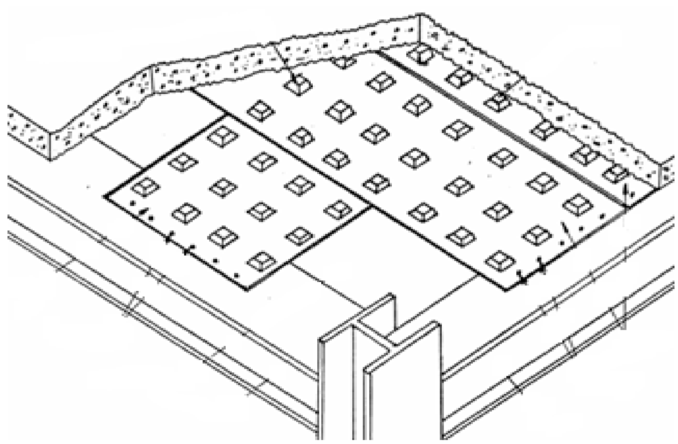

Figure 2.6 Isometric view of proposed two-way composite slab by (Wong, 1987)

Manufacturing process was possible by the technological advancement of the industries. The steel deck produced by passing flat sheet through series of roller imposing the shape and dimension of the profile.

The construction feasibility is concerned with keeping the reasonable ease of construction familiar with the practice. The process starts with positioning the bottom deck. Due to the weak cross section of the deck it needs shoring, pretention and welded. Then the top steel deck will be placed. In order to determine the number of shore and weld aiming to limit deflection and ponding effect, the equation of for tensile force under uniform load (Sandor 1983) was adapted assuming parabolic cable for different steel gage and slab thickness.

In performance evaluation of both uniform and concentrate load were considered. In concentrated load, the worst scenario is when a point load applied at the center of the slab but four concentrated load are more practical and actual. Thin plate theory used to evaluate the deflection, moment and shear performance of the proposed slab by transforming in to equivalent reinforced flat slab.

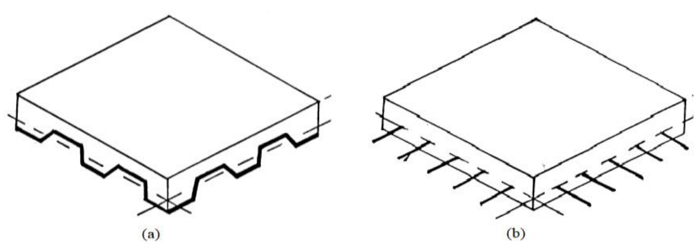

Figure 2.7 a) Nodular composite slab b) Equivalent transformed section

Table 2.3 Expression of Maximum deflection and shear

	Uniform load	Concentrated load
Maxumum Deflection	$w = 0.00406\frac{P_o a^4}{D_i}$ Eq. 2.1	$w = 0.03898\frac{P_t a^2}{D_i}$ Eq. 2.2
Maximum MOment	$M_x = 0.04203a^2$ Eq. 2.3	$M_x = 0.5904P_t$ Eq. 2.4
Maximum Kirchoff's shear	$V_x = 0.4361P_o a$ Eq. 2.5	$V_x = \frac{2.6831P_t}{a}$ Eq. 2.6

The resistance of horizontal shear at the interface is developed by the bond strength between concrete and deck and by the nodules, which act as individual shear connector. Total shear flow per center to center distance between nodules is the sum of concrete bond strength and shear resistance of nodule.

$v_b + v_{sc} > q$ Eq. 2.7

$q = \frac{VQ}{I}$ shear flow produced by load

v_b = bond strength of concrete and steel profile

v_{sc} = shear resitance of the nodule as individuaal connector

V = Kirchoff shear in slab due to loads

Q = first moment of area of about neutral axis

I = moment of inertia of the transormed section

The bond strength between concrete and steel only determined by test but it assumed to be 10% of the concrete compressive strength which is in range from 5 to 40% depending on steel and concrete characteristics as referring Civil Engineering Hand book, Urquhart, 1974. Estimation was made based on the assumption that the nodules shear off at their base. The shear resistance of single nodule is

$F_r = F_{vy} c_p t_d$ Eq. 2.8

Where F_r = Ultimate shear resistance of a single nodule

$F_{vy} = f_y\sqrt{3}$ is allowable shear yield stress of material

c_p = perimeter arround base of nodule

t_d = thickness of deck

The shear flow resistance provided by the nodules over the entire surface of the deck is given

$$v_{sc} = {}^{F_r}/_{s} \qquad \text{Eq. 2.9}$$

v_{sc} = uniform shear flow resistance provided by nodules

s = center to center distance between two nodules

Here the shear capacity of the sheet is only dependent on planar dimension of the nodule, strength and thickness of the sheet. But the surface condition and depth of nodule need to be considered for optimum contribution of shear as it is seen in one-way composite slab. This is to be determined by conducting experiment after reassuring the performance of the system. Comparison with the analogues one-way slab must be done so as to deduce the performance of two-way composite slab to set program for experimental studies.

Numerical study was done using COSMOS/M2.6 with similar structural configuration proposed by (Porter, 1974), in such a way that the slab is supported in all edges but with one way corrugated steel deck. The study (M.E. A-H Eldib, 2009) include parameters like slab aspect ratio, slab slenderness, embossment, steel strap, shear studs to see their effect on the ultimate capacity, distribution of reaction and deflection. Comparison was also done with analogues one-way slab numerical and experimental studies. The presence of studs and straps dramatically increase load and decrease deflection. Eventually in the presence of straps, composite slab tends to behave as two-way action at higher slenderness and aspect ratio. Considering the effect of embossment flattening the finite element result shows good agreement with the experimental value.

Conceptual development of the studies (Porter, 1974) (M.E. A-H Eldib, 2009) focused on only the reaction distribution and deflection by providing support reactions in the weak direction which cant exploit the strength of steel sheet and concrete and still the real two way structural action is not demonstrated.

A numerical study (M. Khalaf, 2013) continued to prove the structural behavior of two way composite slab system suggested by (Wong, 1987). Among different modeling technique (Real shape, Equivalent and grillage), real shape modeling technique found reasonably agreeing technique with experimental values. The major aim was to study the ultimate performance of the system in full interaction between steel and concrete, linear crack elastic behavior of concrete and neglecting the effect of secondary reinforcement were the basic assumptions. Slab aspect ratio, boundary condition, steel sheet profiles, corrugation cell aspect ratio, steel sheet depth with constant slab depth are the parameters considered and a total of one hundred finite element models were tested up to failure by uniformly distributed load using SAP2000.

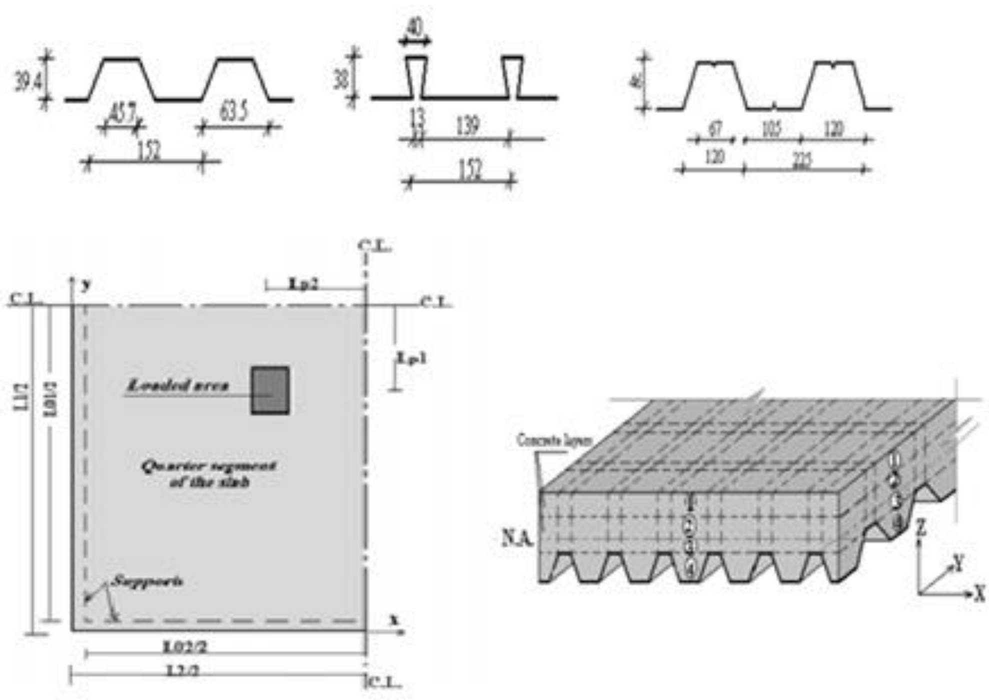

Figure 2.8 Analytical model and structural section of nodular steel deck

The reentrant shaped steel sheet showed lower deflection, higher stress in concrete and steel than the others which probably entail brittle failure behavior to occur. The manufacturing and construction feasibility need to question because of the difficulty for both manufacturing the whole dimension of sheet or difficulty in layering.

The square shape cellular shape gave the uniform load transfer in both square and rectangular slab which validate the proposed steel sheet profile by (Wong, 1987). From construction feasibility aspect, Reentrant shape is not feasible to assemble in layered or too difficult to manufacture the sheet component in desired plane dimension and also cold forming process.

Better composite action was obtained for the steel sheet depth equal to one third of total slab depth. Finally, it is proven that the two-way composite slab (waffle type) could sustain loads higher than one way slabs with lower deflection. (M. Khalaf, 2013) present the work as it was the first study on two way composite slab ignoring the contribution of analytical feasibility study by (Wong, 1987). Thus the result of numerical study needs to be verified and compared with the analytical study using it as a lower bound.

