JIMMA UNIVERSITY

JIMMA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

FACULTY OF CIVIL AND ENVIROMENTAL ENGINEERING

STRUCTURAL ENGINEERING STREAM

PERFORMANCE OF SHEAR CONNECTION IN COMPOSITE SLAB AND STEEL BEAM WITH RE-ENTRANT AND OPEN TROUGH PROFILE STEEL SHEETING

A Thesis Submitted to School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Structural Engineering

By

Alemu Feyissa Dadi

July, 2021

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Alemu Feyissa Dadi

Main Advisor: **Engr. Elmer C. Agon (Associate Professor)**

Co-Advisor**: Eng. Diosdado John Corpuz (Associate Professor)**

July, 2021

Jimma, Ethiopia

DECLARATION

I declare that this research entitled "Performance of shear connection in composite slab and steel beam with re-entrant and open through profile steel sheeting" is my own original work, and has not been submitted as a requirement for the award of any degree in Jimma University or elsewhere.

Alemu Feyissa Dadi ___________________ _______________

Name Signature Date

This thesis has been submitted for examination with my approval as university supervisor.

Engr. Elmer C. Agon (Associate Professor) ________________ __________________

Main Advisor Signature Date

Eng. Diosdado John Corpuz (Associate Professor) _____________ ___________________

Co-Advisor Signature Date

ABSTRACT

The structural composite construction techniques find increasingly wide applications in bridges, ramp, buildings and other different area throughout the world focusing on the composites systems. Composite structures are created by combining two or more structural elements to act as a single combined structural unit, where each element behaves in structurally efficient manner. Component of composite structure in this study were composite slab, profile steel sheet, steel beam, shear connector and reinforcement cages. From thus component the performance of connector was studied in this works. *The main purposes of this paper are to describe the structural performance of demountable shear connectors in composite slab and steel beam with profile steel sheet analysis through ABAQUS 6.14.*

An accurate and efficient non-linear finite element model was developed to study the behavior of demountable shear connectors prefabricated through deck. The material non-linearity of concrete, connector, profiled steel sheet, reinforcement and steel beam were included in the finite element model. A 24 specimen were carried out to investigate the load-slip behavior of the demountable shear connectors in composite floor with profiled steel sheeting.

Result obtained from finite element analysis was carried out and verified with the experiment results obtained from journal[1]. The capacity of shear connection result differences obtained from journal to finite element analysis was 3.29% only, that means the parametric study used in both investigations were nearest to each other in accuracy. The capacity of shear connection, load-slip behavior of stud and failures modes were predicted.

In the result diameter of connector increase 19 to 22mm, then the capacity of shear connection also increases by 26% and 17% in both open and re-entrant trough profile steel sheet respectively and since height of sheet increase from 50 to 64mm the capacity of connector increases by10.6% and 10.06% in open and re-entrant trough profile steel sheet respectively, similarly, since the concrete grade increase C-40 to C-60 the capacity of shear connection again increases 26% and 20% with open and re-entrant profile steel sheet respectively and also the same effect with dilation angle. The capacity of shear connection obtained from finite element analysis were compared with the design strength calculated using European code for stud shear connector in composite slab and steel beam with deck perpendicular to steel beam. It found that the design rules specified in European code were generally conservative.

Keywords*: finite element modeling, demountable shear connector, profile steel sheeting, slip*

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ABBREVIATIONS AND SYMBOLS

AISC	American international standard code
Asc	cross sectional area of connector
Bo	average width of ribs
b1	minimum ribs width
b2	maximum ribs width
CDP	concrete damage plasticity
d	shear connection shank diameter
dt	tensile damage
dc	concrete damage
EC4	European code four
fc	compressive cylinder strength of concrete
fck	characteristic cylinder compressive concrete
fcm	ultimate strength of concrete
FE	finite element
FEA	finite element analysis
Gi	group of models in I specimen
h_p	depth of ribs or sheet
h_{sc}	overall height of shear connectors
LVDT	linear variable displacement transducers
N	number of shear connectors
Nr	numbers of connectors in one rib
OTPSS	open trough profiled steel sheeting
RC	reinforced concrete
RETPSS	re-entrant profiled steel sheeting
UB	universal beam

ACRONYMS

Ec	young's modulus of concrete
Ecm	elastic modulus
fu	ultimate tensile strength of shear connector
fy	yield strength
Gf	fracture energy
Kt	reduction factors as Eurocode
Pr	shear strength of shear connections embedded in profiled metal decks
P_{Rk}	connector characteristic resistance
Q_n	nominal shear connector
r1	reduction factors
Rg	group effect factor
Rp	position effect factors
s_u	ultimate slip capacity
γv	partial safety of shear connection
σc	compressive stress in concrete
εc	nominal strain
εc1	strain at peak point
ε^$(\sim in)$	compressive inelastic strain
ε^$(\sim pl)$	compressive plastic strain
η	nominal to peak strain
β	Material angle of friction

CHAPTER ONE

INTRODUCTION

1.1 Background of the study

Composite structures are more widely used in different construction industry throughout the world. These structures are created by combining two or more structural elements to act as a single combined structural unit, where each element behaves in structurally efficient manner. Is that the properties of each material can be combined to form a composite or form one-unit structures which has a better performance than its separate parts. In steel concrete structures, concrete have well in compression and steel in tension then the combination leads to a highly efficient. In case of composite behavior, the load resistance capacity and the stiffness of the structures in the combination were increase.

Mechanical connectors are used to develop the composite action between steel beam and concrete. Demountable shear connection an interconnection between the concrete slab and steel beam-I section components with profiled steel sheet of a composite member that has sufficient strength and stiffness to enable the all components to be designed as parts of a single structural member. The demountable shear connection provided between the steel-concrete composite structures to interact all part act as a unit structure. The surfaces of the demountable shear connector embedded in concrete slab and the headed part of connector was tied in to the slab that used for resist the uplift forces. The collar edge of the connector tied to the sheet and top flange steel beam with nut parts of the connector to transfer the load between composite slab and steel beam with the deck. This connector was used in composite as: all components have sufficient strength and stiffness, resist longitudinal shear, resist slip, resist uplift forces, as connection and load transmission.in this paper the capacity of demountable shear connection was conducted by finite element analysis ABAQUS software. In finite element model there were different part created as per push off test arrangement present in the paper [1].

In the model the profiled steel sheet and concrete slab were putted in transverse direction and perpendicular to steel I- section of beam parts. Both re-entrant and open trough profiled steel sheet was arranged parallel with concrete slab and the reinforcement cages was embedded in the slab below the height of headed studs. The models in push test were sectioned symmetrically on the web of steel beam and again section symmetrically at left part on half at centerline of flange and concrete slab to made the quarter part of test setup. In this model all composite structure were interact with demountable shear connection to give a sufficient strength and investigation of it. The well arrangement of demountable shear connector, open and re-entrant trough profiled steel sheet, steel beam, concrete slab and reinforcement cages were compiled to form a model, thus used to identify and perform the performance of demountable shear connection.

1.2. Statement of Problem

During the steel and concrete material interconnection to form a composite structure there were a formation of different problems. When the performance of shear connection between the concrete and steel part were not efficient there is a problem occurred in the composite structures. Since the shear connection was absent in the composite action there is the excessive slip formation which means the premature of failures developed.

In steel-concrete composite structures there is the intractable problem was occurred, that means the steel element like steel beam and profiled steel sheet were affected by induced fatigue problems. In construction of composite structures, it takes more time, if the shear connection was not fabricated and absence of profiled steel sheet in the steel concrete composite structures, that means there is a formation of a problem between them like as load transfer from one to another due to high stress occurrence between them.

Since the demountable shear connection was between composite slab and steel beam sections with profiled steel sheeting there is no formation of slip problem, no problem of interaction between the two materials and also used to reduce the time consumption of their fabrications during the construction. When the strength of demountable shear connection was well investigated there is a good mechanism to transfer load between composite structures and also there is no any excessive stress to form a failure.

1.3. Research Question

The main aim of this thesis is to investigate the behavior of an innovative type of demountable shear connector in composite structures. The following research questions should be answered in this thesis:

- What are the effects of adding profile steel sheet types in composite structure while composite beam and slab bond by shear connector?
- Which size of shear connection will give more strength in composite structure with profile steel sheeting?

- What is the effect on the capacity and behavior of shear connection by changing profile steel sheeting geometries and the concrete strength class?
- Can European design rules specifications codes be conservatives in the estimation of shear connection capacity with finite element analysis result?

1.4. Objectives of the Study

1.4.1. General objective

The main objective of this research is to investigate the performance of shear connectors in composite slab and steel I-section beam with profile steel sheeting.

1.4.2. Specific Objectives

Specific objective of this study would be:

- To investigate the effects of adding profile steel sheet type in composite structure while composite slab and beam bond by shear connector.
- To identify the strength and capable of the shear connectors in composite slab and steel I-section beam with profile steel sheet depend on its size.
- To study effect on the capacity and behavior of shear connection by changing the profile steel sheeting geometries and the strength of concrete.
- To study the conservative result in the estimation of shear connection capacity from European design rules specification codes with finite element result.

1.5. Significance of the Study

In an experimental study, in order to acquire more realistic results in terms of composite structure components behavior, the shear connectors may not be lower functions than same certain values of other components. It results requires different data for specimen and that makes the experimental study of all composite structures (slab, beam, shear connector and profile steel sheet) at the same time is more difficult. Therefore, performing such an experimental study of a thus composite structures requires more different and complex test setup, more instrumentation (equipment), and extra human labor and an expectable budget. But conducting all data of thus structures and simulate a finite element analysis in comparison with an experimental study is very preferable and reliable technique from the viewpoint of difficulty, time saving, human force and budget.

In addition to the above, this study will have a significant value to select appropriate dimension, types and shape arrangements of shear connectors in composite beam with profile steel sheeting.

Finally, it may help to minimize the gap for other researchers and gives full confidence to analysis composite structures by these techniques.

1.6. Scope and limitation

It is more necessary to define the scope of the research topic since composite structures are concern with different component and also have variety of strength that can be occurred in buildings. The scope of this research is to simulate the shear connection in composite beam with both re-entrant and open trough profile steel sheeting modeling by finite element method software (ABACUS) to obtain load-slip of the shear connectors, failures of modes, capacity of shear connection from parametric study, effects of change of depth on strength of shear connector are predict from the finite element analysis and compare with experiment results.

All component is simulate using as solid element but profile steel sheet is simulate using shell element, whereas the reinforcement bars are simulate using node-element in finite element. In both comparison purpose there is a need to define the ultimate load of a composite beam and identify the load-deformation and deflection with different composite structure specimen includes stud shear connector.

CHAPTER TWO

LITERATURE REVIEW

2.1. General

This chapter deals with details of the shear connection in composite slab and I-steel beams with re-entrant and open trough profiled sheeting and review of previous research related to experimental studies as a control on the performance of connector in composite structures with profiled sheeting. In construction of steel-concrete the bond must be achieved by the ties of steel member by using a mechanical device known as shear connector. The bonding is more important in steel-concrete composite because it resist shear as well as helps to prevent separation of the structures. So, the shear connector is best solution for the bonding of composite structures. The demountable headed stud connector is most common type of shear connector used in composite structures for transferring shear force at the interface between steel and concrete.

2.2 shear connection

The shear connector is basically used to tie the concrete slab to the steel beam to transfer the horizontal shear between the slab and the beam without slip and at the same time to prevent the vertical separation of the slab from the structural steel member at the inner face. Shear connector are probably most common type of solution of bonding steel elements and concrete elements. composite action achieved between the concrete elements and steel element by using shear connectors, thus increasing both stiffness and strength. Shear connectors prevent slip between the components and achieves the much stiffer and stronger beam. Therefore, shear connectors are to be designed to cater for integral action of the composite structure at all load condition on the transmission of longitudinal shear along the contact surfaces without slip and prevention of vertical separation of the in-situ RC slab from the pre-fabricated structural beam.

2.2.1 headed stud shear connector

According to journal [2] headed stud shear connector is flexible type of shear connectors and is most common type of shear stud connector to resist longitudinal shear stresses between steel-concrete composite elements. Head of stud a resist the separation forces and prevents separation between the concrete slab steel beam crushing failures. Thus, advantages are: Very high rate of production can be achieved during construction, Ease of operation during construction and also no specific skill is required to installation, Shear connector have high load bearing capacity and offering heavy resistance for failure by shearing, flexibility in design of construction and easy to design of shear stud connector

2.2.2. Demountable headed stud shear connector

In this research study to investigate the load slip behavior of demountable shear connector in composite structures with a profiled metal decking composite concrete slab. This research is focused on using the new demountable shear connector without an embedded nut as the connector is shaped to hold the profiled metal deck to the steel beam.

A novel demountable shear connector is the connector uses for high strength steel bolts which are fastened to the top flange of the steel beam with the aid of special locking nut configuration that prevents bolts from slipping within theirs holes. Moreover, the connector promotes accelerated construction and overcomes the typical construction tolerances issues of structures. The more advantages of this connectors are, the precast decks panels can be rapidly uplifted and replaced, connectors can be rapidly removed and replaced and steel beam can be replaced, whereas precast decks and shear connectors can be reused. The experiment results in [1] shows that the shear resistance, stiffness and slip capacity significantly higher than those of other shear studs along with superior stiffness a strength against slab uplift. According to this experimental study, all behavior was study on shear strength, stiffness and ductility of demountable shear connectors in metal decking composite slabs through push-off tests.

Twelve full-scale push-off tests were carried out using different concrete strength, number of connectors and different connector diameter. The experimental results showed that the demountable shear connectors in metal decking composite slabs have similar shear capacity and behavior as welded shear studs and fulfilled the minimum ductility requirement of 6mm required by Eurocode 4[3].As this paper the diameter of connector putted as 19mm and 22mm in diameter of the connector with open trough profile steel sheet with thickness of 0.9mm was used.

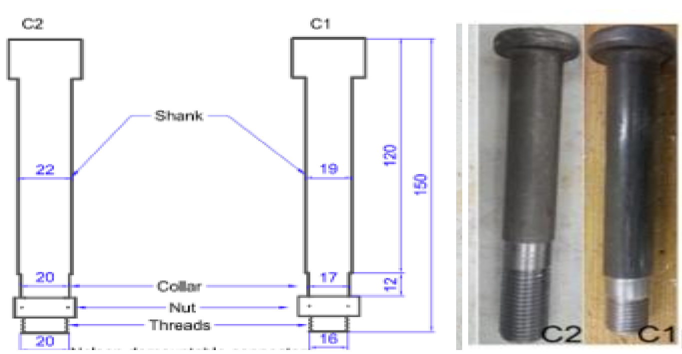


Figure 2. 1 Different type of demountable connectors [3]

It consists of two identical concrete slabs of size 610 ×510 × 150 mm connected through shear connectors as shown in Figures 4 and 5 with a predrilled hole in a steel section (203 × 203 × 52 UB)[4]. To assess the shear capacity, stiffness and ductility of demountable shear connectors, a series of push-off tests were carried out at that University.

Figure 6 shows the test setup of the push-off test. Eight linear variable displacement transducers (LVDTs) were installed at the top of the steel beam and concrete slabs, as shown in Figure 7 to measure the vertical displacements, which subsequently used to calculate the relative slip between the steel beam and the concrete slab. The load versus displacement was recorded by the data logging system. During the test, the applied load was increased by 5kN interval; at each interval, a further 5 minutes between loading is allowed for the load to settle before the next load increment is applied. When the applied load reached 40% of the predicted failure load based on the Eurocode 4 equations, then the load-control method was changed to the displacement-control method, in which a constant increment rate of 0.2mm/min was adopted until the failure of the specimens was observed, i.e., rapid reduction of the load capacity.

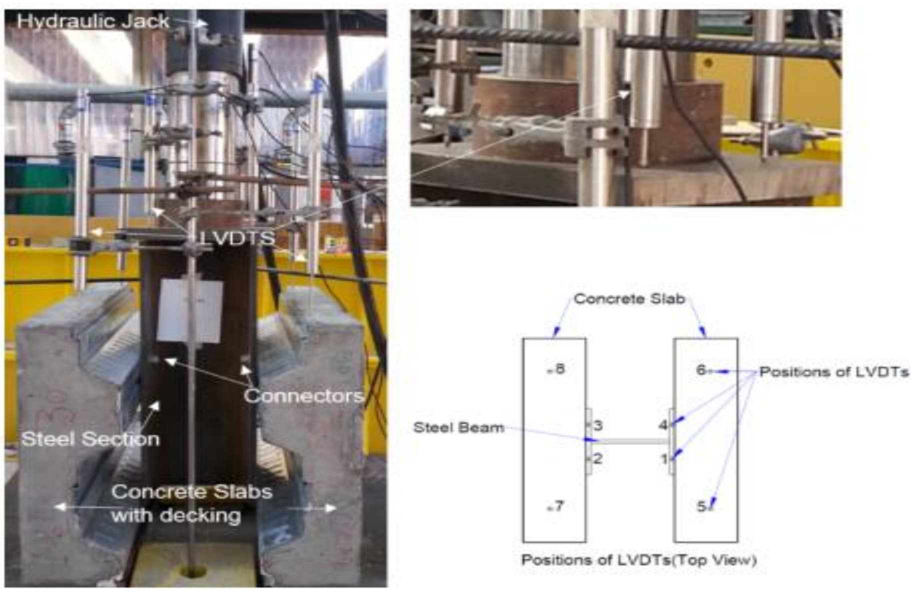


Figure 2. 2 Push-off test set up and monitoring positions [3]

According to the set-up position of push-off test the transverse spacing between two connectors was 100mm in specimens with two studs per trough. The minimum distance between the shear connectors and the vertical reinforcement cage bar was 50mm. The ultimate strengths of the shear stud connectors and steel reinforcement are 510N/mm^2 and 610 N/mm^2 respectively. Richard Lees Rib E60 type profiled metal decking with a thickness of 0.9 mm was used with the steel grade of

S350.As per the experimental results of the test two main failure modes were observed in these tests. The first one crushing of concrete which start from the head of the connector and crushed all surfaces of cone concrete faces. The second modes of failures were formed on the face of connectors. The failures of connector formed as deformed shapes or as fracture of the face of the shear stud connectors. According to paper[5] described that as the strength of concrete increases, the bearing capacity and stiffness of the connectors also increases. As the diameter of the connector increases, the bearing capacity and stiffness of the composite slab increases and with a constant diameter, the higher connector results in a greater bearing capacity and stiffness. Changing the thickness of the profiled steel sheeting does not significantly affect the bearing capacity of connectors. The increasing horizontal spacing between connectors does not significantly impact the behavior of connectors.