3. MATERIAL AND METHODS

3.1. Introduction

The finite element method is a numerical technique of finding approximate solutions to the complex partial differential equations. FEM is a method for dividing a very complicated problem into small elements that can be solved in relation to each other; the differential equations are formulated in such a way that they can be solved approximately with a computer. Since FEM is an approximation the result of a FEM calculation will be an estimation of the solution to the original deferential equation. The result is more consistent with the real solution if the structure is divided into smaller elements or modeled with element types of higher polynomial range. Both procedures mean that the numbers of degrees of freedom increases in the model, which will make the equation system. Due to relatively high cost of large-scale experimental research, a means of modeling composite slab using computer aided program is needed to broaden the current knowledge about the complete behavior and influence of the modified sheet profile and structural action.

3.2. General Overview of ABAQUS/CAE

ABAQUS/CAE provides a collaborative graphical environment that allows for simple modeling and generating complex geometry in to optimized mesh regions. Material properties, load and boundary conditions can be discretely assigned to the geometry.

Abaqus /CAE has Explicit and Standard solvers. Abaqus/Explicit is well suited to simulate brief transient dynamic event and very attractive for simulation of quasi static problems with severely nonlinear contact. Abaqus/Standard is applied to those portion of analysis that are well suited to an implicit solution technique like static and low speed dynamic.

Since ABAQUS/Explicit is a dynamic analysis procedure, a quasi-static solution was sought by applying load slowly and checking the energy balance of the finite element model. Both geometric and material nonlinearity were considered in the FE analysis. Appropriate material models for all components and suitable contact interactions with real boundary conditions were specified to model all test specimens accurately and efficiently.

3.2 Material property

1.3.2. Concrete

Concrete grade of C20 and C30 are used technically for validation of analytical model and experimental results respectively. C30 concrete grade is used for modeling the nodular two way composite slab. The uniaxial compressive and tensile behavior is formulated based on (Miland Hafezolghorani Esfahani, 2017).

1.3.3. Steel sheet and shear stud

Type I Gage 20 steel sheet is used for both type of steel is used in this study. The profile was used by (Porter, 1974). The trapezoidal geometry without embossment is used in for modified two way slab and also engineering properties are adapted for nodular steel deck. However the exact property of nodular steel deck needs to be determined only using experiments. The stress-strain behavior as linear elastic materials until yielding followed by plastic behavior as shown in Fig (3.2). Stress and strain on the flow curve or trues stress strain curve are used for modeling hence the engineering stress and strain are converted to the true stress and strain value which consider the instantaneous cross section.

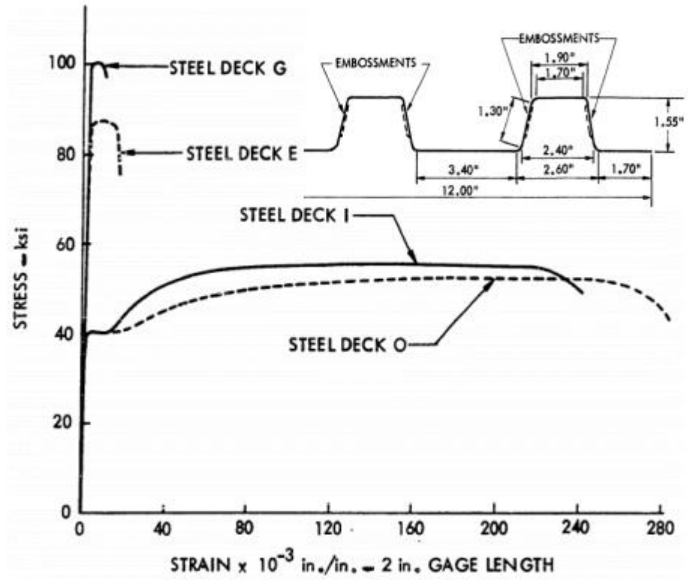

Figure 3.1 Stress strain diagram for Type I steel sheet (Lee porter 1974)

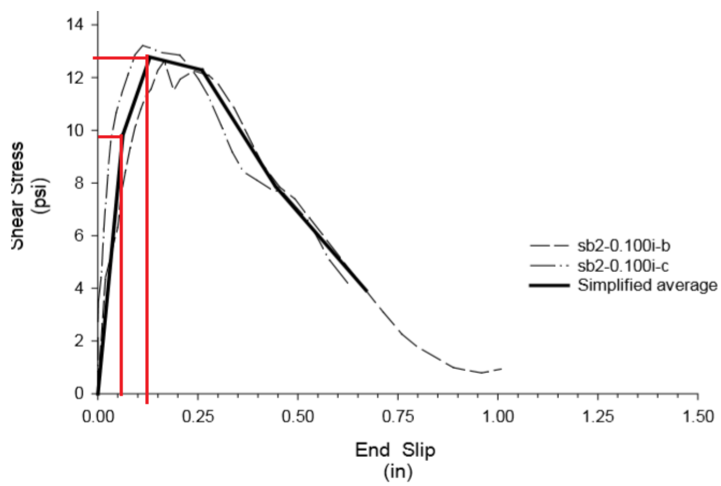

Figure 3.2 Shear slip behavior of Type I sheet profile

3.3 Constitutive model for concrete

3.3.1 Concrete Damage plasticity

The Concrete Damage Plasticity (CDP) is the most comprehensive model that was used in the composite slab simulation to define concrete behavior. The CDP model is applicable to concrete that is subjected to monotonic loading for different types of structures such as beams, trusses, shells, and solids and it is developed based on two concrete failure mechanisms: compressive crushing and tensile cracking. For this study, the model had used isotropic damaged elasticity in correlation with isotropic compressive and tensile plasticity to characterize the inelastic nature of the concrete.

Thus for the simplicity of FE model analysis, the effect of including reinforcement was taken into consideration by introducing some "tension stiffening" into the composite slab modeling to simulate the load transfer across cracks through the rebar. The rebar in the composite slab mainly serve as secondary reinforcement. However, it contributes some strength. Taking this in to consideration tension stiffening is introduced to simulate load transfer across cracks through the rebar.

Plastic parameters for concrete

Dilation angle (**Ψ**) Dilation angle measures the inclination of the plastic strain at a high confining pressure. A material with a low value of this angle experiences brittle behavior, whereas high values indicate that the material has a high ductile behavior.

Eccentricity (ϵ) Eccentricity is a parameter that defines the rate at which the function approaches the asymptote (the flow potential tends to a straight line as the eccentricity tends to zero). By decreasing the value, this may lead to convergence problems. The default is = 0.1, the default value was used in this study.

($\boldsymbol{\sigma_{bo}/\sigma_{co}}$)The ratio of initial equi-biaxial compressive yield stress to initial uniaxial compression yield stress is required as an input. The default is = 1.16, default input value of 1.16 was chosen.

(K) The ratio of second stress invariant on the tensile meridian, to the compressive meridian, adopted for different evolutions of strength under tension and compression. At the initial yield, for any given value of the pressure invariant p such that the maximum principal stress σ_{max} is negative, it must satisfy the condition $0 < K_c < 1$, the default value being 0.667 was used.

Viscos Parameters (μ)The viscosity parameter is defined to represent the relaxation time of the visco-plastic system. By changing this value, the length of the experimental simulation time for each step may be influenced. Moreover, the softening behavior and stiffness degradation behavior of the material models may be influenced. Instead of changing the viscosity parameter and step time simultaneously, by defining a small number μ convergence difficulties were overcome. Default value being zero was used.

3.3.1.1 Concrete behavior in tension and compression

For concrete subjected to uniaxial loading, the stress-strain response of concrete in tension exhibits a linear elastic relationship until the failure stress is achieved and beyond that point the concrete follows the softening stress-strain behavior. When the concrete is unloaded at any point from within the strain softening portion of the curve, the unloading response is weakened and the elastic stiffness of concrete is damaged. This deterioration of the stiffness is defined by damage parameter in tension (dt), which can range from zero, representing the undamaged condition of the specimen, to one, which signifies that the material has lost its total strength.

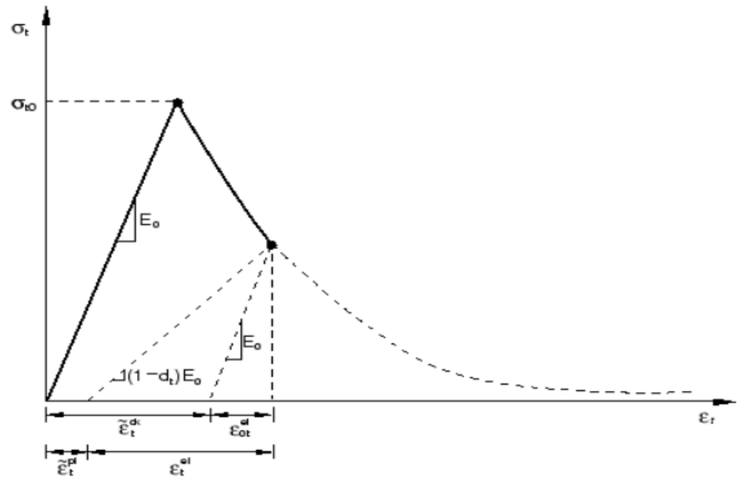

Figure 3.3. Stress strain response of concrete in tension to uniaxial loading

The cracking strain ($\tilde{\varepsilon}_t^{ck}$) is calculated as the difference between the total strain (ε_t) and the elastic strain that corresponds to the undamaged material; given as

$$\tilde{\varepsilon}_t^{ck} = \varepsilon_t - \sigma_t/E_o \qquad \text{Eq. 3.1}$$

In this equation, σ_t is the tensile stress and E_o is the initial elastic stiffness of concrete. Tension stiffening data values are provided in terms of the cracking strain, $\tilde{\varepsilon}_t^{ck}$. The unloading data are inputted in ABAQUS with tension damage curves (d_t versus $\tilde{\varepsilon}_t^{ck}$). ABAQUS/CAE automatically converts the cracking strain data to plastic strain

($\tilde{\varepsilon}_t^{pl}$) using the following Eq.3.2:

$$\tilde{\varepsilon}_t^{pl} = \tilde{\varepsilon}_t^{ck} - \frac{d_t}{1-d_t}\frac{\sigma_t}{E_o} \qquad \text{Eq. 3.2}$$

Under uniaxial loading, the concrete response in compression is linear until the initial yield stress (σ_{co}) value is reached. In the plastic region, the characterization of concrete behaviour is typically initiated with stress hardening and then followed by strain softening beyond the ultimate stress (σ_{cu}) value. Similar to concrete behavior in tension, the weakened elastic stiffness of concrete in compression is characterized with damage parameter (d_c) for the equivalent scale ranging from zero to one.