2.2.2.1 Previous studies on demountable headed stud shear connector

In the thesis done by [6] An innovative type of shear connector consists of a coupler and a bolt which are embedded in the prefabricated concrete deck. The assembly of the concrete deck with the flange of the steel section which has oversized holes is achieved through resin injected bolts. Resin injected bolts are bolts in which the cavity formed by the clearance between the bolt and the hole is filled up with resin. Large hole clearances allow for fabrication tolerances and lead to a faster execution. Push-out tests were conducted in the laboratory in order to examine resin injected bolts in terms of shear capacity, stiffness and ductility. Two different test configurations were created, one with resin injected bolts and the other one with reinforced resin injected bolts. For each configuration three specimens were tested which were nominally identical, one was loaded until failure using displacement control and the other two were loaded initially in force-controlled load cycles and then until failure. The results obtained from the experiments are compared with the results from researches conducted on other types of demountable shear connectors. FEA models were developed with the same geometry, materials and loading as in experiments using the ABAQUS software and push-out test were performed in order to check the validity of the experimental work. In addition, a parametric study was conducted using FEA in order to evaluate the influence of certain parameters on shear resistance and stiffness. The parameters considered are: the concrete strength class, the bolt diameter, the bolt strength class, the embedded bolt height, the hole diameter of the steel section, the effect of the L angle profile and the injection material

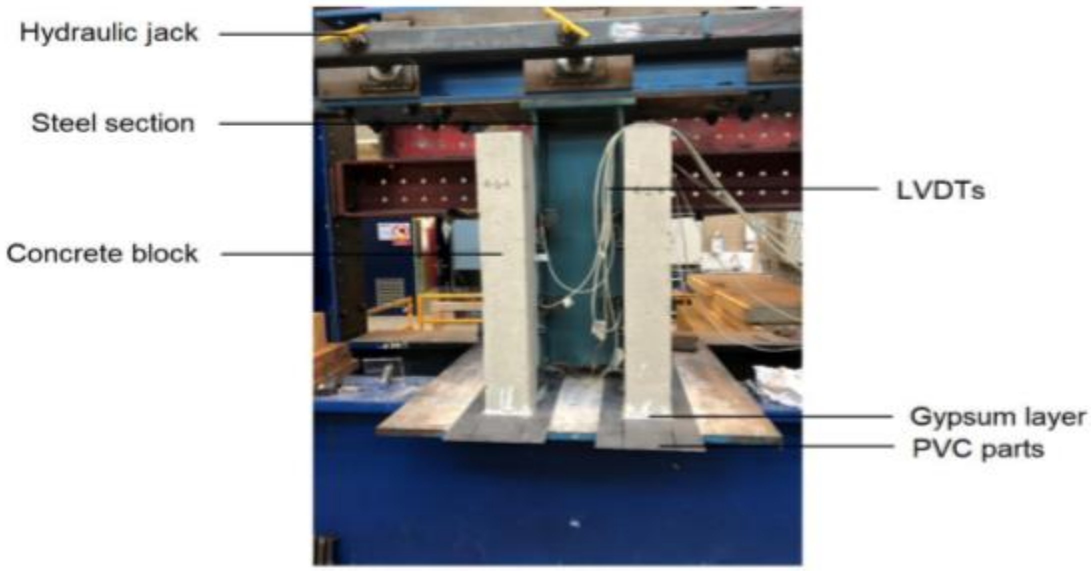


Figure 2. 3 Complete set-up [1]

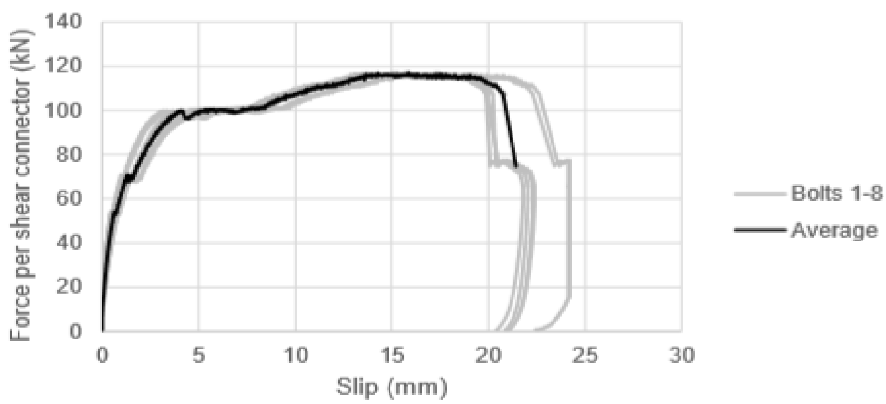


Figure 2. 4 Force-slip curve of shear connectors of test [1]

2.2.3. Welded shear Connectors

On the paper of [7] There have been a number of different types of mechanical shear connectors developed to replace welded headed studs since the 1930s including the bar, spiral, T section, perforate rib shear connectors, H shape and channel connectors. Before the invention of profiled metal decking, the channels shear connectors were used to a large extent in composite beams. However, the headed shear connectors are the most popular and commonly used in today's construction industry due to the through deck welding process and available design rules for welded headed shear connectors. Also, they prevent the uplifting of the composite slab and resists the shear force equally in all directions due to its circular shape. A typical head shear connector can be seen in Figures -7 it is welded to the top flange of the steel beam with profiled metal decking.

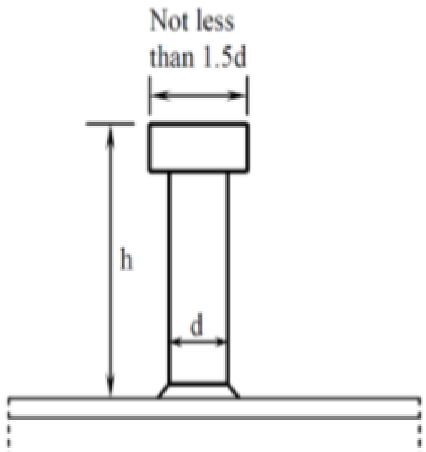


Figure 2. 5 Welded headed shear connector [7]

These headed shear connectors are available in different diameters ranging from 13-25 mm and lengths ranging from 50-250 mm. According to Eurocode 4, the minimum ultimate tensile strength should be 450 MPa, they should have an elongation of 15% and the head diameter should be 1.5 times the shank diameter (d).

2.2.3.1. Previous studies on welded shear connector

On this studies, [8]conducted 44 specimen of Accurate nonlinear finite element models have been developed to investigate the behavior of shear connection in composite beams with profiled steel sheeting perpendicular to the steel beam. The models consider the nonlinear material properties of the concrete, steel beam, profiled steel sheeting, reinforcement bars and headed stud shear connectors. The capacity of shear connection, load–slip behavior of headed stud and failure modes were predicted from the finite element analysis and compared well with experimental results. An extensive parametric study of 44 push-out specimens with different profiled steel sheeting geometries, headed shear stud diameters and heights as well as concrete strengths was performed using the finite element models. The comparison of shear connection capacities obtained from the finite element analysis and the design rules specified in the American Specification, British Standard and European Code have shown that, the American and British specifications overestimated the capacity of shear connection with a maximum value of 27% and 25%, respectively. The design rules specified in the European Code were generally conservative, except for some cases that overestimated the capacity of shear connection with a maximum value of 11%.

On the other hand [9] study about the behavior of headed stud shear connectors in composite beams with trapezoidal profiled sheeting laid transverse to the axis of the beam has been studied through experimental and numerical investigations. The separation of the steel deck from the concrete slab, which helped in accurate determination of failure modes. Although, the finite element model developed in this study predicted well the maximum failure load, slip at failure and failure mechanisms of push tests, it overestimated the ductility of the shear connector beyond peak load. The application of normal load of 10% of the horizontal shear load on top surface of the concrete slab in a single-sided horizontal push testing arrangement, in addition to the horizontal shear load, increased the strength of single and double shear studs by 40% and 23% respectively with no significant effect on the ductility of the shear connector. Similarly, using double layers of mesh resulted in 18% increase in the shear connector resistance as compared with a single layer of mesh, while no improvement in the ductility was observed with the use of double layers of mesh.

On the paper of [10]identify on the paper of Composite Slab with Profiled Steel Deck. According theirs study conclusion, Code provisions are based on experimental investigations the finite element analysis of composite deck slab with nonlinear contacts between the profile deck, shear connectors and concrete.

According paper done by [11] The experimental results demonstrate that the ultimate shear capacity per stud in the push-out specimens increased with the diameter and yield strength of the studs. According to the results when the stud diameter increases from 16 mm to 19 mm, the ultimate shear capacity per stud increases by about 21%. The ultimate shear capacity per stud increased by about 11% when the yield strength of studs increases from 365 MPa to 410 MPa, or when the concrete strength increased from 39.1 MPa to 46.5 MPa. The simplified modeling approach developed in the current study predicted the load displacement responses of steel−concrete composite beams and compared well with experimental scenarios from the literature.

In paper of [12] Present knowledge of the load–slip behavior and the shear capacity of the shear stud in composite beam are limited to data obtained from the experimental push-off tests. For this purpose, an effective numerical model using the finite element method to simulate the push-off test was proposed. The model has been validated against test results and compared with data given in the current Code of Practices, i.e., BS5950, EC4, and AISC. Parametric studies using this model were preformed to investigate variations in concrete strength and shear stud diameter. The finite element model provided a better understanding to the different modes of failure observed during experimental testing and hence shear capacity of headed shear studs in solid concrete slabs.

On other hand the[13] state that the awareness of the structural engineering community to the concepts behind composite steel–concrete structural design for fire exposure. The behavior of reinforced concrete slabs under fire conditions strongly depends on the interaction of the slabs with the surrounding elements which include the structural steel beam, steel reinforcing and shear connectors. This study was carried out to consider the effects of elevated temperatures on the behavior of composite steel–concrete beams for both solid and profiled steel sheeting slabs. This investigation considers the load–slip relationship and ultimate load behavior for push tests with a three-dimensional on-linear finite element program ABAQUS.As a result of elevated temperatures, the material properties change with temperature. The studies were compared with experimental tests under both ambient and elevated temperatures. Furthermore, for the elevated temperature study, the models were loaded progressively up to the ultimate load to illustrate the capability of the structure to withstand load during a fire. It is concluded that finite element analysis showed that the shear connector strength under fire exposure was very sensitive. It is also shown that profiled steel sheeting slabs exhibit greater fire resistance when compared with that of a solid slab as a function of their ambient temperature strength

2.3. Push test technique

Most of the research carried out on welded shear connectors using the technique of push tests. Eurocode 4 (2004) provides a simple procedure for push tests and equations to predict the shear capacity of welded shear connectors in composite beams. However, the push test details provided in Eurocode 4 are for welded shear connectors in solid concrete slabs. The push test technique is well established and used to determine the shear capacity of a shear connector. It is used as a substitute for a full-scale composite beam test to reduce the time and cost related to a full-scale test. A standard push test is described in [3] for welded shear connectors and its layout are presented in Figure 2-6. The push test specimen consists of two identical concrete slabs attached to a steel beam. The slabs should be casted in a horizontal position and cured in open air. The specimen is loaded vertically downward until it fails.

However, this standard push test is not designed for specimens with a profiled metal decking and there is no guidance available in any code of practice for testing composite concrete slabs with demountable shear connectors. The performance of composite concrete slabs with profiled metal decking is checked by using empirical equations as described in Eurocode 4. The characteristic resistance P_{RK} can be taken as the minimum failure load per shear connector from three nominal identical push tests and reduced by 10% according to EC4.

Ductility is another major factor in composite construction. The ductility of a shear connector can also be determined from push tests, this depends on the slip capacity at the interface of the steel beam and the composite concrete slab. The slip capacity is the maximum slip measured at the characteristic load level as shown in Figure 2-7. Shear connectors are considered to be ductile if the connectors have sufficient deformation capacity. According to Eurocode 4, the shear connector will be considered to be ductile if the slip reaches 6 mm. A large plastic deformation of the headed shear connector is characterized as good ductile behavior. On the other hand, if the shear connector does not fulfil the limit of 6mm it is characterized as brittle behavior of the headed shear connector with very little plastic deformation.

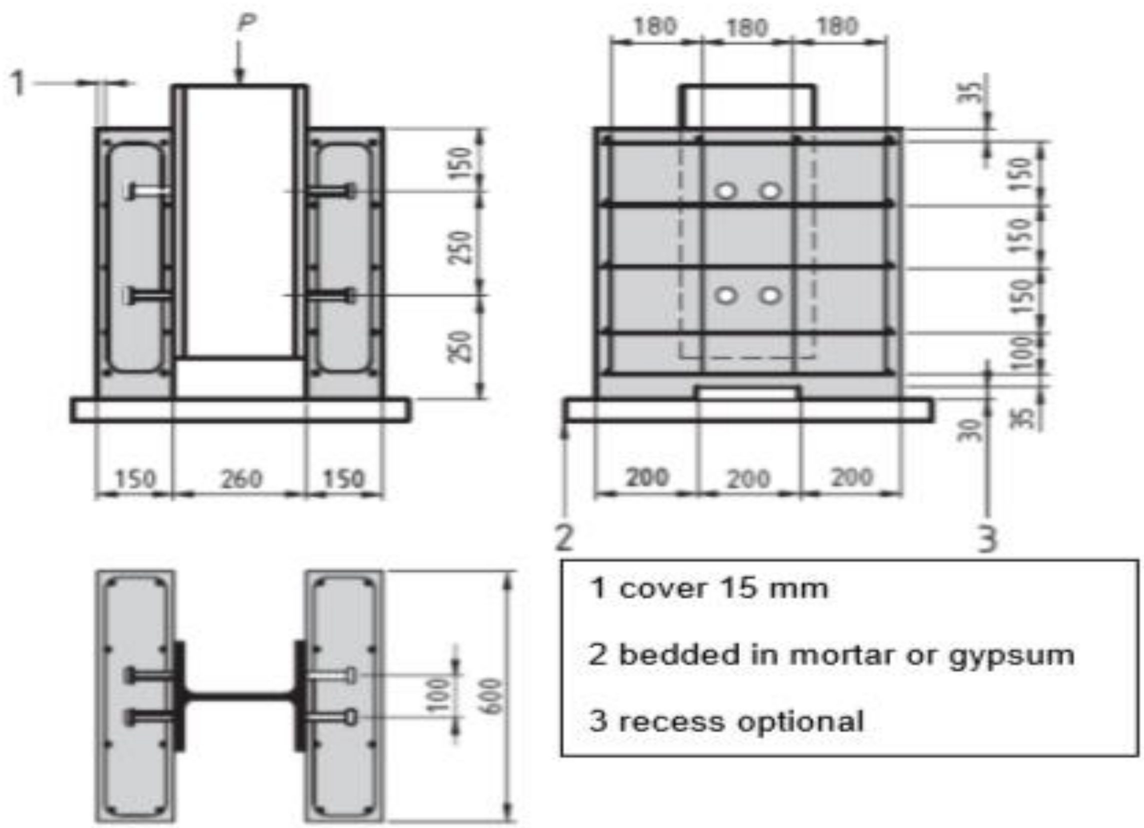


Figure 2. 6 General Arrangements of standard push test in Eurocode 4

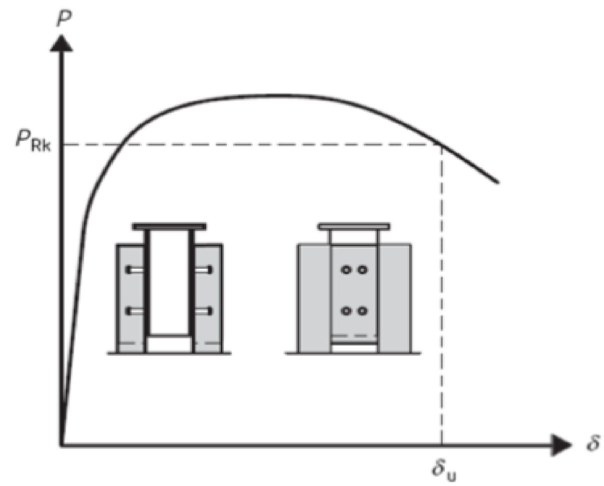


Figure 2. 6 Determination of slip capacity (δ) according to Eurocode 4[3]

2.4. Determination of shear capacity of shear connector using different codes

There is no design guide available for demountable shear connectors in a composite construction. Therefore, the design equations available for welded shear connectors are considered for demountable shear connectors. A short overview of some of the available design codes for the shear strength of welded shear connectors in composite construction is presented in this section.

The strength of the shear connection is mainly influenced by shank diameter, height and tensile strength of the connector, compressive strength and the elastic modulus of concrete. The shear forces are resisted by bending, tension or shearing at the root of the shear connection. The connector's root transmits the horizontal shear forces acting at the interface, while the head prevents the uplifting of the slab. The plastic deformation occurs after reaching the ultimate strength of the stud.

2.4.2 Eurocode 4

Eurocode 4 provides two different equations one for solid concrete slab and one equation for using metal profiled decking.

2.4.2.1. Equations for solid slab

In Europe, the provisions for composite construction as part of Eurocode were included in the 1990s and followed by the issuing of Euro-code 4 more recently. BS EN 1994-1-1:2004 the same with Eurocode 4 clause 6.6.3.1, provides two equations as described in equations 2-1 and 2-2 for shear resistance of welded headed shear connectors in solid slabs. The design resistance (P_{Rd}) should be determined from the minimum value from these two equations. These two equations are related to two main failure modes, concrete cone failure and shear connector failure.

$$P_{Rd} = \frac{0.29\alpha d^2\sqrt{f_{ck}E_c}}{\gamma_v} \qquad 2.1$$

$$P_{Rd} = \frac{0.8f_u\pi d^2}{4\gamma_v} \qquad 2.2$$

Where; - $\alpha = 0.2\left(\frac{h_{sc}}{d}+1\right) \leq 1.0, \text{for } 3 \leq \frac{h_{sc}}{d} \leq d$ and $\alpha = 1, \text{for } \frac{h_{sc}}{d} > 4$

d = is the shear connector shank diameter in mm

h_{sc} = is the shear connector height f_u = is the shear connector ultimate tensile strength

f_{ck} = is the characteristic cylinder compressive E_c = is modulus of elasticity of concrete

γ_v = is the partial safety factor for shear resistance (γv = 1.25)

2.4.2.2. Equations for profiled metal Deck as Eurocode 4

The shear strength of a shear connector embedded in a profiled metal deck composite concrete slab is based on a reduction factor (Kt) and used with the shear strength of a shear connector in a solid concrete slab as presented in equation 2.4.

$$P_r = K_t P_{Rd} \qquad 2.3$$

$$\text{Kt} = \frac{0.6}{\sqrt{n_r}}\frac{b_o}{h_p}\left[\frac{h_{sc}}{h_p} - 1\right] \leq 1.0 \qquad 2.4$$

$$K_t = \frac{0.7}{\sqrt{n_r}}\frac{b_o}{h_p}\left[\frac{h_{sc}}{h_p} - 1\right] \leq 1.0 \qquad 2.5$$

The reduction factor (Kt,) depends on the direction of metal profile decking laydown at the steel flange. Either it is parallel or transverse to the direction of the steel beam as illustrated in Figures 2-9 and 2-10 and in equations 2-3 and 2-5 respectively.

b_o = is effective width of the slab

n_r = is the number of stud connectors in one rib at a beam intersection, not to exceed 2 in computations. h_p= is the overall depth of sheeting excluding embossment

h_{sc}= is the overall height of the stud, but not greater than h_p + 75 mm

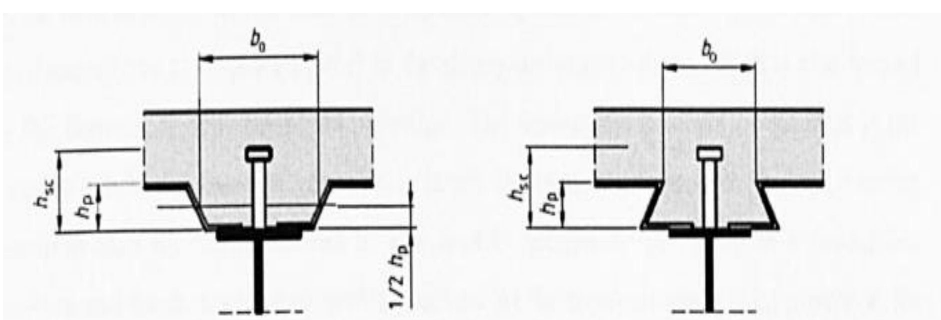


Figure 2. 7 Composite beam with metal profile decking parallel to steel beam [14]

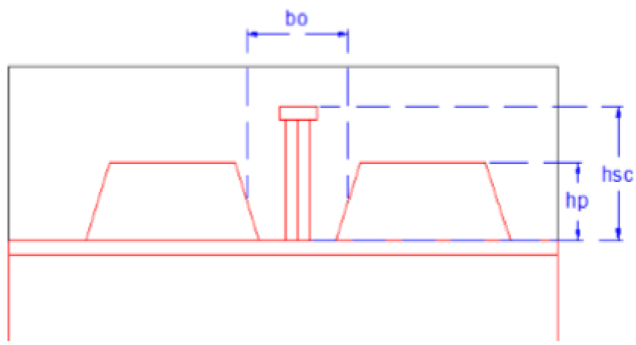


Figure 2. 8 Composite beam with metal profile decking transverse to steel beam [14]

Noted that the cross sectional dimension of this research were taken from the [14] steel factory thus made the steel class according to its functionality. According to paper on[15]The longitudinal shear bond strength between the concrete and steel deck is evaluated analytically using m-k and partial shear connection (PSC) methods and compared the values. The experimental results are verified and compared with the results of both m-k and PSC methods. Comparison of experimental and analytical results of the load-carrying capacity of composite slabs revealed that agreements between these values are sufficiently good. As a result, m-k method proved to be more conservative than PSC method.

2.5 Role of dilatancy angle in Concrete damage plasticity

The concrete plasticity models in Abaqus provides as a general capability for modelling concrete and other quasi brittle materials in all types of structures (beams, trusses, shells and solids).it used to simulate the behaviors of concrete. concrete damage plasticity components like as dilation angle, eccentricity, fbo/fco, Kc and viscosity parameter are used to compute the behavior and strength of materials in finite element analysis. Abaqus manual [15] under the section of DRUCKER-PRAGER/CAP PLASTICITY MODEL putted that the value of thus plastic parameter. According to this manual the values of the plastic parameter were depend on the material types and its contents. The dilatancy angle is the constant of the Mohr-coulomb (MC) models,Ψ, that define the plastic volumetric strains. As per models in [16] indicates that the roles of this angle in the plastic potential function is analogous to the roles of the friction angle in the yield function. In the program Abaqus the dilation angle of internal friction of the material and values that oscillated between 30° and 40° are recommended by [17].

CHAPTER THREE

RESEARCH METHODOLOGY

3.1 General

In this paper, a total specimen of 24 and one additional control models were created and analyzed in ABAQUS software to determine the shear capacity and failure modes of demountable shear connectors in composite slab and steel I-section beam with re-entrant and open trough profile steel sheeting. The headed and embedment surface of the demountable stud connector were interacted with slab against uplift force and the nodes of collar were tie to profile steel sheeting. The surfaces of the collar were installed with steel I-beam and at the top of the beam there was a displacement loaded which is done against all degree of freedom and also there were castrated or fixed at bottom surface of the slab with restricted the symmetry surface of slab and beam web to resist the movement during displacement applied at top face of beam. In this section of the report; the study variables, sample detailing, Finite Element modeling and analysis steps of the selected demountable shear connector with composite structures were discussed in detail.

3.2 Research design

Non-linear finite element analysis of demountable shear connection in composite slab and steel beam with profiled steel sheet is performed using ABAQUS/standard explicit. Geometry and material non linearities are considered in order to properly simulate the strength of shear connection to analyze the performance of shear connector under applied displacement dynamically. In the preprocessing stage of ABAQUS, geometry, boundary condition, element type, material properties and non-linear analysis solution were defined. Concrete damage plasticity (CDP) was selected and introduced to the numerical models. The core and essential element of model based on plasticity theory which were the yield criteria, flow and hardening rules were all effectively considered in damage plasticity models. The numerical modelling of shear connection, concrete slab and all steel part those present in the model material property data were calculated by using the numerical formula discussed in next section.in simulation stage, which normally runs as a background process, that means in which ABAQUS solves the numerical problem defined in models. It is also a stage were outputs obtained are stored in binary files. finally, at last stage or at post processing stage the result generated in the analysis are visualized once the simulation completed.

3.3. Study Variables

The study variables both dependent and independent were assess in this research which displays the performance of shear connection in composite slab and steel I-beam with re-entrant and open trough profile steel sheeting.

3.3.1. Dependent variable

The dependent variable of this study was:

- Ultimate strength of shear connection in composite slab and steel I-beam with re-entrant and open trough profile steel sheeting.

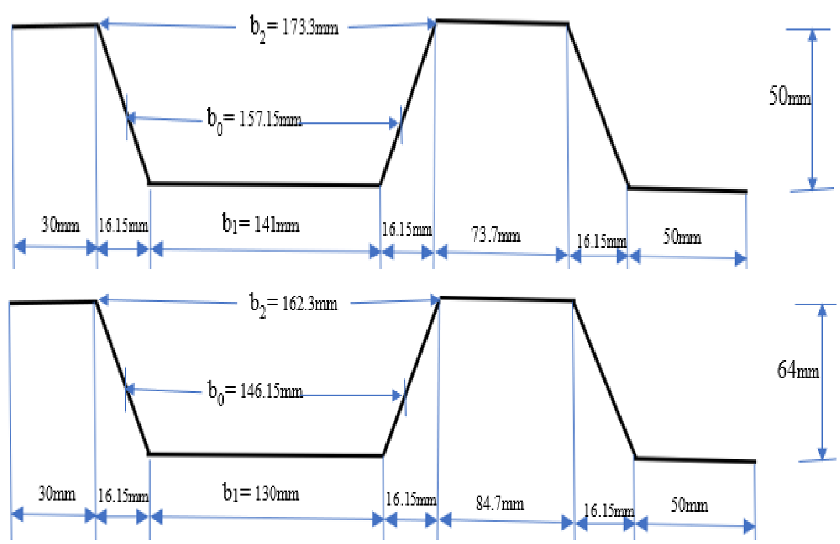
3.3.2. Independent variables

The independent variables include:

- Shear connector diameter
- Geometries of Re-entrant and open trough profile steel sheeting
- Strength of concrete

3.4. Sample Detailing

In order to study the strength of headed stud demountable shear connector, the dimension of studs, strength of concrete, dimension of slab and steel beam and profile steel sheeting profile were the parameters investigated throughout this study. According to the Figure below the dimension of both open profile steel sheeting (OTPSS) and re-entrant trough profile steel sheeting (RETPSS) were conducted as parametric study as Figure 3.1 as per present in [10] and also the detail of the all data were described in Table 3.1. Totally the thickness of all sheet in the specimen were 1.0mm and different by height of sheet(h_p)and width of rib(b_o) as tabulated in table.



a)

b)

c)

d)

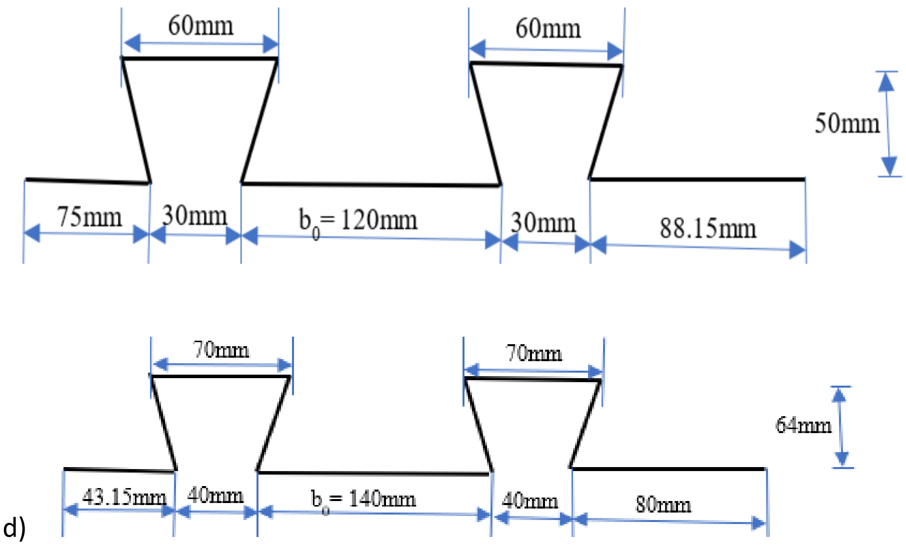


Figure 3. 1 Definition of symbols for profiled steel sheeting

Table 3. 1. Dimensions and concrete strength of parametric study

Group	Specimen	Dimension							Concrete strength (Mpa)
		Sheeting			Stud		Slab		
		Type	b_o(mm)	h_p(mm)	d(mm)	h(mm)	B(mm)	D(mm)	
G1	sp1	(a)	157.15	50	19	110	343.15	150	40
	sp2	(a)	157.15	50	19	110	343.15	150	50
	sp3	(a)	157.15	50	19	110	343.15	150	60
G2	sp4	(a)	157.15	50	22	110	343.15	150	40
	sp5	(a)	157.15	50	22	110	343.15	150	50
	sp6	(a)	157.15	50	22	110	343.15	150	60
G3	sp7	(b)	146.15	64	19	110	343.15	150	40
	sp8	(b)	146.15	64	19	110	343.15	150	50
	sp9	(b)	146.15	64	19	110	343.15	150	60
G4	sp10	(b)	146.15	64	22	110	343.15	150	40
	sp11	(b)	146.15	64	22	110	343.15	150	50
	sp12	(b)	146.15	64	22	110	343.15	150	60
G5	sp13	(c)	120	50	19	110	343.15	150	40
	sp14	(c)	120	50	19	110	343.15	150	50

	sp15	(c)	120	50	19	110	343.15	150	60
G6	sp16	(c)	120	50	22	110	343.15	150	40
	sp17	(c)	120	50	22	110	343.15	150	50
	sp18	(c)	120	50	22	110	343.15	150	60
G7	sp19	(d)	140	64	19	110	343.15	150	40
	sp20	(d)	140	64	19	110	343.15	150	50
	sp21	(d)	140	64	19	110	343.15	150	60
G8	sp22	(d)	140	64	22	110	343.15	150	40
	sp23	(d)	140	64	22	110	343.15	150	50
	sp24	(d)	140	64	22	110	343.15	150	60

According input data taken from push off test the cross-sectional properties of steel I section beam was presented in the[4] 203x203x52 universal beam. was taken and also 343.15x150mm for composite slab were conducted from [16].

3.5. Non-linear Finite element analysis

The finite element method is a technique for approximating the governing different equation for a continuous system with a set of algebraic equations relating a finite number of variables. These methods are popular because they can be easily programmed the FE techniques problems. In this study, the finite element program ABAQUS [17]was used to investigate the behavior of demountable shear connection in composite beams with profiled steel sheeting. In order to obtain accurate results from the finite element analysis, all components associated with the demountable shear connection must be properly modeled. The main components affecting the behavior of demountable shear connection in composite beams with profiled steel sheeting are concrete slab, steel beam, profiled steel sheeting, reinforcement bars and demountable shear connector. Both geometric and material nonlinearity are included in the finite element analysis. Experimental tests are needed to provide input data to the model and for the purpose of verification of simulation results. When the model has been validated it can be used for parametric studies to investigate the influence of various parameters. The modeling and analysis procedure were described as shown below. ABAQUS (2014) is a general-purpose finite element programmed and it may be used to simulate the push off test. The dynamic explicit analysis approach was used in this study only quarter of the geometry of the push off test was created due to the symmetrical conditions across the center line of the web of the steel beam to save computational time. The main components of the FE model included the profiled metal decking, concrete slab, steel beam, reinforcement steel cage and demountable shear connector as shown in below in detailed. All components were modelled separately and assembled together to form one quarter of the push off test specimen.

Note: all the specimens were modeled and analyzed after the FE software was validated for the specific case of this research.

3.5.1 Parts (geometry) modeling

3.5.1.1 concrete slab part

concrete slab parts were modeled on a 3D modeling space as deformable type with solid and extrusion base feature.

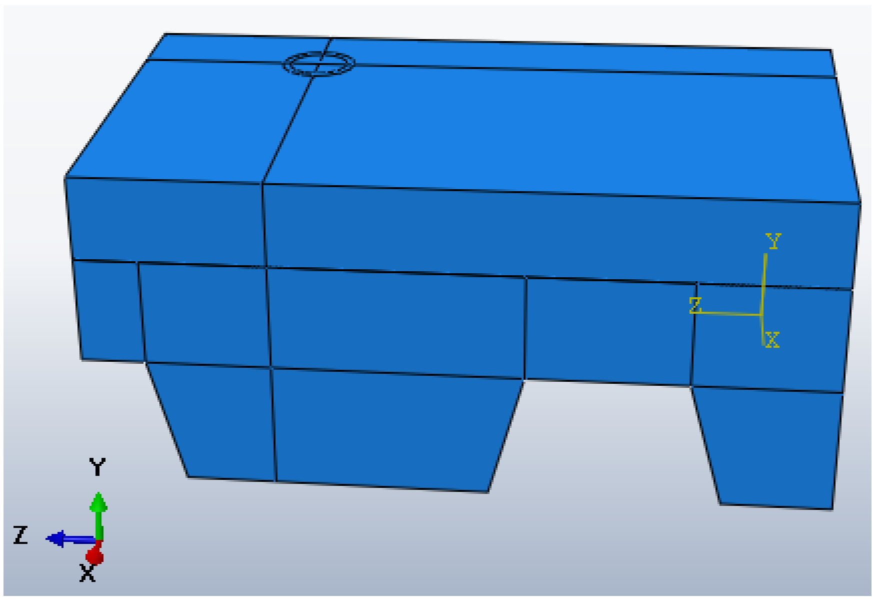


Figure 3. 2 Concrete slab part modeling as open trough shapes

3.5.1.2 steel I- section beam part

The steel I- section beam was modeled the same procedures with the concrete slab part in all case of modeling space means 3D, type as deformable and in basic features type extrusion and solid shapes.