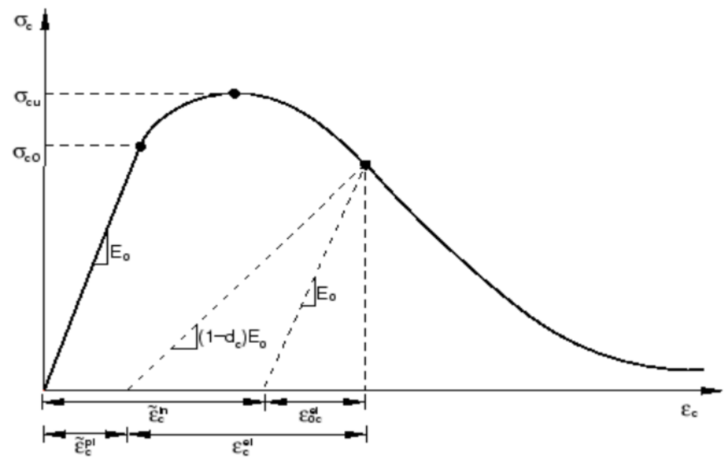

Figure 3.4. Compression stress-strain response of concrete to uniaxial loading

Compressive yield stress values are provided in ABAQUS as a tabular function of inelastic or crushing strain ($\bar{\varepsilon}_c^{in}$). The input of compressive stress and strain values are required to be positive (absolute) values. The compressive hardening data are provided in terms of inelastic strain ($\bar{\varepsilon}_c^{in}$), rather than plastic strain ($\bar{\varepsilon}_c^{pl}$), which is determined as the difference between the total strain and the elastic strain that corresponds to undamaged material as provided in

$$\bar{\varepsilon}_c^{in} = \varepsilon_c - \frac{\sigma_c}{E_o} \text{ Eq. 3.3:}$$

In this equation, E_o represents the elastic stiffness of concrete. The unloading concrete information is provided to ABAQUS in the form of compressive damage curves ($d_c$versus $\bar{\varepsilon}_c^{in}$).

Similar to the tension behaviour, ABAQUS also automatically converts the inelastic strain into plastic strain ($\bar{\varepsilon}_c^{pl}$) using the following Eq. 4.4:

$$\bar{\varepsilon}_c^{pl} = \bar{\varepsilon}_c^{in} - \frac{d_c}{1-d_c}\frac{\sigma_c}{E_o} \quad \text{Eq. 3.4}$$

3.4 Material Model for steel sheet and rebar

True stress and logarithmic strain are used to perform the finite element analysis of steel, which can be derived from the tensile coupon test results. The following two equations, Eq. 3.5 and Eq. 3.6, derived by Lubliner (1990) were used in this study to determine the true stress (σ_{true}) and logarithmic plastic strain (ε_{in}^{pl})

$$\sigma_{true} = \sigma_{nom}(1 + \sigma_{nom}) \quad \text{Eq. 3.5}$$

$$\varepsilon_{in}^{pl} = \ln(1 + \varepsilon_{nom}) - \frac{\sigma_{true}}{E} \quad \text{Eq. 3.6}$$

In the provided equations, σ_{nom} represents the nominal stress, ε_{nom} is the nominal strain, and E represents the modulus of elasticity. Table 3.1 illustrates the plastic behaviour of the composite steel deck that was used in ABAQUS to define the yield stress and plastic strain tabular data.

Table 3.1 Strucutral behavior of Type I steel sheet profile (tonne-N-mm)

	Elastic		Plastic		Shear bond	
Density	Elastic modules	Poisson Ratio	True stress	Platsic strain	Shear	Slip
7.85e-9	203000	0.282	290	0	43.643.6	1.52
			410	0.1823	56.4	30.48

3.5 Structural dimensions and modeling

3.5.1 Modeling program

Finite elements models are scheduled and modeled to answer sets of objective in this study. A total number of twelve (12) models are used. The first two models has a purpose to verify the feasibility nodular composite floor which are analytical based. Thus models will be important to reassure the performance of the new slab system proposed by (Wong, 1987). The next group consists of eight models which are used to revel structural behavior of two way composite slab of max lee porter (with trapezoidal profile) and slab with nodular steel deck. The last two models used to the contribution of secondary reinforcement for the deflection capacity by locating at the depth of centroid of the sheet profile and right the top of nodules.

Table 3.2 Designed finite element models

Designation	Depth of sheet	Profile	Concrete	Contact
KW125C20NPF	40	Nodular	C20	TIE
KW125C20NPfri	40	Nodular	C20	PENALITY
LP125C30NPF_Tr	40	Trapezoidal	C30	TIE
LP125C30NPfri_Tr	40	Trapezoidal	C30	PENALITY
LP140C30NPF_Tr	46	Trapezoidal	C30	TIE
LP140C30NPfri_Tr	46	Trapezoidal	C30	PENALITY
KW125C30NPF	40	Nodular	C30	TIE
KW125C30NPfri	40	Nodular	C30	PENALITY
KW140C30NPF	46	Nodular	C30	TIE
KW140C30NPfri	46	Nodular	C30	PENALITY
KW140C30fric_top	46	Nodular+rebar	C30	PENALITY
KW140C30fric_ep	46	Nodular+rebar	C30	PENALITY

3.5.1.1 Structural Dimension and boundary condition

All slabs have the same span ratio to see the ultimate performance of two-way action, for evaluation of the analytical based performance feasibility study (Wong, 1987). The slab has square geometry of 6m length.

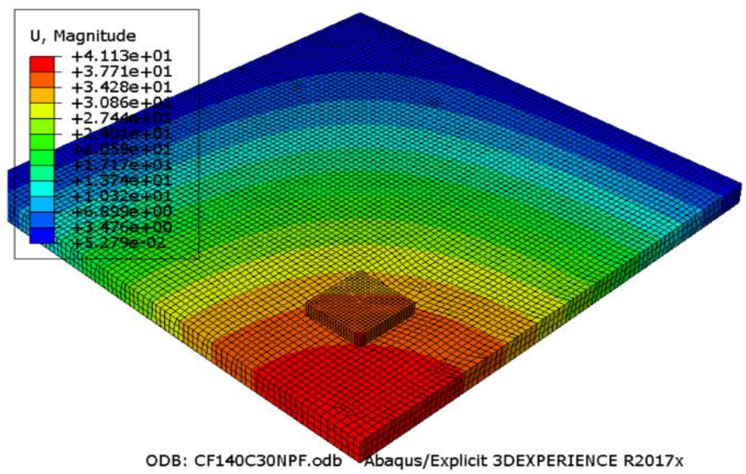

Figure 3.5. Sample FEM model

Taking advantage of symmetry, only quarter of the slab portion is modeled which reduce computational time by reducing elements. ZSYM and XSYM boundary conditions are applied for internal section faces and only vertical displacement is restrained for the faces closest to edge reaction.

Layered steel deck was first idealized for the possibility of nodular steel deck but this makes the section to be over reinforced while using normal gages. It is also difficult to make interlock specially when using profiles with embossment and difficulty to create perfect bond between sheets so structurally this technique would result a potential failure contact. Due to technological advancement of building industrialization it is considered that a complete set of steel sheet could be fabricated and transported to the construction site. Two steel sheets of gage 20 with different nodular details are used.

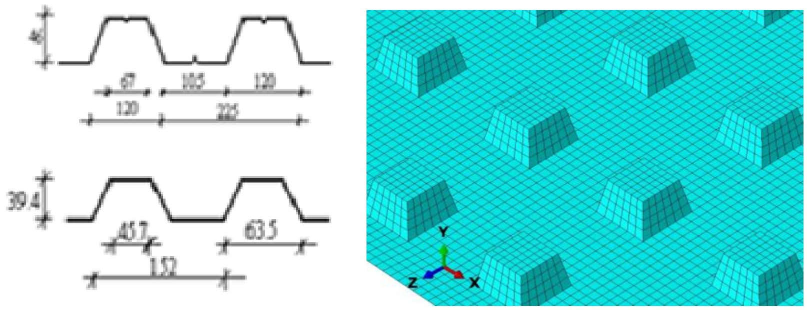

Figure 3.6. Proposed Nodular section

The two types of embossments are chosen here because of their performance of shear contribution seen in one-way composite slabs. The depth of embossment is the optimal depth recommend from previous studies but the numbers and dimensions are according to the type of nodular steel deck.

3.5.1.2 Constraints and contact interaction

Structural performance of composite slab majorly depends on the composite action between surface of contact between concrete and steel sheet. Once steel and concrete are in contact normal and tangential forces are generated due to self-weight and on set of load. Abaqus/Explicit packages capture such kind of mechanics between steel and concrete even in complex surface and nonlinear material property (Attarde, 2013).

For studying the performance of composite slab under different degree of interaction, appropriate and computationally non expensive contact modeling techniques are chosen. Currently contact modeling with Radial thrust connection elements is the most efficient one.

However other models don’t have shear bond law to be modeled by this technique. For full interaction behavior of steel and concrete tie constraint method found to be suitable to simulate perfect bond between steel and concrete. Tie constraints simulate as if nodes of the contact surfaces are kinematically constrained fully. It is also used to constrained neoprene and end stiffeners to the top surface of concrete.