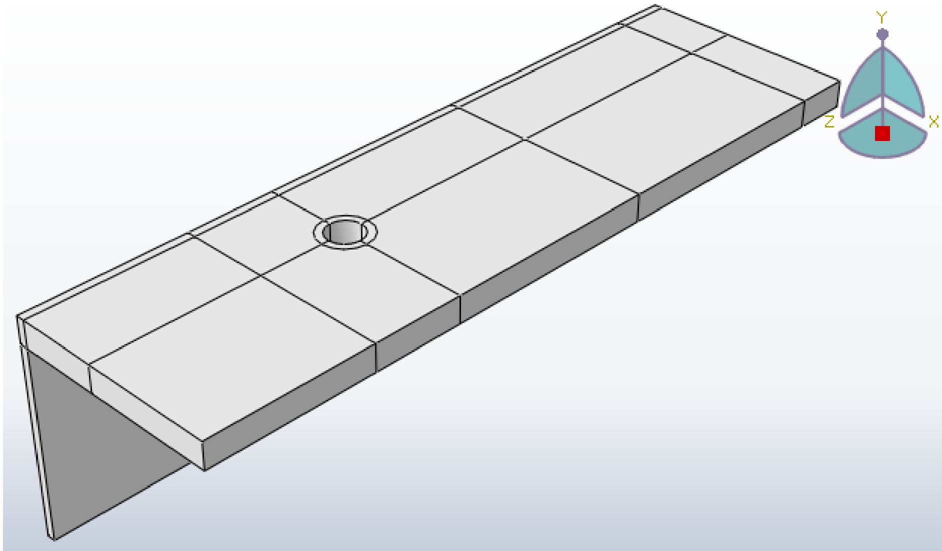


Figure 3. 3 Quarter of steel I-section beam part

3.5.1.3 demountable headed stud shear connector

This connector was modeled on a 3D modeling space as deformable type with solid and base feature as revolution (extrusion base feature creates solid extrude).

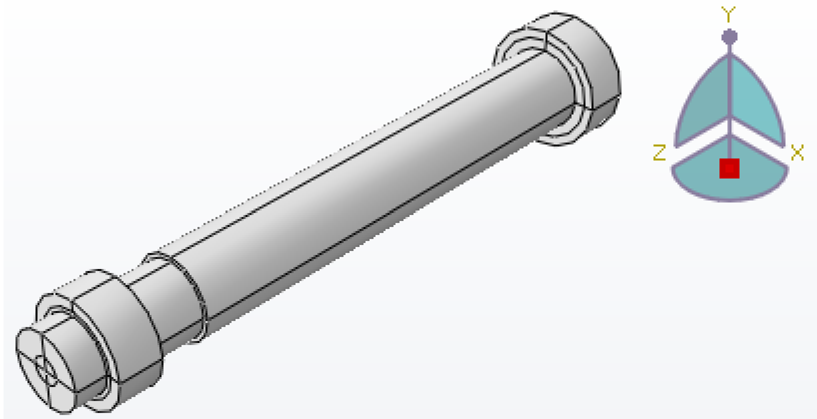


Figure 3. 4 Demountable headed stud shear connector part

3.5.1.4. Reinforcement cage part

Reinforcement cage parts were modeled on a 2D modeling space as deformable type with wire and planar base feature.

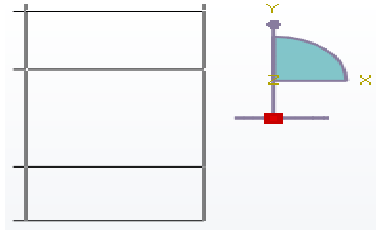


Figure 3. 5 Reinforcement cage part

3.5.1.5. profile steel sheeting part

Both re-entrant and open trough profile steel sheeting parts were modeled on a 3D modeling space as deformable type with shell and extrusion base feature.

a) b)

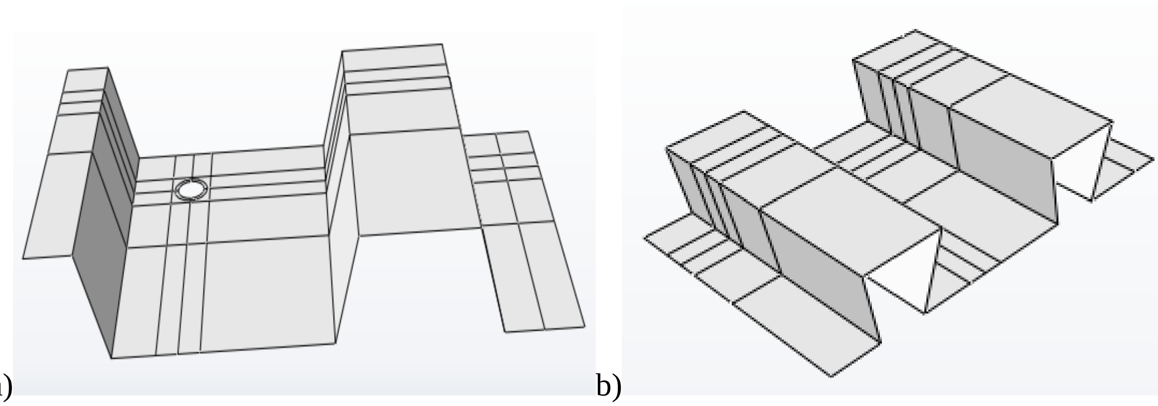


Figure 3. 6 Open trough (a) and Re-Entrant trough (b) Profile steel sheeting part

3.5.2. Material in Modelling

3.5.2.1. Material model of Concrete

The concrete model is a very important part of the simulation for the push off model, the concrete is mainly subjected to compression and tensile splitting as observed in the experimental study. It was observed that the failures were either due to concrete or connector shearing or a combination of sheared connectors and concrete cone failure. Therefore, a suitable concrete material model is very important for the accuracy of the FE model. In terms of the behavior of concrete, the concrete damaged plasticity model was used to define the concrete in this study. This concrete model is capable to model the concrete under arbitrary loading, including dynamic loading and assumes an isotropic damaged elasticity in tension and compression to present the inelastic behavior of concrete. The damaged plasticity model can be used for plain concrete as well as for RC structures subjected to monotonic, cycling and dynamic loading under low confining pressure (ABAQUS, 2014). Concrete damage plasticity model has been successfully used for the concrete slab by [18] in numerical modelling. The damage plasticity model may deal with two basic failure modes, compressive crushing and tensile cracking of concrete. In this study, the damage plasticity model was adopted and the maximum tensile strength of concrete was taken as 10% of its compressive strength.

Elastic behavior

The elasticity modulus E_{cm} used in the modelling was calculated based on ESEN PART 1-1, [19] method, as shown in equation 3-1. The poison's ratio for concrete was taken as 0.2 and density was considered as 2400 kg/m^3.

$$E_{cm}=22\left(\frac{f_{cm}}{10}\right)^{0.3} \tag{3-1}$$

Concrete Damaged Plasticity model

Compressive behavior

ESEN 2015-1-1[19] provides an equation for determining the compressive stress σ_c of concrete as shown in equation 3-2 under uniaxial compression. The stress strain relationship of concrete for nonlinear analysis is shown in Figure 3-3.

$$\frac{\sigma_C}{f_{cm}} = \frac{k\eta-\eta^2}{1+(k-2)\eta} \tag{3.2}$$

$$f_{cm} = f_{ck}+8 \tag{3.3}$$

σ_c = Compressive stress in the concrete

Where, f_{cm}, is the mean value of the concrete cylinder compressive strength and f_{ck}, is the characteristic compressive cylinder strength of the concrete.

$$k = \frac{1.05E_{cm}\,|\varepsilon_{c1}|}{f_{cm}} \tag{3.4}$$

$$\eta = \frac{\varepsilon_c}{\varepsilon_{c1}} \tag{3.5}$$

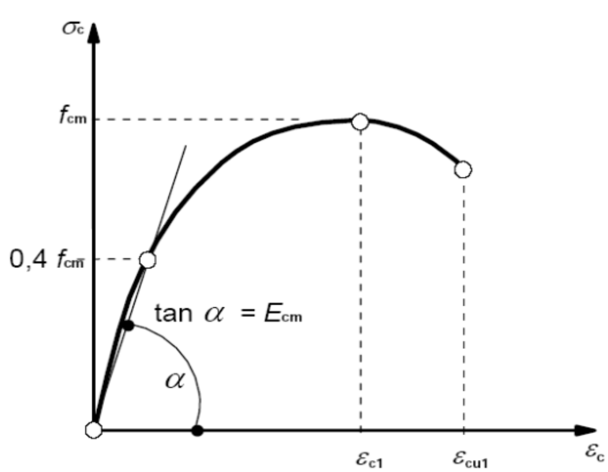


Figure 3. 7 Schematic representation of stress strain relationship [20]

The value of the strain at peak ε1 and nominal ultimate strain ε_{u1} can be taken as 0.0022 and 0.0035 respectively, according to ES EN 2015-1-1, for the compressive cylinder strength of 12 to 50MPa and also for high strength above of this value. The nominal ultimate strain $\varepsilon u1$ for concrete with a compressive strength greater than 50MPa can be calculated using the equation 3.6.

$$\varepsilon_c\,(\%100) = 2.8 + 27\left(\frac{98 - f_{cm}}{100}\right)^4 \tag{3.6}$$

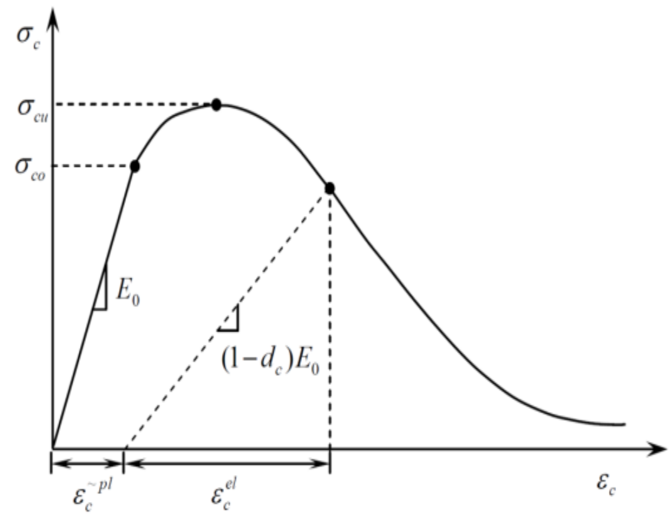


Figure 3. 8 Response of concrete to uniaxial loading in compression [21]

The response of concrete under uniaxial compressive loading is linear up to the initial yield stress σ_{co} as presented in Figure 3-8. After that concrete behavior becomes plastic with stress hardening and followed by strain softening beyond the peak compressive stress σ_{cu} as shown in Figure 3-8. In the strain softening branch, the elastic stiffness of the material appears to be damaged if it is unloaded at any point and is known as the compressive damage variable, dc. The value of zero for dc is considered as undamaged and 1 is fully compressive damaged. The value of dc can be worked out using the equation 3-7 and 3-8 provided in the ABAQUS manual (2014). If E_0 is taken as elastic stiffness for undamaged concrete and ε_c is taken as total compressive strain.

$$\sigma_c = (1 - d_c)E_0(\varepsilon_C - \varepsilon_c{}^{\sim pl} \tag{3.7}$$

$$\varepsilon_c{}^{\sim in} = \varepsilon_c{}^{\sim pl} + \frac{d_c\sigma_c}{(1-d_c)E_0} \tag{3.8}$$

$\varepsilon_C{}^{\sim pl}$ means the compressive plastic strain and $\varepsilon_c{}^{\sim in}$ the compressive inelastic strain

Tensile Behavior

The softening behavior of concrete can be expressed in a number of ways such as linear, bilinear and exponential. The ABAQUS manual provides a linear approach as shown in Figure 3.9 (a). [18]

developed a bilinear and an exponential relationship model for concrete softening behavior relationship as shown in Figure 3.9 (b) and (c) respectively.

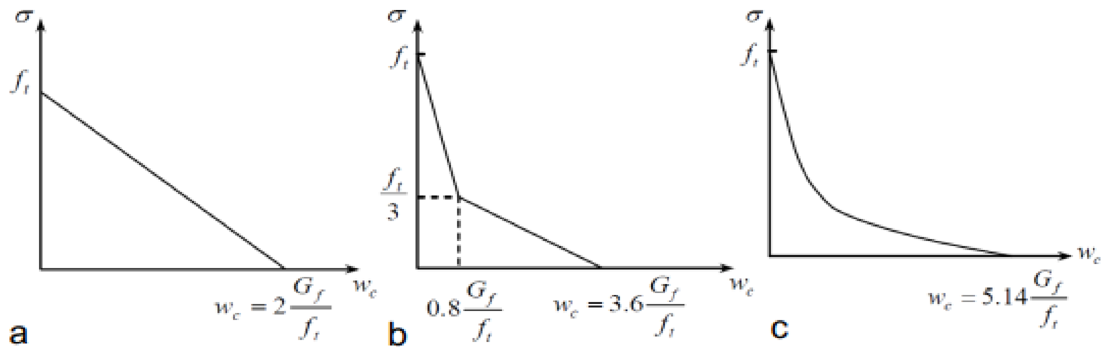


Figure 3. 9 Tension softening model (a) linear (b) Bilinear (C) Exponential

According to figure 3.9 putted in above, the axial tensile strength of concrete is worked out according to ES EN 2015-1-1 and the maximum tensile strength was taken as 10% of the compressive strength. The tensile damage variable d_t is worked out using the equation 3-9. The cracking displacement w is obtained from equation 3-10 using equation 3-11 to 3-13.

$$d_t = 1 - \frac{\sigma_t}{ft} \tag{3.9}$$

$$\frac{\sigma_c}{ft} = f_{(w)} - \frac{w}{w_c} f(w_c) \tag{3.10}$$

$$f(w) = \left[1 + \left(\frac{c_1 w}{w_c}\right)^3\right] exp\left(1 - \frac{c_2\, w}{w_c}\right) \tag{3.11}$$

$$w_c = 5.14 \frac{G_f}{ft} \tag{3.12}$$

$$G_f = 73[f_{cm}]^{0.18} \tag{3.13}$$

c_1, is taken as 3.0 and c_2 is 6.93 for concrete with normal density. w_c is crack displacement at fully damaged and no tensile stress can be transferred. The fracture energy G_f is determined from design code as expressed in equation 3-13

3.5.2.2 Plasticity parameters adopted

Five members need to be determined in order to use the concrete damage plasticity (CDP) model provided in the ABAQUS programmed. The dilation angle (ψ) which is defined as a material parameter that controls the plastic strain of concrete. It is also defined as the internal friction angle of concrete or the angle of inclination of the failure surface which evaluates the inclination of the plastic potential under high confining pressure. The dilation angle was iteratively calibrated during analysis to get the best results and it was found 36^0 giving the best results. The eccentricity defines the rate of hyperbolic flow potential and default value recommended by ABAQUS 2014 was used. The other parameter is the ratio of concrete strength in the biaxial state to that in the uniaxial state (σ_{bo}/σ_{co}). The value chosen for the proposed model is 1.2 which is the default value in ABAQUS. The viscosity parameter used for viscos-plastic regularization of concrete is very minimized nearest to ignored in this analysis. Table 3-2 summaries the five parameters.

Table 3. 2 Plasticity parameters

Dilation Angle	Eccentricity	σ_{bo}/σ_{co}	Kc	Viscosity Parameters
36° and 40°	0.1	1.2	0.6	0.001

where σbo/σco is the ratio of initial equi-biaxial compressive yield stress to initial uniaxial compressive yield stress, Kc is the ratio of the second stress invariant in tension to that in compression and there were two dilation angles as Table 3.2 tabulated in above, but in this paper all specimen were done by using 36° in addition to control. On other hand the 40° was used to check the gap between high sensitivity of concrete strength present in the variable of this study on the capacity of shear connection.

3.5.2.3. Material modelling of steel members

According to the Eurocode 2, the stress-strain relationship of steel starts with a linear elastic ascending branch up to the yield strength followed by a linear strain hardening up to the ultimate strength. In the current study, the mechanical properties of reinforcement steel bar, steel I-sectional beam, profiled metal decking and shear connectors are determined using coupon test and used to idealize the stress-strain relation shown in Figure 3-10. The modulus of elasticity (Es) is 210 GPa for all and 200GPa for reinforcement bar only. Three coupon tests were carried out to determine the mechanical properties of demountable shear connectors as described in section of [3]. The values of yield strength (f_y) and ultimate tensile strength (f_u) were used to model shear connectors for the FE modelling.

The steel beam was modelled using the yield strength of 610 MPa the same with demountable shear connectors taken from the manufacture's specification according to push off test in [1]. The effect of the steel beam was not very significant as observed during the push off tests. Therefore, the steel section was considered as linear elastic material with the young modulus of 210,000MPa and Poisson ratio of 0.3. The mechanical properties of the profiled metal decking were determined through the coupon test in which have been presented in section [1]. The measured values were used in the FE models Therefore, the profile steel sheeting section was considered as linear elastic material with the young modulus of 210,000MPa and Poisson ratio of 0.3 with yield strength of 450Mpa. but here, the yield strength of demountable shear connector was the same with steel beam structures.in the shear connector the whole parameter like as Ductile Damage for connector, Ductile Damage Evolution for connector, Damage evolution for ductility of connector, Shear Damage for connector with its evolution and damage evolution were determined.

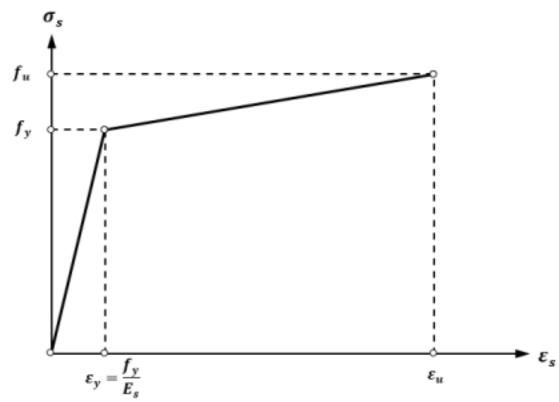
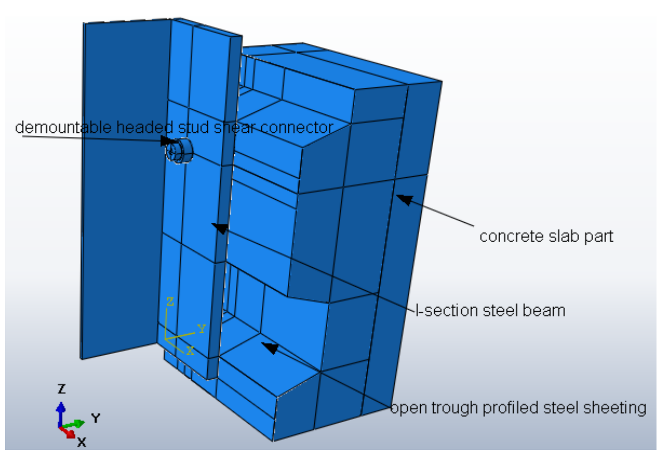


Figure 3. 10 Stress strain relationship of steel reinforcement (BS EN 1992-1-1:2002)

3.5.3. Part assembly in Modeling

After material properties, profiles and sections were created and assigned for the parts; instances were created for all parts and assembled to their relative position. Dependent instance was used for all parts. In these sections all parts were arranged on theirs position according to push off test set up to form one modeled. But the modeled created on this paper was the quarter modeled on the full push off test to minimize the amount of increment and time consuming during the analysis by ABAQUS software. Here there were two types of assembled model which differentiate by shapes of profiled steel sheet and concrete slab parts.

a)



b)

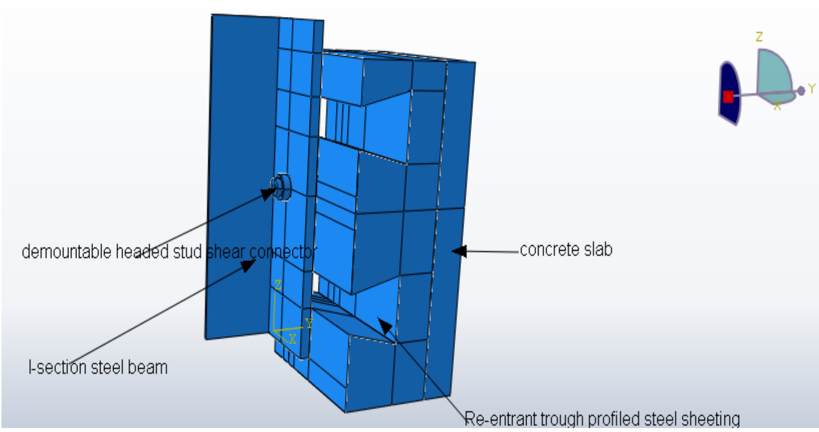


Figure 3. 11 Assembled all part with both[(a) and (b)] as shapes of profiled steel sheeting

3.5.4. Analysis step in modeling

In addition to initial step only one step was created. In order to avoid the convergence issues of the implicit static analysis, the dynamic explicit analysis was used to analyses the Finite Element models. ABAQUS/Explicit solves each problem as a wave propagation. In more dynamic explicit was used to avoid the unresolved element error during the analysis. In these paper there were many parts assembled together to form a model, from the divergent of elements there was a formation of file product error so in this analysis dynamic explicit step was sufficient in addition to initial steps.

3.5.5. Boundary conditions and interactions

In order to save computational time, one quarter of the push-out test specimen was built. Once all the parts of the push off test model are assembled together as shown in Figure 3.12 the appropriate contact interactions are defined between interacting surfaces of different components. The contact pair method is used to define the surface-to-surface contact between the concrete slab and the metal profiled decking. The normal behavior is assumed to be hard as this type of normal behavior allows minimum penetration of the salve surface into the master surface. The penalty method is used to define the tangential friction with a coefficient of 0.4 and 0.5 which give more reliable results as compared with the experimental results.

The contacts among the steel beam, demountable shear connector and nut was also defined using the contact pair method. The normal behavior is defined as hard contact and the tangential behavior is defined as penalty frictional with a coefficient of 0.50 and other which interact directly with shear connector were the same unless profile steel to demountable shear connector interacted as tied through nodal. The other part interaction define surface to surfaces were concrete slab to sheet,

sheet to steel beam by penalty method was used to define the tangential friction with a coefficient of 0.4.

All nodes of the concrete slab at the base are fully fixed(castrated) along Z directions as shown in Figure 3-12 to simulate the actual experimental boundary conditions. The symmetry conditions were applied on the web of the steel beam. The reinforcement steel cage truss elements are embedded inside the solid elements of concrete using embedded constraint as it is assumed that no slip or deboning occurred between the concrete and steel bars.

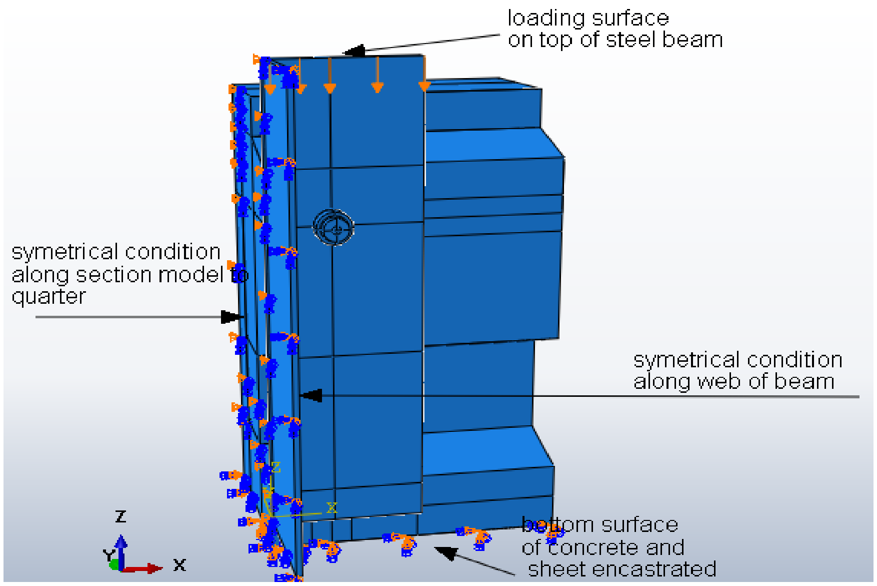


Figure 3. 12 Contact interaction and boundary conditions

3.5.6. Loading technique

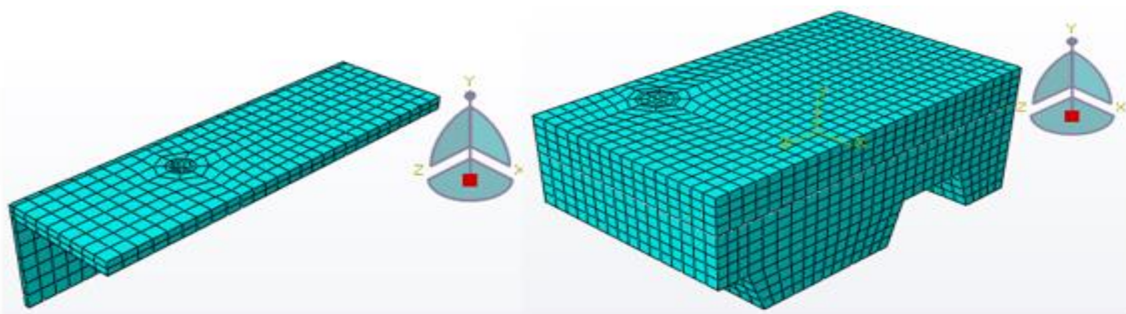
The push off FE model is analyzed using the dynamic explicit method, uniform velocity was used on the loading surface as shown in Figure 3-12 to push the steel beam downwards. The velocity rate of 0.20 mm/s was found to be the more appropriate velocity rate to simulate the push off model. That push off test used displacement control method used in the setup in addition to applied load techniques.in the specimen and control of this research paper the applied displacement was 9.5mm with 0.2sec time interval incrementation that taken from push off test done by[1] present as control. The total force applied to the specimen was calculated by summing up the vertical reaction forces for the loading surface.

3.5.7 Meshing

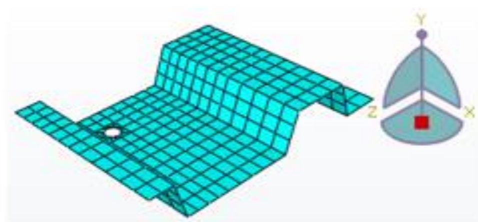
Different analysis was done in validation work using different mesh size until the analysis result was conforming to experimental result obtained from the journal. So, 12mm,12.5mm,12mm, 15mm and 30mm of mesh size for concrete slab, steel beam, profile steel sheeting, demountable shear connector and reinforcement bar respectively gave the best result which conforms to the result obtained from the journal and takes reasonable running time for the model. Since dependent instances were used and mesh was done for parts. In element type of the concrete slab, steel beam and demountable shear connector parts C3D8R were used which are 8 nodes linear brick, reduced integration hourglass control element and other were tabulated in Table below.

Table 3.3. Meshing size and element type

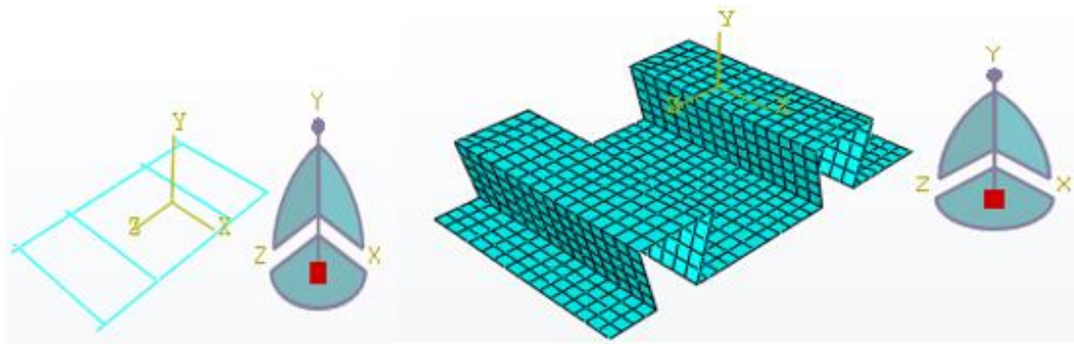
Parts	Mesh size (mm)	Element type
steel beam	12.5	C3D8R: An 8-node linear brick, reduced integration, hourglass control.
concrete slab	12	C3D8R: An 8-node linear brick, reduced integration, hourglass control.
profile steel sheeting	12	S4R: A 4-node doubly curved thin or thick shell, reduced integration, hourglass control, finite membrane strains.
demountable shear connector	15	C3D8R: An 8-node linear brick, reduced integration, hourglass control.
reinforcement	30	T3D2: A 2-node linear 3-D truss.



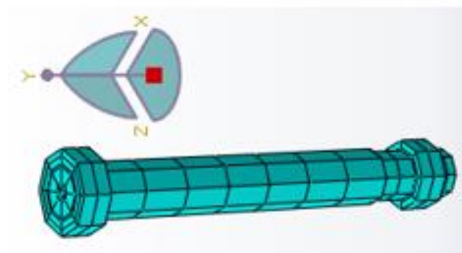
a) steel beam b) concrete slab



c) Open trough profile steel sheeting



d) Reinforcement e) Re-entrant trough profile steel sheeting



f) demountable headed stud shear connector

Figure 3. 13 all type of part with mesh

3.6 Data quality assurance

In order to assure data quality, the following measure are taken:

- The ABAQUS software was checked for the known simple structural element to check whether it was working well or not.
- The loading and all structural modeling were double checked to remove errors.
- In case of any unreliable (illogical) results due to some unobserved errors, the structure was remodeled and reanalyzed.
- A due attention and care was taken when extracting results from ABAQUS and plotting them in Excel.

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1. General

This chapter presents the results obtained using the finite element analysis method through the use of the program, ABAQUS to study the effect of shear connector geometries, profile steel sheet and concrete slab depend on the parameter of study discussed in previous chapters. The study was undertaken by considering the concrete strength, dimension of slab and geometries of sheet its types and the cross-sectional part of demountable shear connectors.

4.2. Comparison of the FE analysis model with experimental results

Before starting modeling of all specimens, the validation work was done in order to decide on different parameters. simulations of the experimental testing were compared with those from the experimental testing data obtained from the journal. The shear capacities, load slip behavior and failure modes obtained by the FE models are compared with experimental observations. The results of FE modelling have very good agreement with the experimental results.

According to finite element analysis and experimental result two main failure modes were observed in these tests. The first mode of failure is concrete cone failure with concrete crushing and cracks where no connector fracture was observed. In this type of failure, the concrete around the connector failed in compression before the connector is yielded. Crushing of concrete started from the vicinity of the connector head and cracks developed through the depth of the concrete slab forming a cone shape around the shear connectors.

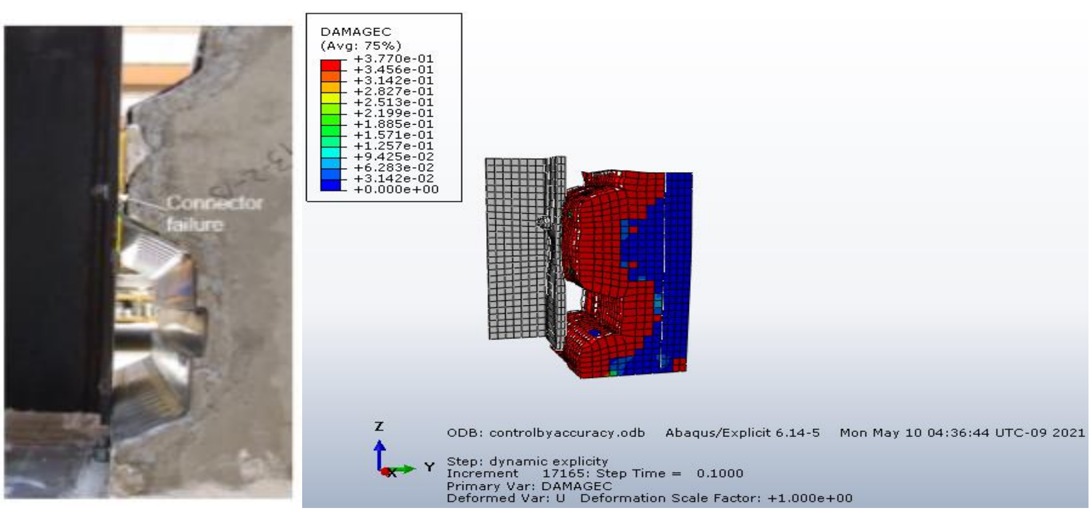


Figure 4. 1 Concrete slab failure in both of experimental and FE results.

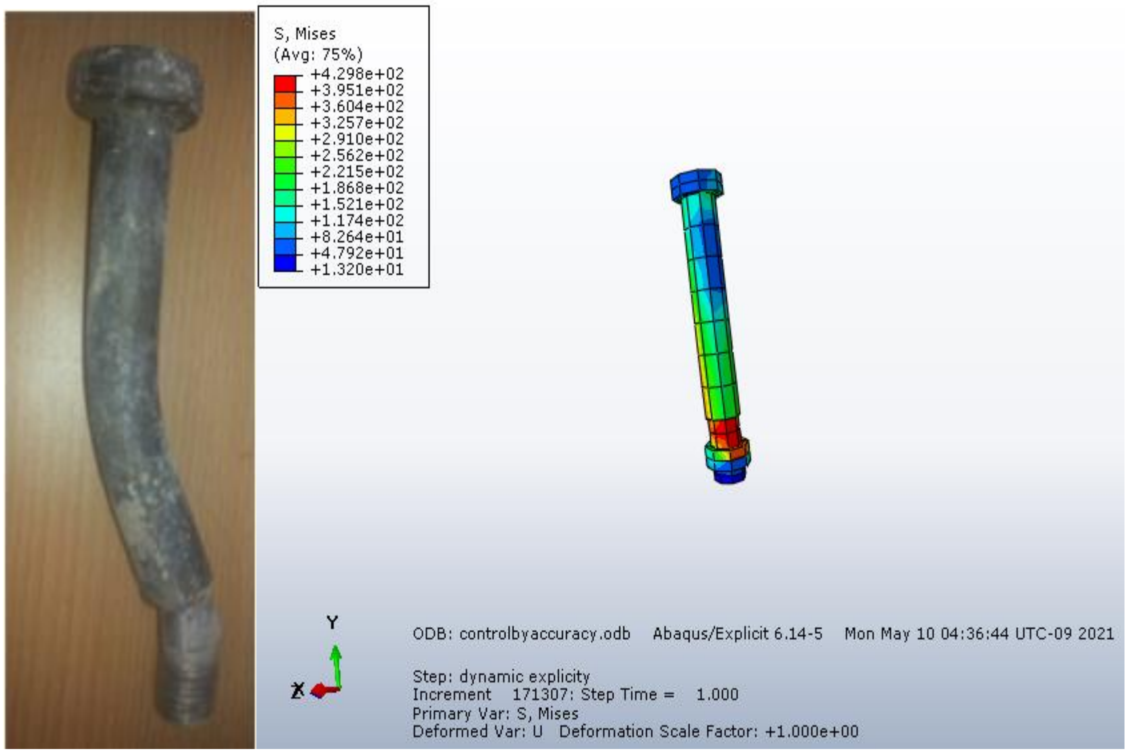


Figure 4. 2 Deformed shape of shear connector in both of experimental and FE results.