For partial or natural bond contact definition of both nodular steel decks and embossed profiles, penalty contact method is used. It depends on friction formulation of contact between concrete and steel. It allows relative motion with finite sliding of contact surfaces handling the normal contact as hard mechanical contact and the tangential as isometric uniform friction. (Eurocode 4: Design of composite steel and concrete structures, 2004) suggest coefficient of friction for steel section without painting to be 0.5. According to (Burak Evirgen, 2014) static friction is taken to be 0.55. Embedded constraint type is applied to represent the embedded reinforcement where all nodes of the reinforcement is fully constrained concrete.

3.6 Mesh Size and Element type

3.6.1 Mesh size

The accuracy of finite element result depends on quality and size of element. The size and quality of element is directly depends on the complexity of the geometry and the engineering problem. Finer element result more accurate results but as the size become smaller, the computational cost will increase. Due to concrete softening concentration this concept is not necessarily applicable for structures like composite slabs. It is difficult to avoid convergence problem arise from using smaller mesh size. Abaqus recommend the size of the mesh depending on the size the smaller elements in a part.

In this study, the nodular steel deck influence the size of elements and makes stable time too smaller. The size of element decided by trial process of downsizing the recommended size until good quality of mesh is achieved. Mesh size of 40mm, 10mm and 150mm is chosen for concrete, steel sheet and rebar.

3.6.2 Element selection

The finite element specimens are modeled to simulated flexural test acting in composite with interface contact. So elements must be suitable for flexure with contact problem. Multiple elements are used for models with complicated geometry because of embossments and a single element is assigned if solid elements could generate hexahedral element.

For solid parts with rectangular shape elements C3D8I used. It is incompatible mode eight node brick element used for modeling solid element almost in all problem types. It is an improved version of C3D8 element. Shear locking is removed and volumetric locking is much reduced. This is done by supplementing the standard shape functions with so called bubble function, which, which have zero value at all nodes and non-zero values in between. C3D8I is recommended for all problem types specifically it is the best for problem of bending with contact.

The steel sheet were modeled as 4 node doubly curved thin shell, reduced integration element S4R. The reinforcing bars is assigned by a 2-node linear 3-D truss element T3D2. This element has been selected because axial force plays a key role in the analysis of bars and there is no need for elements with several nodes. Thus, volume and time of computation will be significantly reduced. With the purpose of obtaining accurate results.

3.7 Loading and solution control

The explicit solution method is a true dynamic procedure. Monotonic loading is achieved by "displacement control" loading application. Loading defined at specified reference point with prescribed deflection value. The displacement increased linearly using smooth function over a time step period which is longer than the natural period of the structure. This insure to obtain a quasi-static result, which confirmed by checking the kinetic energy below 5% of the internal energy at any instance during analysis.

Mass scaling is a technique of scaling up the mass of the model to increase the stable time increment and increase computation time. Scaling could be done by multiplying the mass of an instant in the model directly using scaling factor or by setting appropriate stable time increment first by looking the natural stable time. Semi-automatic with target stable time increment was implemented to reduce the computational time. However the rate of loading would become in dynamic range. Which disturb the energy balance so appropriate target stable time increment was validated.

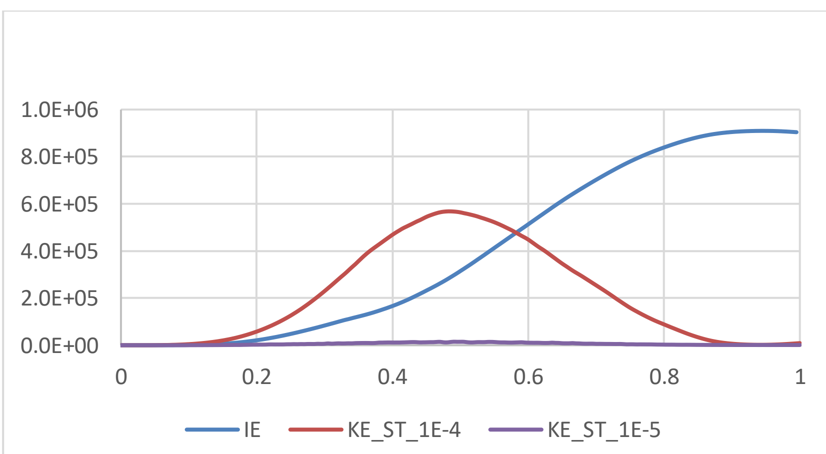

Figure 3.7 Energy balance for target stable time increment

Collapse is checked by looking at displacements of nodes getting very large displacements suddenly. Also looking at the global force. The force must drop at approximately the same time when nodes having large displacements.

3.8 Validation of finite element models

An experimental study on simply supported two-way composite slab was conducted by Max lee porter in 1974 at Iowa University. Five full scale slabs of the same size were subjected to static and cyclic loading. Includes several variables which influence the composite action and responses. Those variables are gage of steel deck, cross sectional area of reinforcement bars, thickness of slab, strength of steel sheet and concrete. From the five slab experiments, ‘slab 1’ is chosen because of its similarity with models of this study in geometry, structural arrangement, loading and responses.

3.8.1 Description of the simulation of model

The model is assembled from four parts. The basic part instances are the concrete, steel sheet and welded wire fabrics. To represent the natural contact interaction between concrete and steel sheet it was necessary to model the exact geometric property. As the steel sheet is profiles with embossment, it is a must to provide embossments physically to the surface by extrusion of face of steel sheet and concrete. Welded wire fabrics is modeled as a wire and assigned by truss element. Stiffeners for edge restraint and Neoprene are the other parts which are modeled as three dimensional rigid elements.

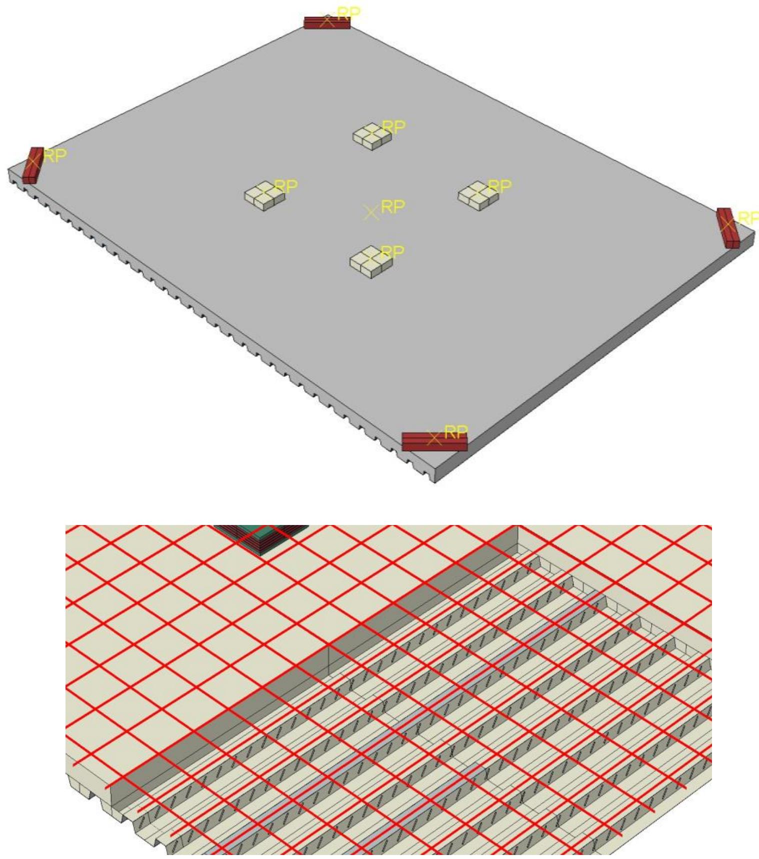

Figure 3.8 Validation model

3.8.2 Loading and boundary conditions

Test loading applied as uniformly increasing up to some points and unloading happens when popping sound is heard. Initially 1.4 kips was applied at each loading point and loading increased uniformly in increment of 2kips up to 13.4kips then loading and unloading continued up to the complete failure was considered imminent. The time of testing from zero loads until the test was terminated is approximately 4hr and 30min. There was eight minute elapse for recording of data up to the next loading. This loading rate is considered in the form of load versus time amplitude by trial and error. In reverse the failure load was investigated by displacement control application in a quarter of slab model.

The actual boundary conditions of the experiment consider thermal stresses and vibrational load and provide roller supports to relief such stresses. But in the model simply supported slab is represented by four sided pin support. At the corners the uplift is similarly constrained by introducing fully constrained rigid body. For the quarter model, four faces are constrained to their appropriate displacement and rotations. In this case there is no need to provide stiffeners.

3.8.3 Constraint and Contact Interactions

The best method of defining contact interaction in composite slab is using connector element called radial trust connector. Connectors need shear slip data to simulate the interaction and it is tiresome in creating wire connector in the model with coincident end nodes and computationally expensive. Specimens are to be modeled without knowing the shear bond behavior so the real geometric property of parts are introduced. Surface to surface contact of frictional formulation with penalty enforcement is applied.

3.8.4 FEM and Experimental result

3.8.4.1 Load deflection validation

The load deflection behavior is depending on the rate of loading. The loading was semi cyclic and there was an average time elapse of 8min while taking reading from strain gage, loading cell and theodolite. At the moment the applied load remained loaded and its static effect then collected on the next reading. This makes the simulation too difficult to get the exact loading rate. Also as it is seen in the deflection curve there is a missing data on the curve so the sharp curve is observed. Both load control and deflection control loading applied to validate. The load deflection curve shows better stiffness and greater peak load in deflection control case of which is the exact quasi static loading mechanism. When the load control loading is applied with load rate approximating amplitude by trial and error, it agrees with the experimental result.