As it seen from the above figure both the experimental tested and the numerical finite element model of demountable shear connector in composite slab and steel I-section beam with open trough type profile steel sheet was failed as fracture and deformed more at collar surfaces of stud and at interact face with concrete respectively.

The comparison of ultimate strength (shear capacity of connection) versus slip curve shown in Table 4.1 as below.

Table 4. 1 Capacity of shear connector versus slip of experimental and FE model

Test specimen	Parameters	FE model result	Experimental result	% Of difference
Control Model	Ultimate load of connectors (KN)	79.879	82.6	3.29%
	Slip(mm)	6.013	7.0	12.85%

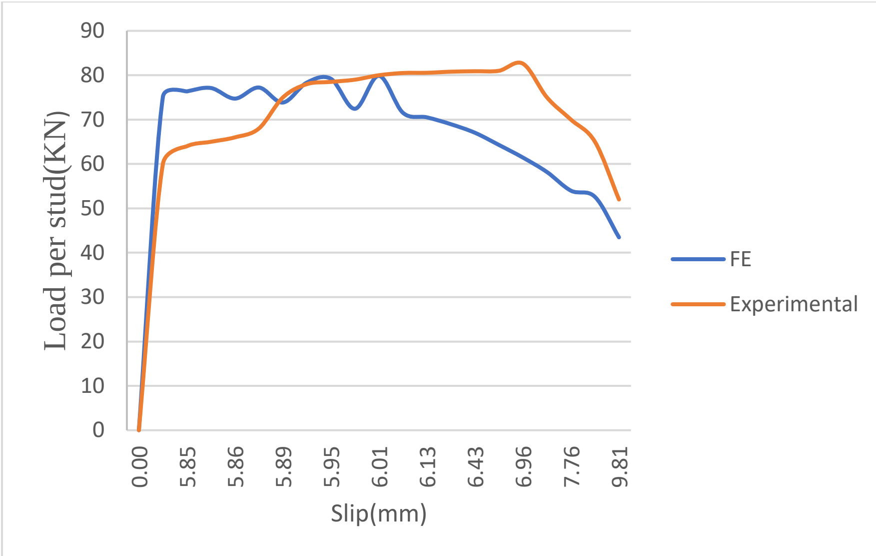


Figure 4. 3 Load-Slip curves of FE analysis and Experimental result

After the verification of the finite element model the effects of concrete strength, size of connector and type of sheet with geometries analysis have taken and finally the results are presented in this section.

4.3. Effects of change rib depth and width of open-trough profile steel sheet (OTPSS) on the capacity of shear connection

Fig.4.4. shows the load per stud versus slip relationship for the analysis by finite element results specimens Sp-3 and Sp-9 in group G-1 and G-3 respectively. The specimens had 19 × 150mm demountable headed stud shear connectors, and profiled steel sheeting type with 50mm rib depth and 157.15mm of average ribs width as Fig.3.1 (a) for G-1 and with 64mm rib depth and 146.15mm of averages rib width as Fig.3.1(b) for G-3 as well as the same concrete strength of 60Mpa for both groups.

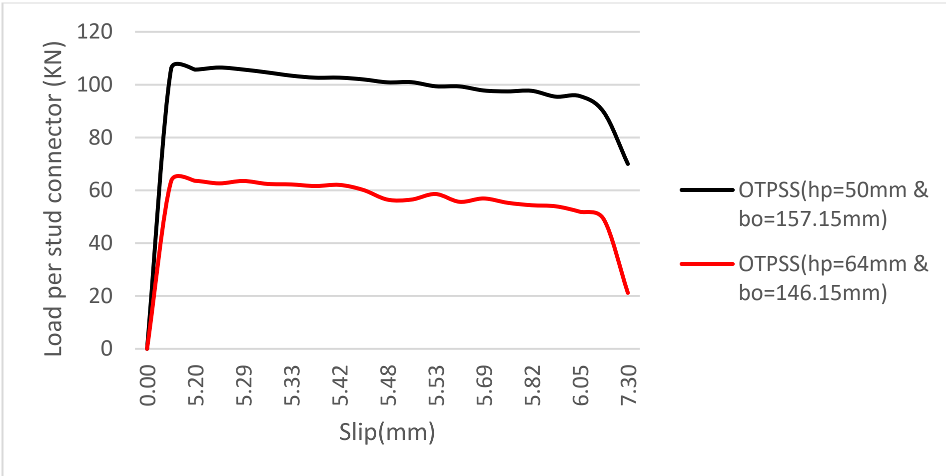


Figure 4. 4 Effect of change of rib depth(hp) and width(bo) OTPSS on capacity of shear connection of G-1 and G-3

4.4. Effects of change rib depth and width of Re-entrant trough profile steel sheet (RETPSS) on the capacity of shear connection

Fig.4.5. shows the load per stud versus slip relationship for the analysis by finite element results specimens Sp-15 and Sp-21 in group G-5 and G-7 respectively. The specimens had 19 × 150mm demountable headed stud shear connectors, and profiled steel sheeting type with 50mm rib depth and 120mm of ribs width as Fig.3.1 (c) for G-5 and with 64mm rib depth and 140mm of rib width as Fig.3.1(d) for G-7 as well as the same concrete strength of 60Mpa for both groups

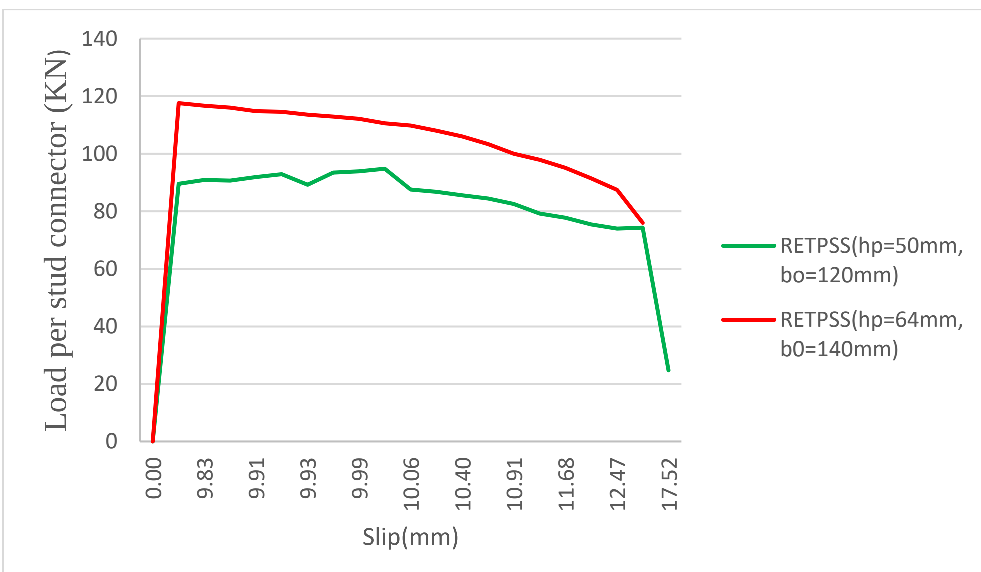


Figure 4.5 Effect of change in rib depth(hp) and width(bo) RETPSS on capacity of shear connection of G-15 and G-21

4.5 Effect of change in diameter of connector on the capacity of shear connection with OTPSS.

Looking to the parameters study of specimens in groups G1-G4 only by OTPSS as shown in Table 3.1, the FE specimens had identical dimensions, but with two different stud diameters with two different type of profiled steel sheeting as 19 and22 mm with on Fig 3.1 (a) and (b) respectively by incremental data of concrete strength as arrangement of specimen order. Fig. 4.6 shows the effect of change in stud height on the load per stud versus slip relationships obtained from the FE analysis in groups G-1 taken for specimen two (Sp-2), G-2 taken for specimen two (Sp-2), G3 taken for specimen five (Sp-8) and G-4 taken for specimen two (Sp-11), having headed studs of 19 mm diameter and heights with collar of 150 mm and having 22mm diameter and height of 150mm. In this section of data arrangement G-1 and G-2 were under OT1PSS and G-3 and G-4 were categorized under OT2PSS as shown in Figure 4-6.

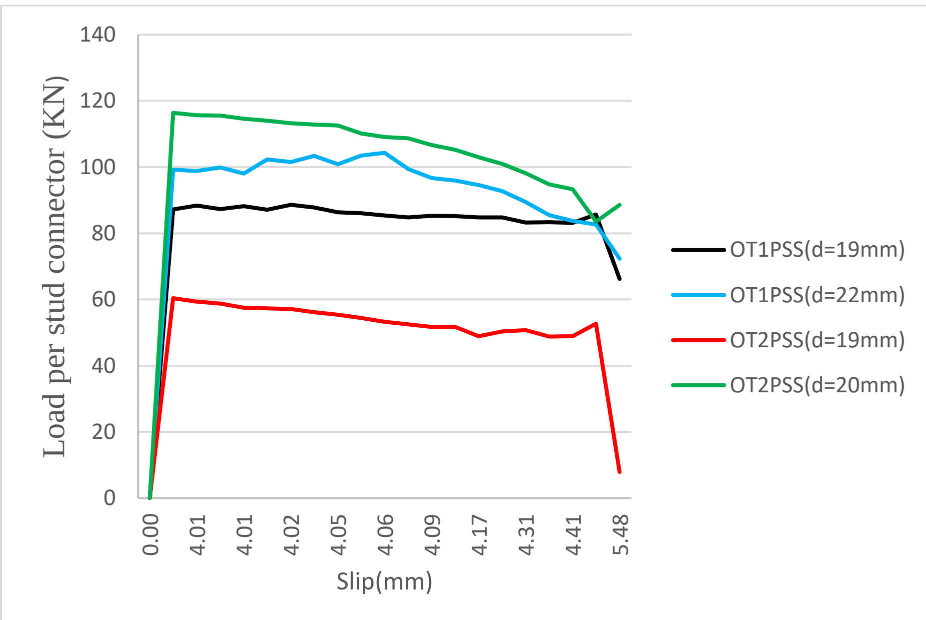


Figure 4. 6 Effect of change in diameter of connector on the capacity of shear connection with OTPSS.

4.6 Effect of change in diameter of connector on the capacity of shear connection with RETPSS

Looking to the parameters study of specimens in groups G5-G8 only by OTPSS as shown in Table 3.1, the FE specimens had identical dimensions, but with two different stud diameters with two different type of profiled steel sheeting as 19 and22 mm with on Fig 3.1 (c) and (d) respectively by incremental data of concrete strength as arrangement of specimen order. Fig. 4.7 shows the effect of change in stud height on the load per stud versus slip relationships obtained from the FE analysis in groups G-5 taken for specimen fourteen (Sp-14), G-6 taken for specimen seventeen (Sp-17), G7 taken for specimen twenty (Sp-20) and G-8 taken for specimen twenty-three (Sp-23), having headed studs of 19 mm diameter and heights with collar of 150 mm and having 22mm diameter and height of 150mm.In this section of data arrangement G-5 and G-6 were under RET1PSS and G-7 and G-8 were categorized under RET2PSS as shown in Figure 4-7.

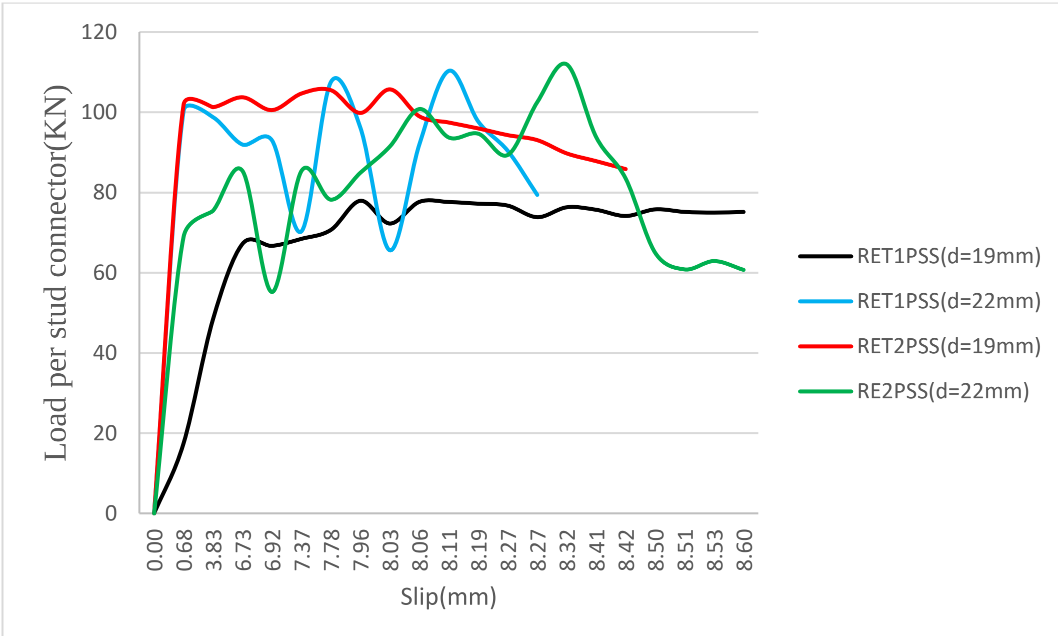


Figure 4. 7 Effect of change in diameter of connector on the capacity of shear connection with RETPSS

4.7 Effects of change in Type of profile steel sheet on the capacity of shear connection

According result taken from finite element there were two type of profile steel sheeting results with different parameter studies. In these cases, it effects the shear capacity of connector as described in load per stud versus slip on fig.4.8. As the Figure indicated the data under OT1PSS were contains G-1 to G-2, OT2PSS were also contains G-3 to G-4 with their specimens again also RET1PSS and RET2PSS were contains G-5 to G-6 and G-7 to G-8 respectively. As a sample from G-1 specimen taken was Sp-1 and from G-2 specimen Sp-7 was selected. From the RETPSS for the first data of RET1PSS the specimen of SP-13 was taken and from RET1PSS the specimen Sp-19 was taken from the output as a result and presented on the Figure 4.8 as below. In all case the data and parametric study was taken according to Table that tabulated under section 3.3 in Table 3. 1 presented as input data and research variable of these papers.

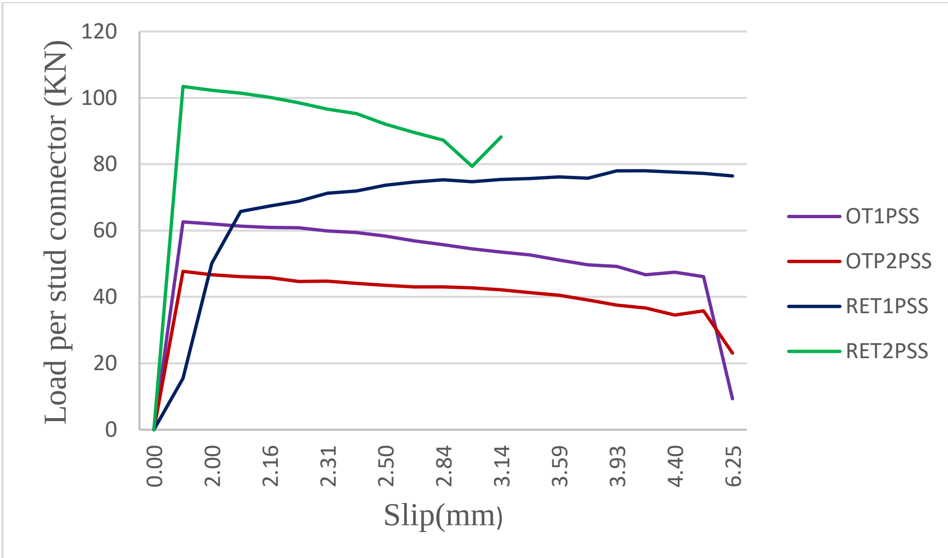


Figure 4. 8 Effects change in Type of profile steel sheet on the capacity of shear connection

4.8 Effects of concrete strength on the capacity of shear connection

According to Fig 4.9 and fig 4.10 Figured in below the concrete strength effect the result of shear connector analyzed by FE with concrete class of C-40, C-50 and C-60 with different type of profiled steel sheet discussed in parametric study sections.