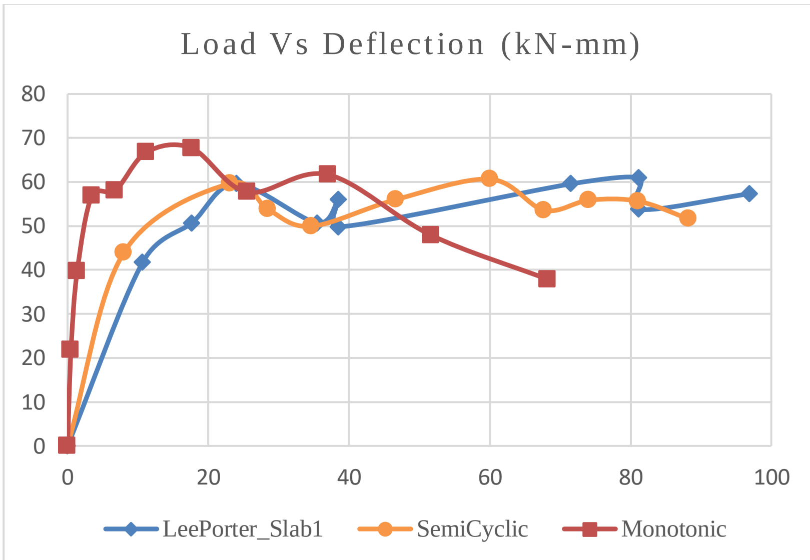

Figure 3.9 Load deflection curve of validation model (Lee porter 1974, slab 1)

3.8.4.2 Crack patterns

Crack patterns are drawn in scale from captured photographs. In real situation crack will initiate just after concrete start to set. The crack growth depends on shrinkage, support condition and temperature. The thermal and shrinkage effect on crack considerably make difference on crack pattern. The top crack pattern averagely agree with the finite element tensile damage plot. The bottom crack pattern didn’t match because it is plotted after the effect of crane load lifting to detach from the steel sheet. At this moment the lifting load and traction will trigger the flexure cracks. Due to this the bottom crack would not be captured from the model.

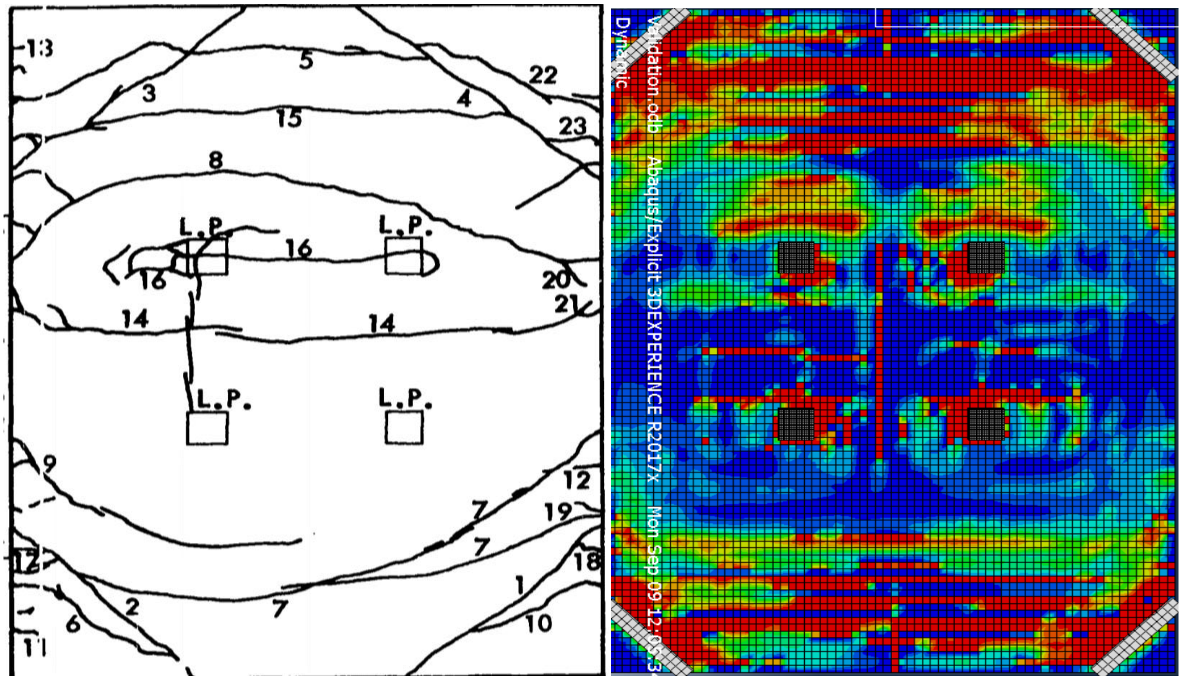

Figure 3.10 Crack pattern on the top of the slab

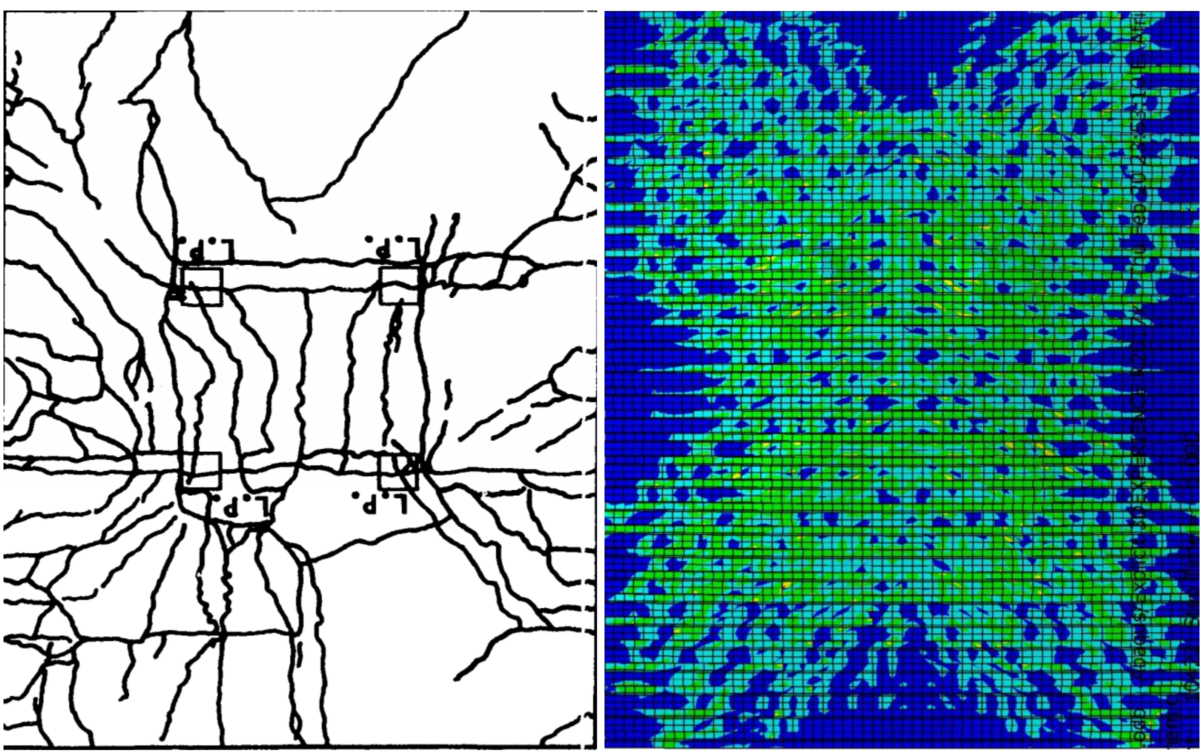

Figure 3.11 Crack pattern on the bottom of the slab

4. RESULT AND DISCUSSION

As the numerical models consistencies different parameters which influence the behavior of the composite slab important structural responses were extracted. Thus responses are failure load, center point deflection, section moment, contact shear and accumulated slip.

4.1 Comparison of analytical and the finite element result

Analytical approaches estimate the performance of the composite slab by; we are limited to see the response behavior. The maximum points on finite elements will be used to verify the performance.

Table 4.1 Evaluation of feasibility study with finite element result

	Mmax (kN-m)	Max deflection(mm)		Maximum Load(kN)			Shear bond (kN/m)
		Ie	Icr	Mmax	Dmax	Vmax	
Analytical	10.86	27.4	28.194	40.94	42.275	93.45	222.047
FEM-Full	37	39.29		67.132			-
FEM-Par	26.621	45		48.41			281.5

As mentioned in the literature, feasibility study was based on thin plate theory. For calculation of deflection, Naviers solution for differential equation was used. It is also found in Timoshenko and Woinowsky, 1959. Calculation was made using the cracked flexural and effective rigidity. Un-cracked section for deflection is not used because the loading magnitude to cause maximum permissible deflection is being determined without it. In the finite element model center point deflection examined in partial and full interaction cases. The full interaction case could represent the uncracked deflection and the possible maximum load carrying capacity of the specimen. Considering the deflections of effective and cracked section are correspondent with the deflection at full and partial shear connection, it can be observed that the finite element result shows greater deflection in both cases. But the partial shear connection can only be assumed when the slab is under shear bond failure because the deflection of cracked section is calculated from transformed section. In which it is based on flexural failure. The results are not conflicting each other but give information about the capacity in different mode of failure and still sound with the deflection of full shear connection and effective section.

In determining the moment capacity is was assumed that nodular steel deck transformed to equivalent flat slab, no slippage occurs, flexure is the predominant mode of failure and slab behavior is simulated as series of individual beam acting together in perpendicular direction. For this case, the slab model with the full shear connection suit with the assumptions. Also this assumption is improved and used in the code for calculating moment capacity of one way composite slab in full shear connection. The finite element result shows 2.4 times higher value which seems more realistic because it is determined assuming the real slab section.

The Navier's solution was only used to determine the deflection because moment expression is singular. It was recommended that it gives reasonable result when moment is evaluated with in 60mm of the point load would give reasonable result. The following results are found by using the simplified expression of Navier to examine the result of moments. Navier's solution for moment shows only 7% variation with both finite element models.

Table 4.2 Comparison of Navier's moment solution

Method	Pmax	Mc=0.5904Pmax	Mclassic	Mfem	Dev
FEMfull	67	39.53		37	0.068378
FEMpar	48.41	28.5619		26.621	0.072909
Beam theory	40.94	24.1546	10.86		1.22418

The maximum load capacity is stated for different conditions in the analytical method. These conditions arises from limit states conditions which limit the maximum load capacity of the slab. Indirectly these limits dictate the load capacity for different failure conditions. The maximum load cases are recommended when the slab is governed by maximum moment, maximum deflection and maximum shear. The finite element result of the full shear connection is reasonably comparable with load governed by maximum shear. But also the perfect bond mobilizes the full composite action for yielding the maximum moment. Similar to the higher value of the moment capacity over the analytical value, the maximum load value is again greater for flexure governing case. Also the maximum load in deflection governing case is very similar with the result of model partial shear connection, which is indicating that deflection will be higher when the interface is partially interacting and failed to achieve maximum moment.

The shear flow estimation is found to be 27% lower than the finite element result. The finite element result seems to be realistic than the analytical method. The analytical method was based on rough estimation of concrete to steel contact bond and which don't consider the shape of the connector, the size of the connector and deformation of the connector during loading.

4.2 Comparison Max lee porter and Chee Khong Wong two-way slabs

Max lee Porter (1974) was first scholar in developing experimental study on two-way composite slabs with one-way cold formed steel deck by providing support reactions for free edges of one-way slab. But Chee Khong wong's two-way slab better fit two-way action because of the nodular deck.

The problem of response no uniformity was addressed and said loud. But here for only checking, the reaction uniformity is assessed. Despite the limited parameters it dealt with, the nodular steel deck (Wong's slab) shows uniform reaction distribution. This can be seen in even shorter span and square dimension slab.

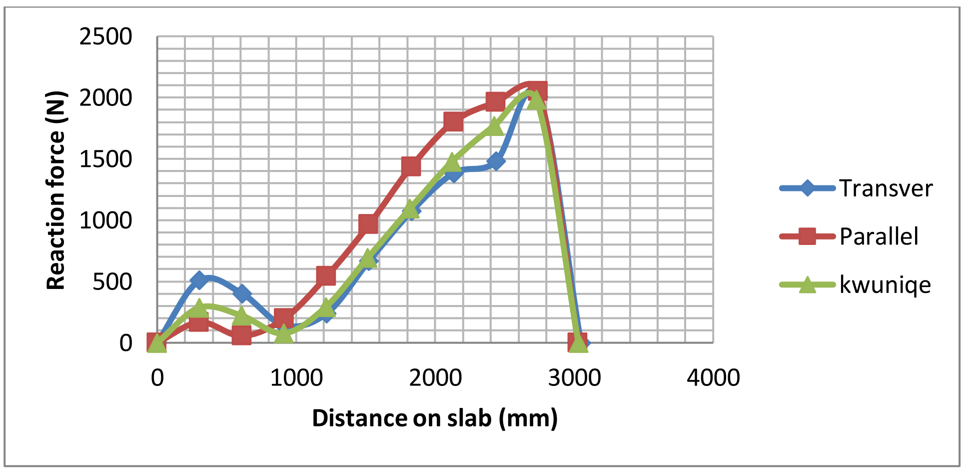

Figure 4.1 Reaction distribution of Lee porter and Chee Kehong Wong

The feasibility study with analytical method and the finite element verification models are more similar. So the performance of two way nodular steel deck and Max Lee porter's two way slabs need to be verified. For this purpose, load deflection and shear slip behavior are collected. For better shear bond strength and composite action the concrete strength is improved from the previous model.

4.2.1 Load capacity of nodular steel decks

Load deflection behavior of composite slab directly depends on, strength of concrete, both total depth of the slab and depth of steel deck, profile shape and surface (M.E. A-H Eldib, 2009) (M. Khalaf, 2013). The composite slabs with greater slab depth, concrete strength and perfect bond yield the maximum load carrying capacity. For similar scenarios the nodular composite slab gives much greater load capacity.

Table 4.3 Peak load comparison

	Shear Connection		(1)	(2)	(3)
	Full shear	Partial	% P/F	(%LP/KW)Par	(%LP/KW)Full
LP125	68.5583	48.1809	70%		
KW125	78.617	59.8274	76%	24%	15%
LP140	74.0963	60.9778	82%		
KW140	95.287	84.4212	89%	38%	29%

Column (2) and (3) tells us the percentage increase in peak load because of the nodular steel deck. Among those decks, greater increase in peak load is observed in the slab with greater depth (KWComFl_140). It improves by 29 and 38% while KWCom_Fl_125 has 15 and 24% in partial and full connection respectively. Column (1) of Table 4.3 shows the percentage of peak load of a model in partial and full shear connection in a single model. If we compare these percentages, the peak load of the nodular steel deck model much closer than the Lee porter slabs decks. Directly we can understand that the nodular sheet profile requires little effort to achieve perfect bond. All curves similar slope before the peak load. In the post peak region Lee porter slabs show smooth curves than the nodular decks. This indicates that the nodular steel decks have brittle behavior. But this behavior is best determined from load slip behavior.

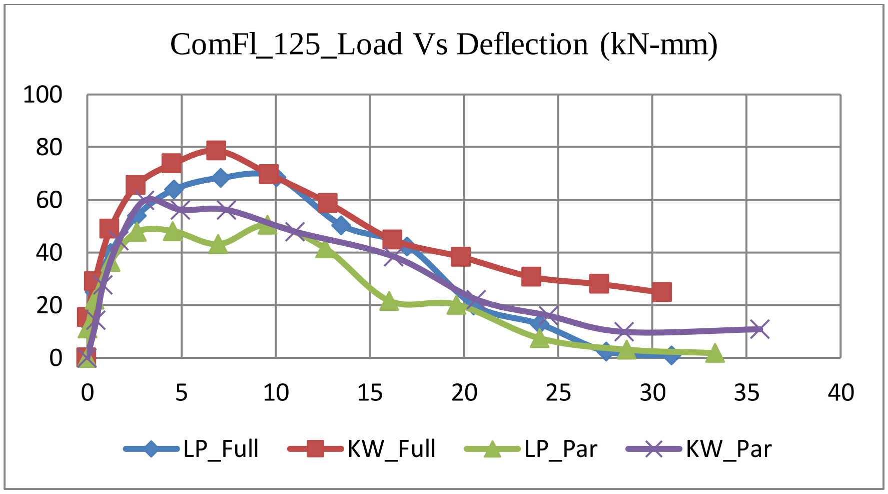

Figure 4.2 load versus deflection curve for ComFl_125

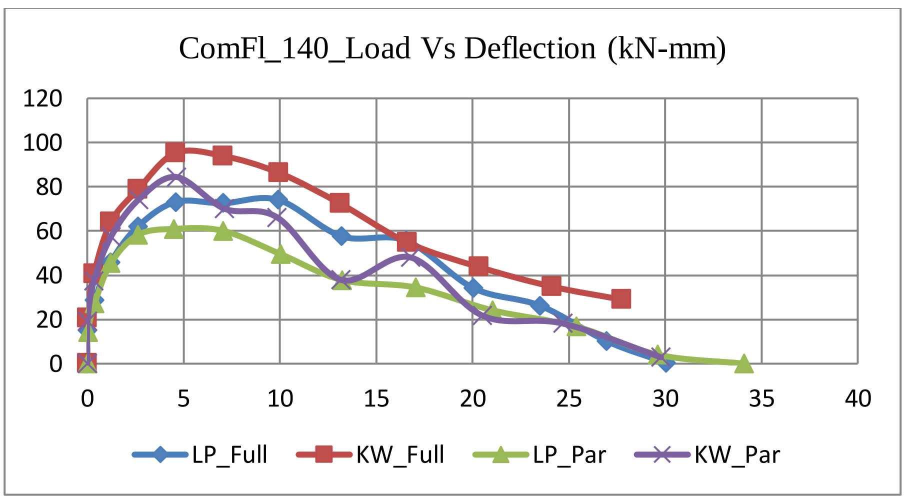

Figure 4.3 load versus deflection curve for ComFl_140

4.2.2 Shear capacity of nodular steel decks

A significant increase in shear capacity is observed in nodular steel deck. In one way corrugated composite slab shear capacity was dependent in the number of ribs. As can be seen the Figure 4.4 lee porter slab of depth 125mm exhibit superior magnitude of shear than the steel deck with greater sheet and slab depth. This is due to the number of ribs which are increasing the contact surface between concrete and steel sheet. Here also the nodular steel deck of depth 125mm shows superior shear capacity of all slabs.