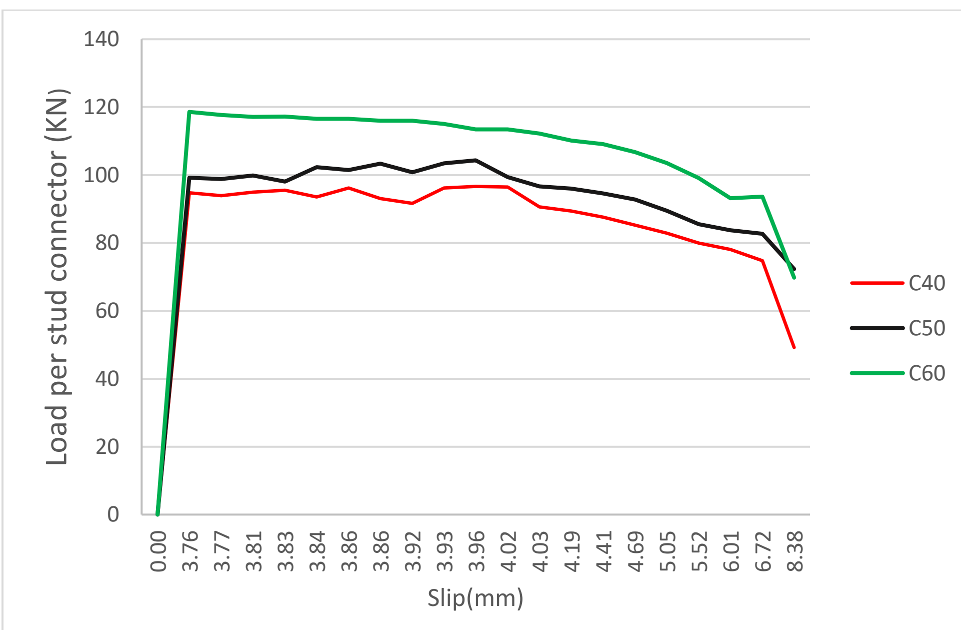


Figure 4. 9 Effects of concrete strength on the load-slip behavior of 22x150mm stud connector of group-2 with OTPSS

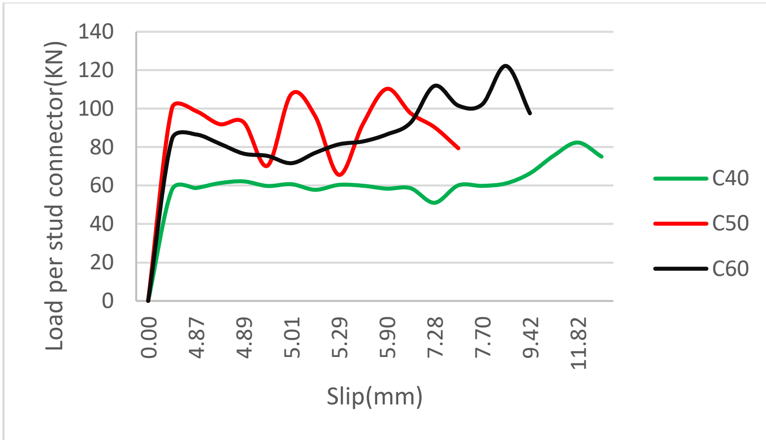
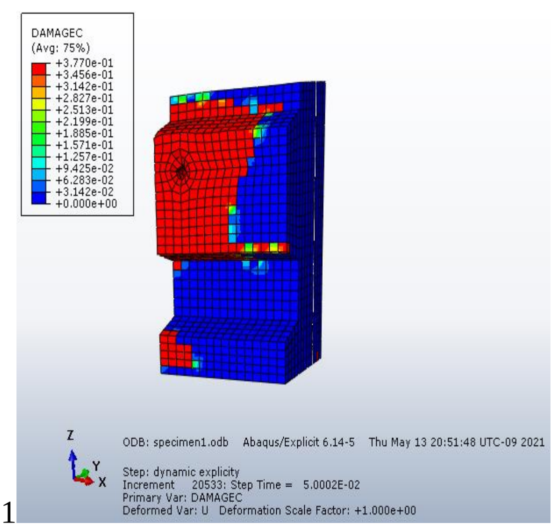


Figure 4. 10 Effects of concrete strength on the load-slip behavior of 22x150mm stud connector of group-6 with RETPSS

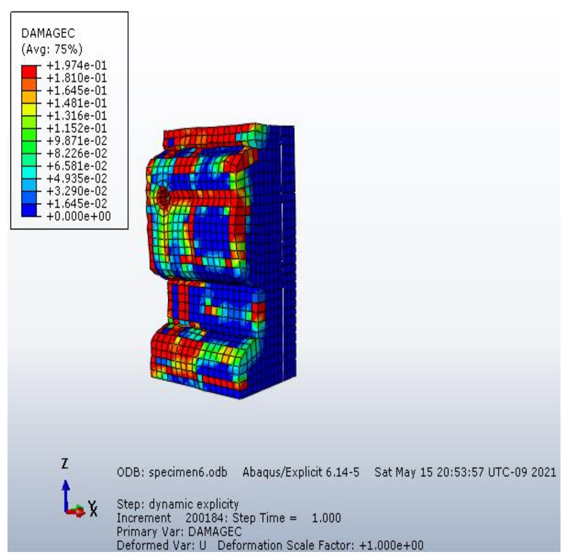
4.9 Damages of concrete slabs compared for the different concrete strength classes

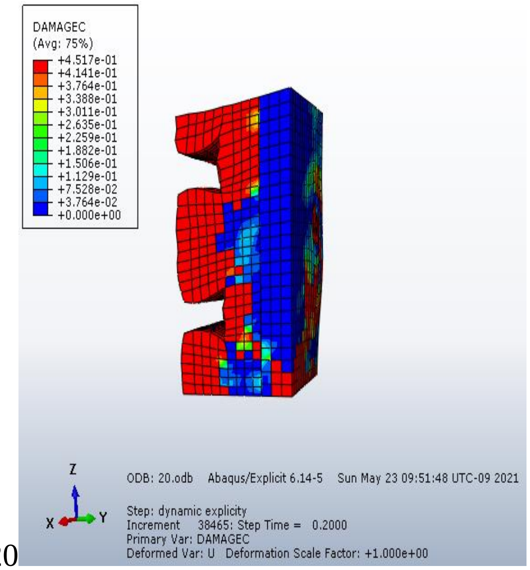
The concrete compressive damage (DAMAGE C) derived from ABAQUS software are compared for the different concrete strength classes putted in Figures below. Even though the force per shear connector-slip curves were increased, both the stiffness degradation and the compressive damage of concrete around the bolt decrease from strength class C40 to C60. It can be observed that cracking in the finite element model of the concrete occurred at the location around the shear connectors. The compression and tensile damage variables (dc and dt) in the finite element model show that concrete failure happened during finite element analysis.

sp-1



sp-6



sp-20 

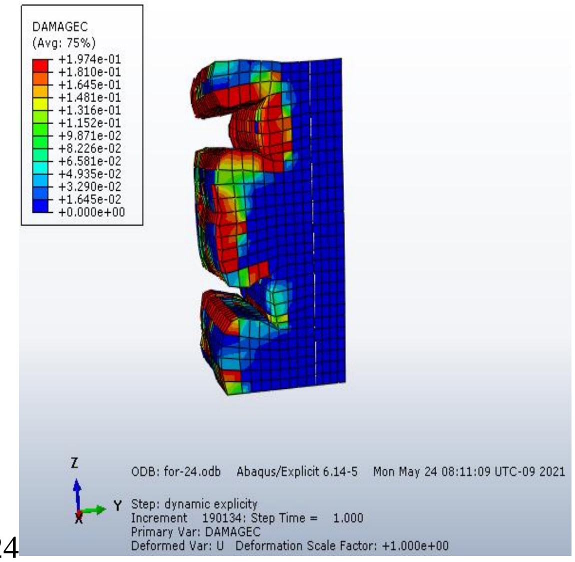
sp-24 

Figure 4. 11 Damages of composite slab in model

4.10. Failure (deformed shapes) of demountable shear connector for different concrete strength classes.

The stress contour clearly shows the yielding of the demountable shear connector, the failure modes were also investigated due to shearing of the demountable shear connectors this was correctly simulated through the finite element model. When the failure load was reached, deformation in the demountable shear connector became more prominent and the maximum stresses were experienced at the position of the collar as is evident from the stress contour plot of the shear connector in Figure 4-12 to 4-15. In this investigation, the studs in the FE specimens are specified with the yield strength of 450Mpa and with diameters of 19 and 22 mm. A number of concrete strengths f_{cu} from C40, C50, C60 are considered, resulting in 24 specimens in the FE calculation. The calculated ultimate shear capacity per stud is defined according to the maximal value in the load-slip curve simulation by FE analysis.

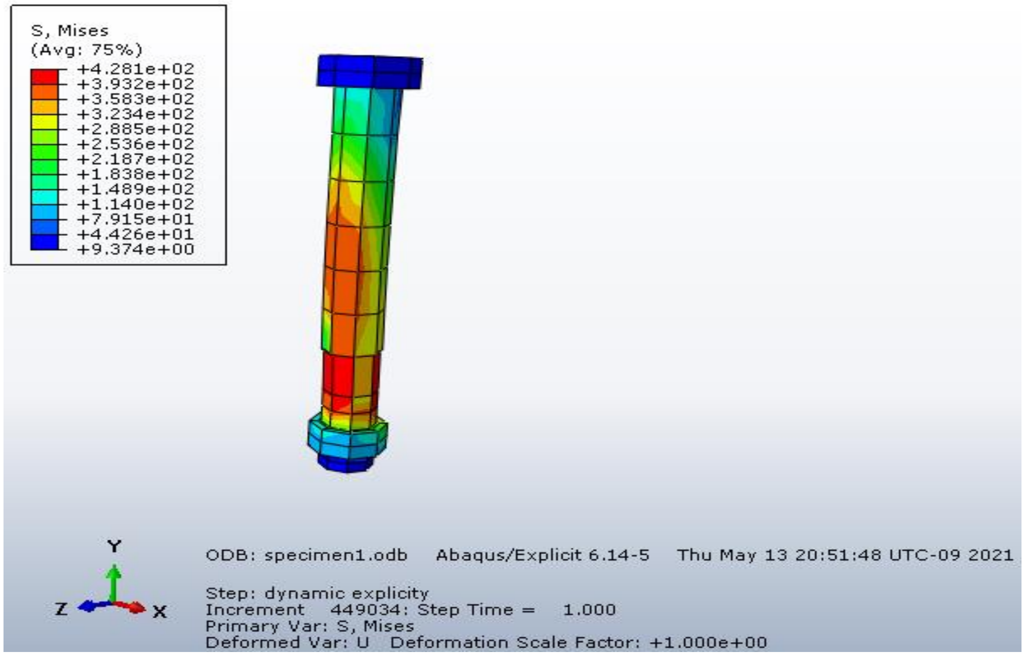


Figure 4. 12 failure of demountable shear connector of d=19mm in specimen Sp-3

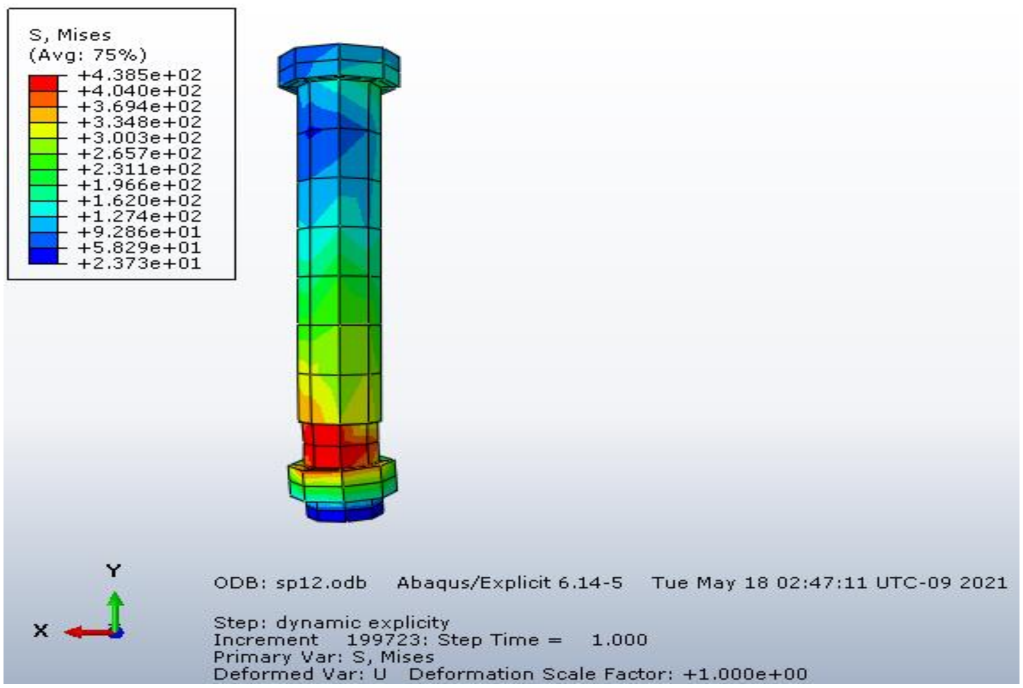


Figure 4. 13 Failure of demountable shear connector of d=22mm in specimen Sp-12

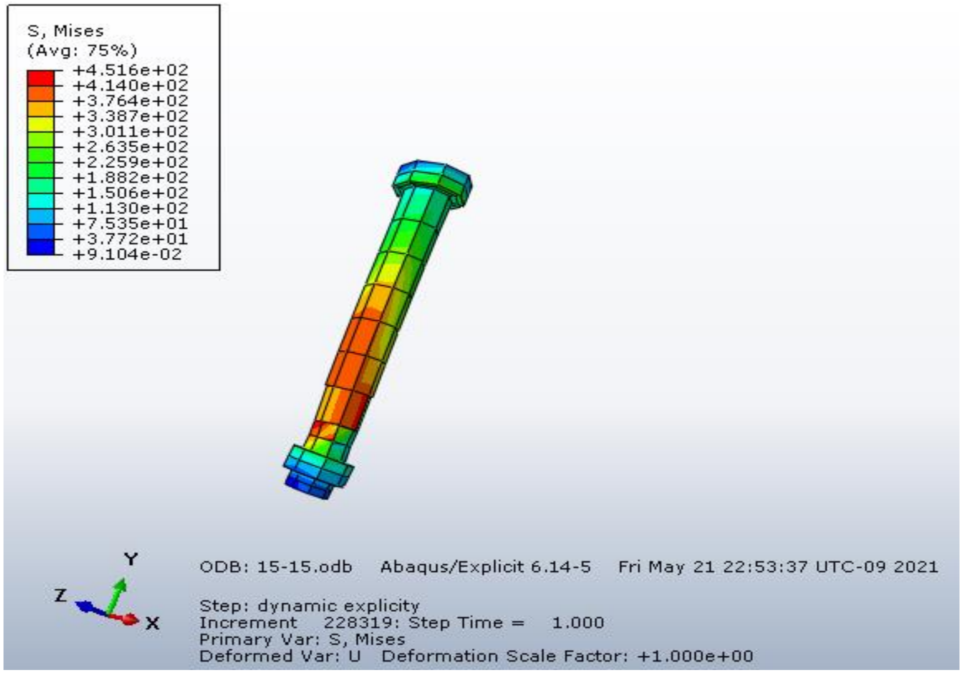


Figure 4. 14 Failure of demountable shear connector of d=19mm in specimen Sp-15

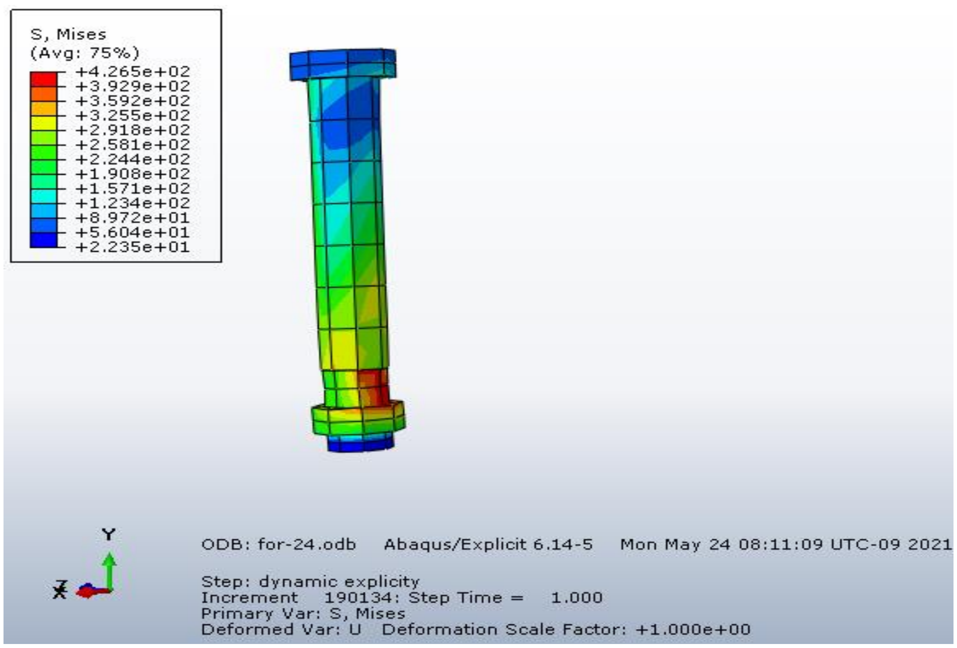


Figure 4. 15 Failure of demountable shear connector of d=22mm in specimen Sp-24

4.12 Damages of profiled steel sheeting compared for the different concrete strength classes

There are two types of sheets in this research paper as parametric study means open trough profiled steel sheeting (OTPSS)and re-entrant trough profiled steel sheeting (RETPSS) as discussed previous chapter. The OTPSS type were contain two parts as seen in Fig.3.1((a) and (b)) depending on its geometric properties which means discussed in above of section-3. The other type of sheet was RETPSS also contains two part like as Fig 3.1((c) and (d)) based on the geometrical data which discussed in there. Here there were the result of thus sheet with demountable shear connection in composite structures simulated by finite element methods. After analyzed there is a damaged or failure parts of the sheet putted in Fig.4.16 presented as follows according to the specimen arranged by group.