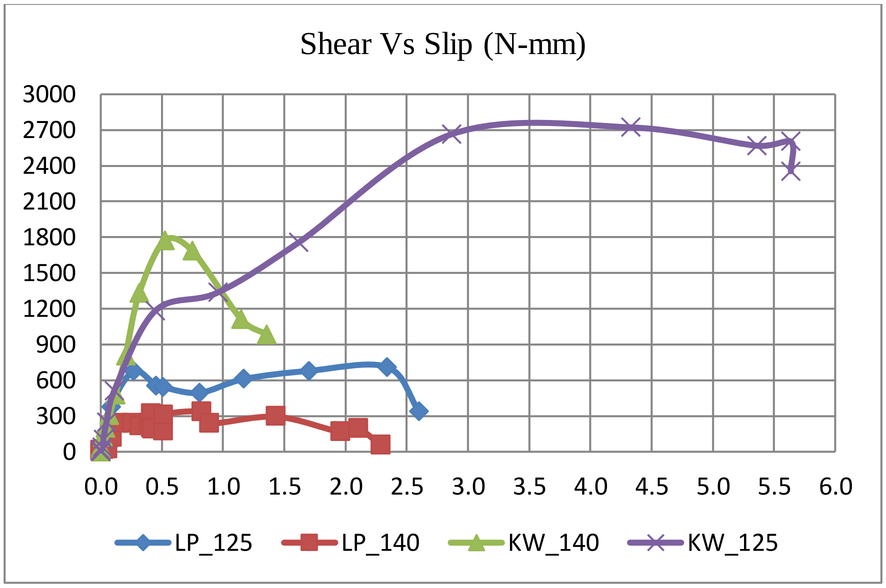

Figure 4.4 Shear-slip curve

In addition to the number of the nodules and number of ribs, the dead weight above the steel sheet also affect the shear slip behavior. The concrete thickness above the sheet make the normal force to dominate the tangential force. The geometry of the steel deck specifically the inclination or slope of the rib and the nodule affect shear mobilization.

4.2.3 Load-slip behavior of nodular steel deck

Load versus slip curve revel weather the failure behavior is brittle or ductile in one-way slab composite slab. Failure of slab is considered as brittle if the ultimate load is less than 110% of the load at the first slip (Eurocode 4: Design of composite steel and concrete structures, 2004). If the load slip curve of two way composite slab is evaluated with the same criteria of failure behavior, both slabs exhibit brittle failure. Thus shows steep slope before the first slip and suddenly falling curve. Among the curves 'LP_125' slab has an ultimate load of 126% over the load at first slip which is confidently ductile. Despite the failure is ductile for 'KW_125', post failure behavior is not sudden.

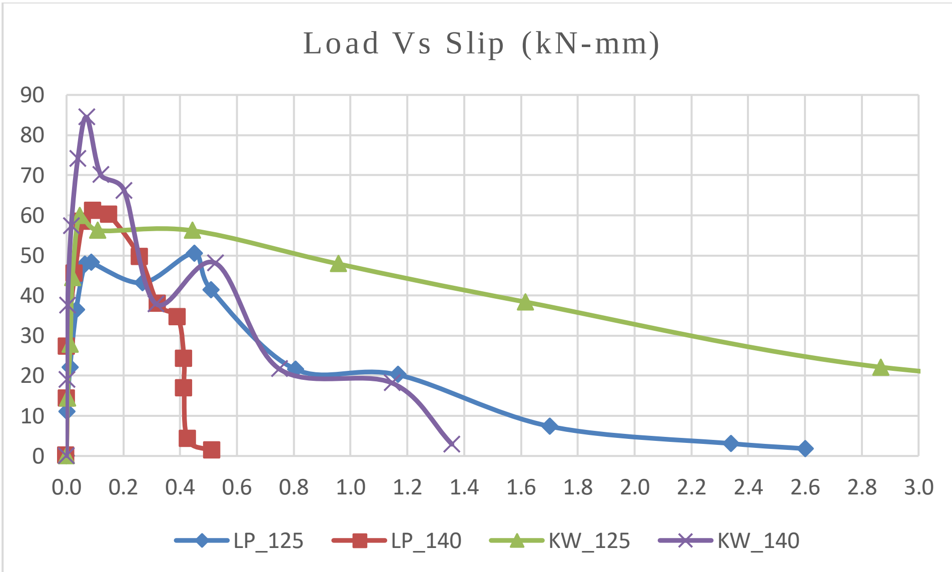

Figure 4.5 load versus slip curves

4.3 Contribution of secondary reinforcement for deflection

The deflection contribution of minimum reinforcement for shrinkage and creep was assumed to be null by (M. Khalaf, 2013). Here the deflection contribution at different depth of slab contribute considerable amount of load capacity. Difference in load capacity between the perfect bond and partial degree of connection was 12.9% without secondary reinforcement. For minimum reinforcement provided load capacity of slab with partial connection improved by 4.5% and 5.5% when bar is located at the top of the deck and at centroid of steel deck respectively.

Since secondary reinforcement is not in spectrum of an ultimate limit state, structural capacity could not be relied on it. However the nodular profile create a chance to locate it in more effective position (closer to concrete-steel sheet interface) in such a way that it could resist shrinkage effect.

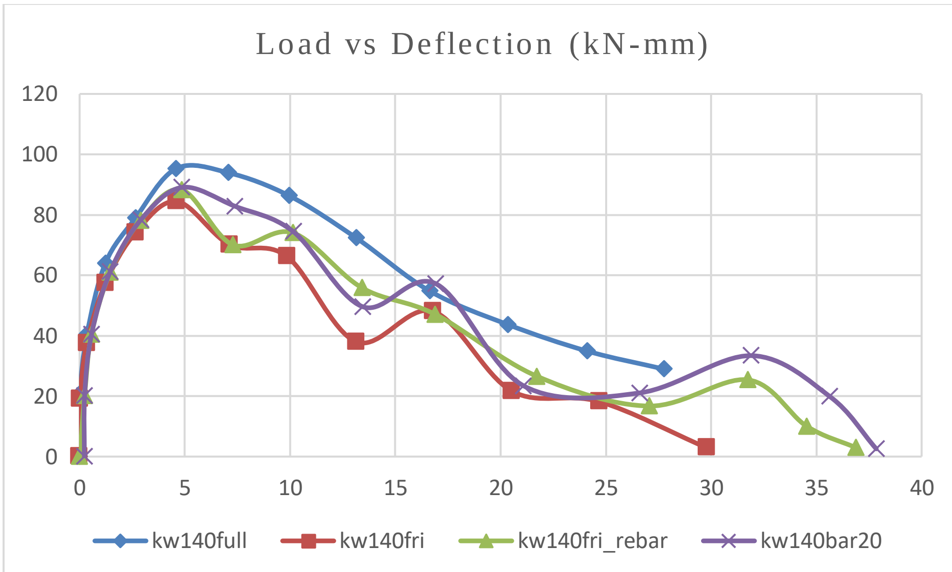

Figure 4.6 Load contribution of secondary reinforcement

5. CONCLUSION AND RECOMMENDATION

5.1 Conclusion

Finite element models had verified that the performance of nodular steel deck is more than the expected values by which it is predicted by plate theory. Despite the transformed section analysis assume flexural failure, the moment and deflection capacity are admissible as a lower bound for the finite element result. Independently the shear flow models have predicted the shear capacity with reasonable level of accuracy. So, it could be used with little modification on shear bond between concrete and steel sheet.

Nodular steel deck had shown a greater load capacity when comparing with the analogous two-way composite slab having one-way corrugated steel sheet under partial and full shear connection. The nodular steel deck with greater depth has small variation in load capacity between partial and full shear condition.

The shear capacity of the nodular steel deck increased more than 100%. This superiority is due to the nodules which are acting as independent shear connector. Among the nodular steel deck, a sheet with more number of nodules has shown a much higher value of shear even with smaller depth. In relation to this nodular steel deck has wider slip range before failure.

Despite all the advantage of nodular steel deck, they exhibit brittle failure which need to be considered in limit state condition to avoid such failure.

The topology of nodular steel deck creates a chance of positioning secondary reinforcement in more efficient location in due resisting shrinkage cracks simultaneously it contributes to the load deflection capacity to considerable amount.

5.2 Recommendation

However the rate of shear bond strength degradation is dependent on curvature of slab, modified push out test for bi directional shear need to be studied. This will help to formulate the shear behavior of nodular steel deck in finite element modeling.

The effect of sheet forming process on the elastic and plastic behavior of sheet profiles need to be determined by performing coupon test.

An intense parametric studies should be done further to know depth and span limit, failure behavior, optimized profile section, efficient embossment, end anchorage.

In addition to specific advantage of reducing weight of slab, utilization of light weight concrete is necessary for circumstances where normal force from applied dead weight dominate the contact force and reduce shear bond.

The geometric nature of the nodule will allow us to put reinforcement at the effective depth in perpendicular directions with small gage sheets. This condition may create ultra-high performance hybrid composite and reinforced slab.

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APPENDEX A: PARAMETERS OF CONCRETE DAMAGE PLASTICITY MODEL

Table A.1 Damage plasticity parameters for C20 concrete

Material parameter	C20	Plastic parameters	
Concrete elasticity		Dilation angle	31
E	21.2Gpa	Eccentricity	0.1
v	0.2	fbo/fco	1.16
		k	0.67
		Viscosity parameter	0
Concrete compressive behavior		Concrete compression damage	
Yield stress(Mpa)	Inelastic strain	Damage Parameter	Inelastic strain
10.2	0	0	0
12.8	7.74E-05	0	7.74E-05
15	0.000173585	0	0.000173585
16.8	0.000288679	0	0.000288679
18.2	0.000422642	0	0.000422642
19.2	0.000575472	0	0.000575472
19.8	0.00074717	0	0.00074717
20	0.000937736	0	0.000937736
19.8	0.00114717	0.01	0.00114717
19.2	0.001375472	0.04	0.001375472
18.2	0.001622642	0.09	0.001622642
16.8	0.001888679	0.16	0.001888679
15	0.002173585	0.25	0.002173585
12.8	0.002477358	0.36	0.002477358
10.2	0.0028	0.49	0.0028
7.2	0.003141509	0.64	0.003141509
3.8	0.003501887	0.81	0.003501887
Tension behaviour		Concrete tension damage	
Yield stress(Mpa)	Cracking strain	Damage Parameter	Cracking strain
2	0	0	0
0.02	0.000943396	0.99	0.000943396

APPENDEX A: PARAMETERS OF CONCRETE DAMAGE PLASTICITY MODEL

Table A.2 Damage plasticity parameters for C30 concrete

Material parameter	C30	Plastic parameters	
Concrete elasticity		Dilation angle	31
E	26.6Gpa	Eccentricity	0.1
v	0.2	fbo/fco	1.16
		k	0.67
		Viscosity parameter	0
Concrete compressive behavior		Concrete compression damage	
Yield stress(Mpa)	Inelastic strain	Damage Parameter	Inelastic strain
15.3	0	0	0
19.2	4.82E-05	0	4.82E-05
22.55	0.000119844	0	0.000119844
25.2	0.000214786	0	0.000214786
27.3	0.000333074	0	0.000333074
28.8	0.000474708	0	0.000474708
29.7	0.000639689	0	0.000639689
30	0.000828016	0	0.000828016
29.7	0.001039689	0.01	0.001039689
28.8	0.001274708	0.04	0.001274708
27.3	0.001533074	0.09	0.001533074
25.2	0.001814786	0.16	0.001814786
22.5	0.002119844	0.25	0.002119844
19.2	0.002448249	0.36	0.002448249
15.3	0.0028	0.49	0.0028
10.8	0.003175097	0.64	0.003175097
5.7	0.003573541	0.81	0.003573541
Tension behavior		Concrete tension damage	
Yield stress(Mpa)	Cracking strain	Damage Parameter	Cracking strain
3	0	0	0
0.03	0.001167315	0.99	0.001167315

APPENDEX B: FINITE ELEMENT RESULTS load-deflection and shear-slip output results of (Wong, 1987) and (Porter, 1974)

LP125C30NPF	
Deflection	Load
0	0
0.032884	12.9893
0.3959	24.9052
1.23931	39.8588
2.64627	54.012
4.61134	63.9232
7.10693	68.3798
10.0628	68.5583
13.5102	50.289
17.005	42.3911
20.5467	19.9385
24.0316	12.8068
27.5824	2.38997
31.0293	0.903688

KW125C30NPF	
Deflection	Load
0	0
0.033754	15.3954
0.395058	28.8455
1.22532	48.7712
2.59766	65.3014
4.52173	73.5688
6.90858	78.617
9.70342	69.5214
12.8222	58.6546
16.254	44.7084
19.873	38.2715
23.6362	30.6643
27.2376	27.9694
30.5756	24.7318

LP125C30NPFric	
Deflection	Load
0	0
0.034933	11.0911
0.398986	22.0166
1.24419	36.4669
2.65233	47.8231
4.5477	48.1809
6.97893	43.16
9.58294	50.5174
12.6596	41.4477
16.044	21.6292
19.6257	20.2285
24.043	7.39294
28.6679	3.09511
33.3666	1.82828

KW125C30NPFric	
Deflection	Load
0	0
0.483909	14.3283
0.845321	27.7597
1.68584	44.4542
3.06631	59.8274
4.97351	56.2232
7.41481	56.1686
11.063	47.8691
16.2954	38.3799
20.6783	22.139
24.5018	15.9968
28.5156	9.83674
35.7284	10.8744

LP140C30NPF	
Deflection	Load
0	0
0.027096	15.2043
0.383809	28.757
1.22059	45.9305
2.62592	61.9992
4.59001	73.1389
7.05858	72.566
9.9113	74.0963
13.2073	57.6039
16.5686	55.5841
20.0459	34.2078
23.5187	26.1988
26.9571	10.2431
30.051	0.362952

KW140C30NPF	
Deflection	Load
0	0
0.014317	20.7433
0.347839	40.4544
1.22221	63.9242
2.65197	78.9101
4.58693	95.287
7.07242	93.9011
9.96331	86.2816
13.1327	72.3872
16.6389	54.7379
20.3409	43.6897
24.1212	34.8808
27.7583	28.9603

LP140C30NPFric	
Deflection	Load
0	0
0.029672	14.2363
0.390028	27.2399
1.22566	45.413
2.62864	58.2507
4.51281	60.9778
7.05579	60.0758
10.0631	49.4626
13.2424	37.8029
17.0549	34.4579
21.0393	24.1458
25.4128	16.7391
29.6087	4.10296
34.1072	1.322.95

KW140C30NPFric	
Deflection	Load
0	0
0.014181	19.0415
0.346208	37.5085
1.23126	57.3908
2.66643	74.1853
4.62252	84.4212
7.12225	70.1164
9.86172	66.158
13.1546	37.9473
16.7869	48.0495
20.5284	21.6992
24.6854	18.1904
29.7862	2.92077

LP125C30NPFric	
Slip	Shear
0.0	0.0
0.0	3.5
0.0	10.1
0.0	21.9
0.1	47.9
0.1	379.0
0.3	677.0
0.5	555.0
0.5	541.0
0.8	494.0
1.2	610.0
1.7	679.0
2.3	710.0
2.6	340.0
3.2	51.8

KW125C30NPFri	
Slip	Shear
0.0	0.0
0.0	10.2
0.0	35.1
0.0	104.0
0.0	247.0
0.1	513.0
0.4	1180.0
1.0	1340.0
1.6	1760.0
2.9	2670.0
4.3	2720.0
5.4	2570.0
5.6	2600.0
6.4	2830.0
7.5	2830.0

LP125C30NPFric	
Slip	Load
0.0	0.0
0.0	11.1
0.0	22.0
0.0	36.5
0.1	47.8
0.1	48.2
0.3	43.2
0.5	50.5
0.5	41.4
0.8	21.6
1.2	20.2
1.7	7.4
2.3	3.1
2.6	1.8

KW125C30NPFric	
Slip	Load
0.0	0.0
0.0	14.3
0.0	27.8
0.0	44.5
0.0	59.8
0.1	56.2
0.4	56.2
1.0	47.9
1.6	38.4
2.9	22.1
4.3	16.0
5.4	9.8
5.6	10.9

LP140C30NPFric	
Slip	Shear
0.0	0.0
0.0	4.8
0.0	8.6
0.0	14.0
0.1	20.9
0.1	117.0
0.1	234.0
0.3	238.0
0.3	216.0
0.4	238.0
0.4	318.0
0.5	307.0
0.8	335.0
1.4	294.0
2.0	164.0
2.1	191.0
2.3	53.0

KW140C30NPFri	
Slip	Shear
0.0	0.0
0.0	15.1
0.0	39.6
0.0	123.0
0.0	199.0
0.1	308.0
0.1	479.0
0.2	807.0
0.3	1330.0
0.5	1780.0
0.7	1690.0
1.2	1110.0
1.4	985.0
1.4	224.0
1.4	0.0

LP125C30NPFric	
Slip	Load
0.0	0.0
0.0	11.1
0.0	22.0
0.0	36.5
0.1	47.8
0.1	48.2
0.3	43.2
0.5	50.5
0.5	41.4
0.8	21.6
1.2	20.2
1.7	7.4
2.3	3.1
2.6	1.8

KW140C30NPFric	
Slip	load
0.0	0.0
0.0	19.0
0.0	37.5
0.0	57.4
0.0	74.2
0.1	84.4
0.1	70.1
0.2	66.2
0.3	37.9
0.5	48.0
0.7	21.7
1.2	18.2
1.4	2.9

APPENDEX C: ANALYTICAL PERFORMANCE PRIDICTION BY (Wong, 1987)

Table C.1 Total shear resistance at the interface of deck and concrete

Steel deck thickness	Shear flow resistance(lbs/in)		
	By natural bond	By nodules	Total
18 gage	300	1262	1562
20 gage	300	968	1268
22 gage	300	822	1122

Table C.2 Concentrated load capacity of two way composite slab

		Capacity of slab for concentrated load		
Steel deck thickness	Concrete Thickness (in)	By Deflection	Moment (ft-kip)/ft	shear
20 gage	4	5.5	7.2	>21
	5	9.5	9.2	>21
	6	15	11.1	>21

Table C.3 Allowable concentrated load as governed by slab moment capacity

Steel deck thickness	Concrete Thickness (in)	Moment capacity (ft-kip)/ft	Concentrated Unfactored load (kips)
20 gage	4	6	7.2
	5	8	9.2
	6	10	11.1
Load above 9.2 kips produce deflection greater than L/180			

APPENDEX D: IMPORTANT VARIABLES OF VALIDATION SLAB

Table D.1 Summary of important slab variables of validation slab

	Slab 1
Applied load	
Ultimate load Pu-Kips/load point	13.7
Cyclic load-Kip/Load point	none
Thickness and support	
Average out to out thickness in, inches	4.83
Average depth of concrete over top corrugation, inches	3.28
Corner support condition	Restrained
Steel deck reinforcement	
Type of steel decking	I
Cross section of deck shown in Figure	D.1
Crross sectional area, Asd-insq/ft	0.625
Yield point, Fy-ksi	42.2
Supplementary reinforcement	
(welded wire fabric, WWF)	
Type of WWF	6x6x6/6
Position of WWF	on decking
Area of WWF parallel to deck corrugations	0.057
Area of WWF or T-wire transvers to deck	0.057
Yield strength (@0.005 strain), ksi	79
Concrete	
Average compressive strength, ksi	4160

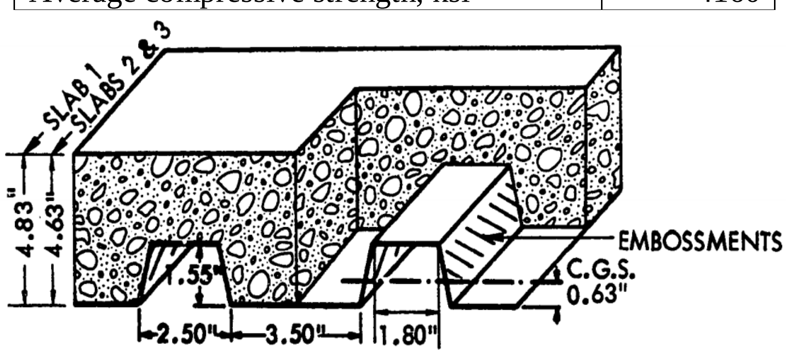

Figure D.1 Typical view of steel deck used for slab 1