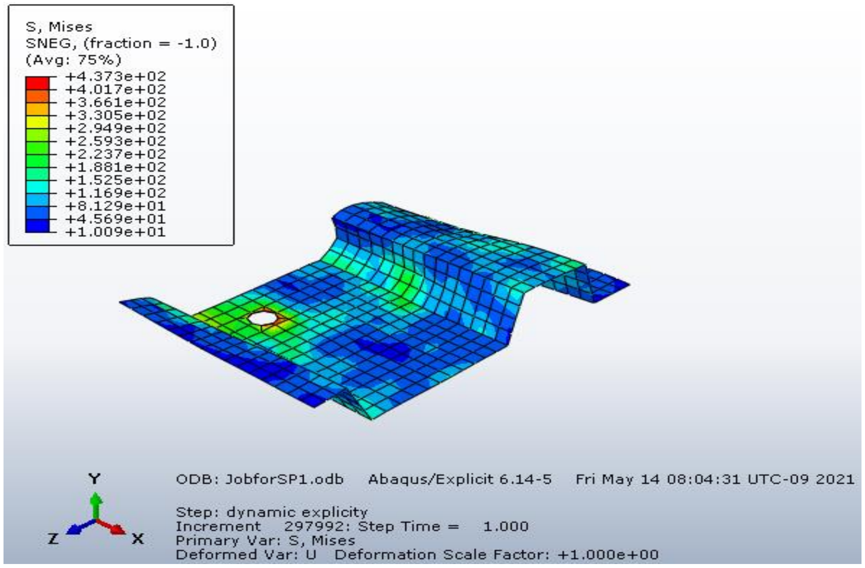


Figure 4. 16 Failures of OTPSS in specimen Sp-2

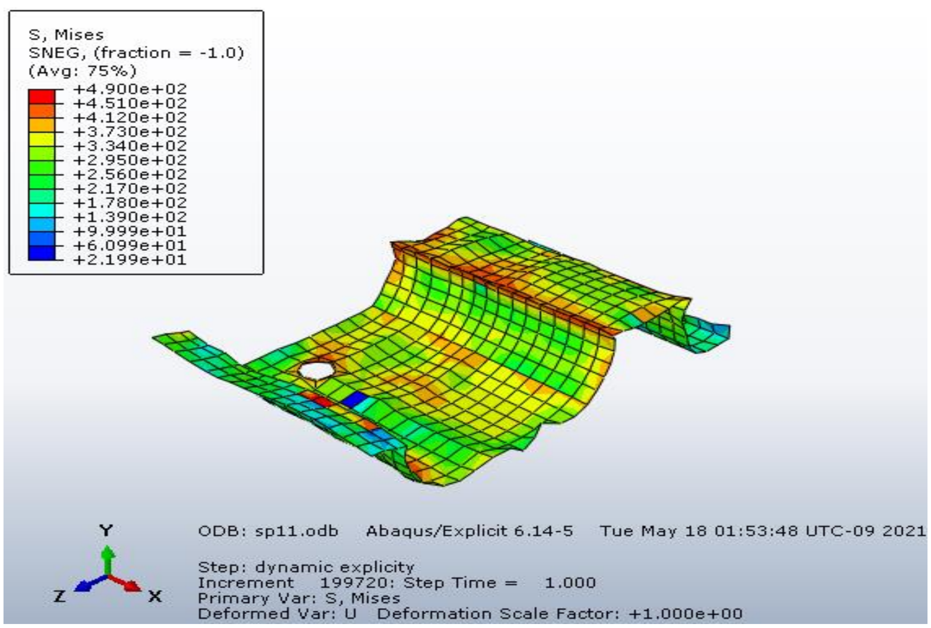


Figure 4. 17 Failures of OTPSS in specimen Sp-11

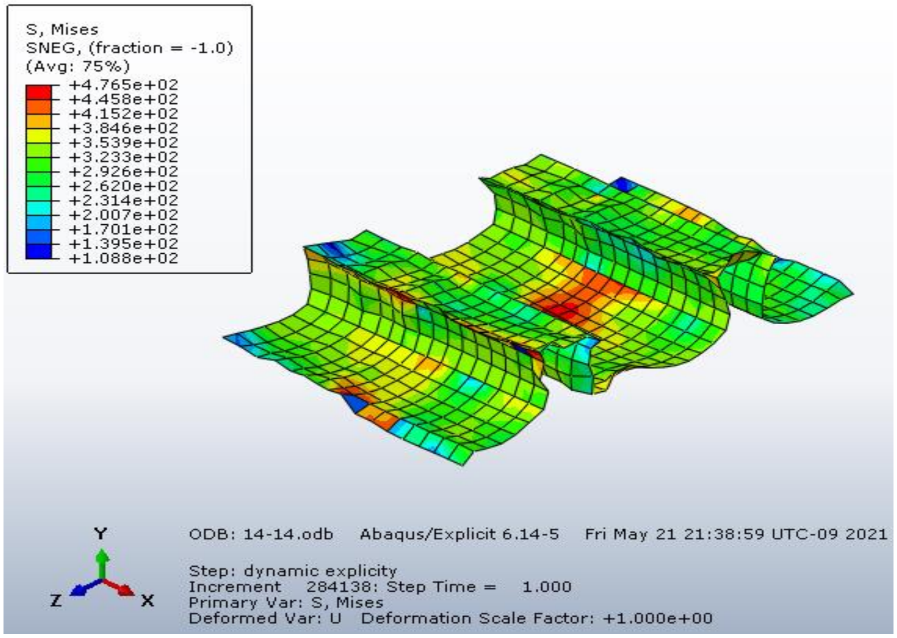


Figure 4. 18 Failures of RETPSS in specimen Sp-14

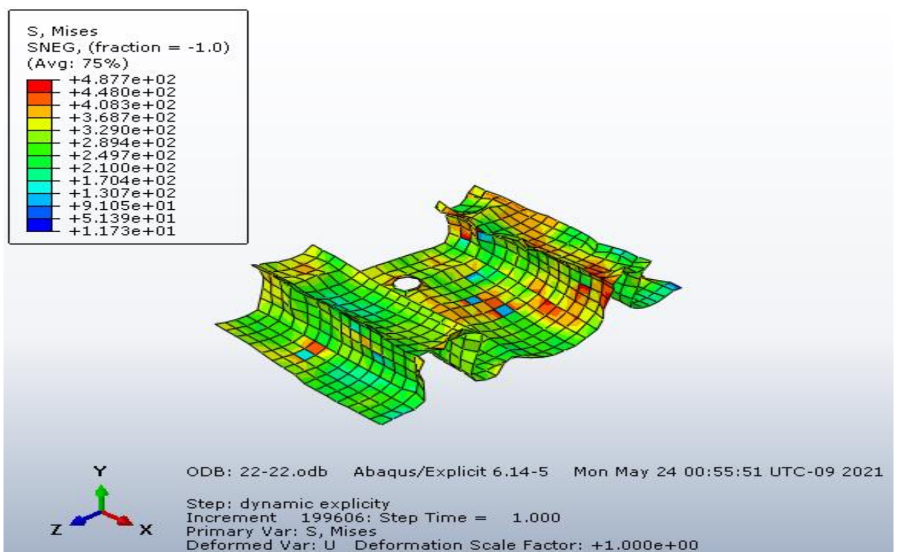


Figure 4. 19 Failures of RETPSS in specimen Sp-22

4.13. Comparison of shear connection capacities obtained from finite element analysis and the European design Code specification

The shear connection capacities obtained from the parametric study were compared with the nominal unfactored design strengths of headed stud shear connectors predicted by European Code (EC4)[20]. The EC4 took a similar approach to other Specification by presenting Equation 2.1 to 2.6 for determining the design strength (P_{EC4}) of the headed stud in composite beams with profiled steel sheeting perpendicular to the steel beam.

Table 4.2 shows the comparison of the shear connection capacities obtained from the finite element analysis and European Code (EC4) [20]. The EC4 predictions were generally conservative. except for some cases that overestimated the capacity of the shear connection with a maximum value of average from all specimen 1%. It can also be seen that the difference between the shear connection capacities obtained from the FE analysis and the design rules specified in the three specifications increases with the increase in concrete strength. The mean values of P_{FE}/P_{EC4} ratios are 1.00543 with the corresponding coefficients of variation (COV) of 0.0439464. Figs. 10–12 plotted the relationship between the load per stud versus the concrete cube strength for the FE model specimens in groups G1, G4, G5 and G8 having 19 × 100 mm for G1 and G5 and 22 × 100 mm for G4 and G8 also headed studs respectively. The curves show the shear connection capacities obtained from the FE analysis and the design rules for the specimens with profiled steel sheeting type (a), (b), (c) and (d) all were shown in Fig. 3.1.

The numerical determination of shear connection capacity in composite structures with profiled steel sheeting analyzed as follow thus described in literature.

$P_r = K_t P_{rd}$ 4.1

From the both of the following formula the value of P_{rd} shall be the smallest of: -

$$P_{Rd} = \frac{0.29\alpha d^2\sqrt{f_{ck}E_c}}{\gamma_v} \quad 4.2$$

$$P_{Rd} = \frac{0.8 f_u \pi d^2}{4\gamma_v} \quad 4.3$$

Where; - $\alpha = 0.2\left(\frac{h_{sc}}{d}+1\right) \leq 1.0, \text{for } 3 \leq \frac{h_{sc}}{d} \leq d \text{ and } \alpha = 1, \text{for } \frac{h_{sc}}{d} > 4$

d = the shear connector shank diameter in mm

h_{sc} = the shear connector height

f_u = the shear connector ultimate tensile strength

f_{ck} = the characteristic cylinder compressive

E_c = modulus of elasticity of concrete

γ_v = the partial safety factor for shear resistance (γv = 1.25)

For the transverse direction the values of K_t are: -

$$K_t = \frac{0.7}{\sqrt{n_r}}\frac{b_o}{h_p}\left[\frac{h_{sc}}{h_p} - 1\right] \leq 1.0 \quad 4.4$$

Sample calculation for specimen one according to the European code specification formula discussed in the above sections: -

Sample one

In specimen one h_p=50mm and h=110mm above the embedment surface only.

From the $\frac{h_{sc}}{d} = \frac{110}{19} = 5.8 > 4$ take $\alpha = 1.0$

$\text{Prd} = \frac{0.29*1*19^2*\sqrt{40*35000}}{1.25} = 99.09\text{KN}$ from equation 4.2 and $\text{Prd} = \frac{0.8*450*\pi*19^2}{4*1.25} = 81.6\text{KN}$ from equation of 4.3.

Take P_{rd}=81.6KN selected because the smallest one governs it.

Sample two

In specimen four h_p=50mm and h=110mm above the embedment surface only.

From the $\frac{h_{sc}}{d} = \frac{110}{19} = 5.8 > 4$ take $\alpha = 1.0$

$\text{Prd} = \frac{0.29*1*22^2*\sqrt{40*35000}}{1.25} = 132.85\text{KN}$ from equation 4.2 and $\text{Prd} = \frac{0.8*450*\pi*22^2}{4*1.25} = 109.5\text{KN}$ from equation of 4.3.

Take P_{rd}=109.5KN selected because the smallest one governs it.

Table 4. 2 Comparison of shear connection capacities obtained from finite element analysis and current codes of practice

Group	Specimen	FE(KN)	EC4(KN)	FE/EC4
G1	sp1	62.6	81.6	0.767157
	sp2	88.6	81.6	1.085784
	sp3	106.5	81.6	1.305147
G2	sp4	96.6	109.5	0.882192
	sp5	104.3	109.5	0.952511
	sp6	118.6	109.5	1.083105
G3	sp7	47.7	81.6	0.584559
	sp8	60.4	81.6	0.740196
	sp9	63.6	81.6	0.779412
G4	sp10	106.4	109.5	0.971689
	sp11	116.4	109.5	1.063014
	sp12	132.3	109.5	1.208219
G5	sp13	78	81.6	0.955882
	sp14	78	81.6	0.955882
	sp15	94.8	81.6	1.161765
G6	sp16	82.4	109.5	0.752511
	sp17	110.3	109.5	1.007306
	sp18	122.2	109.5	1.115982

G7	sp19 sp20 sp21	103.4 105.7 117.6	81.6 81.6 81.6	1.267157 1.295343 1.441176
G8	sp22 sp23 sp24	89.1 102.5 110	109.5 109.5 109.5	0.813699 0.936073 1.004566
Mean COV				1.00543 0.0439464

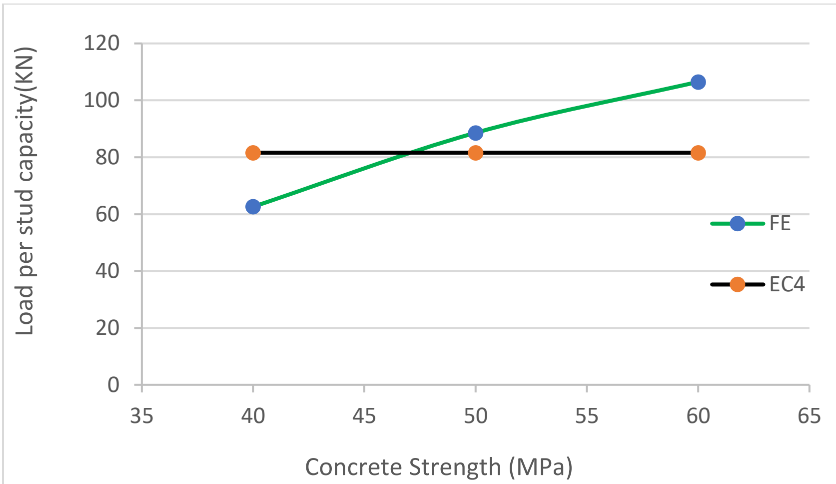


Figure 4. 20 Capacity of shear connection from parametric study of group G1.

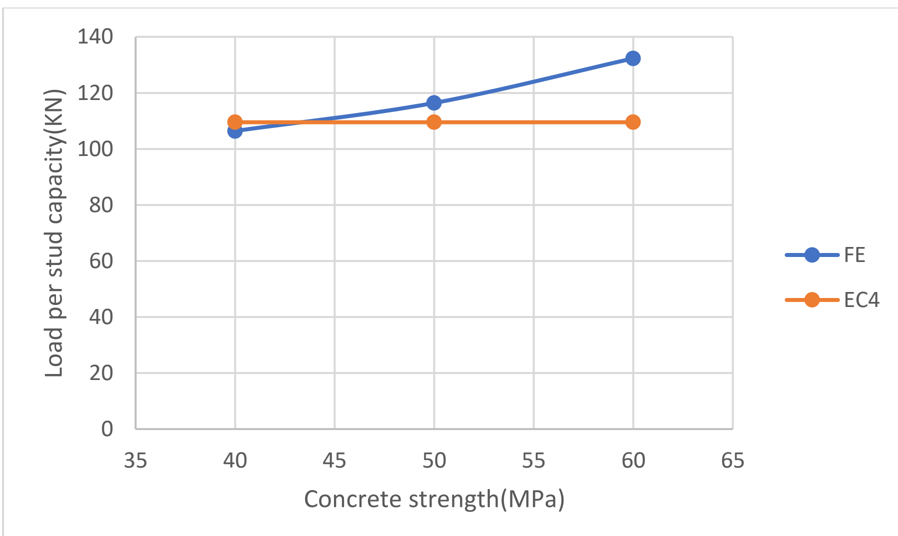


Figure 4. 21 Capacity of shear connection from parametric study of group G4.

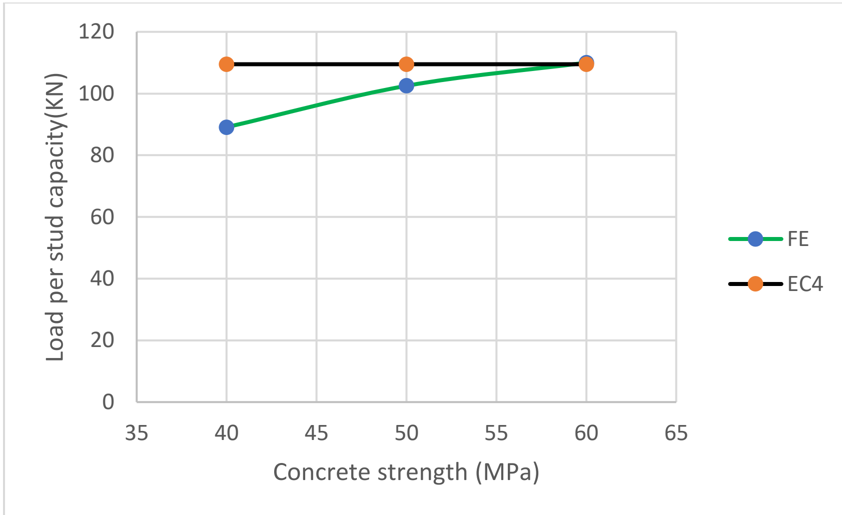


Figure 4. 22 Capacity of shear connection from parametric study of group G8.

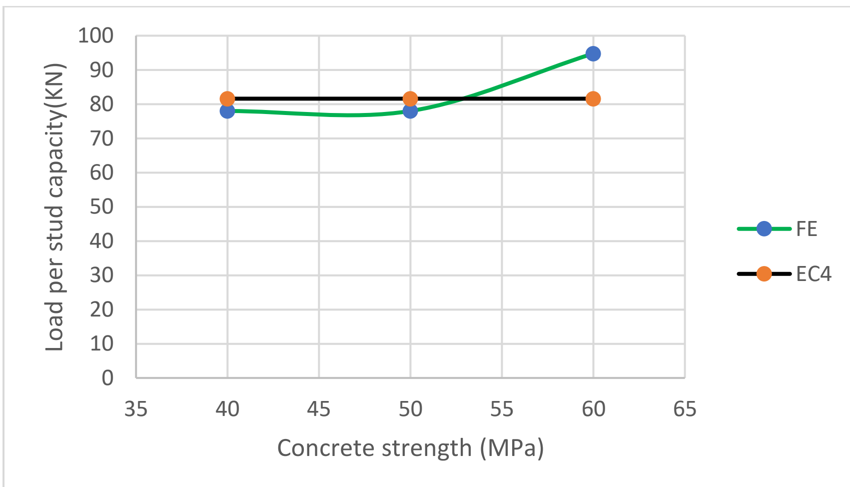


Figure 4. 23 Capacity of shear connection from parametric study of group G5.

4.14 Effect of change in ribs depth and width on the capacity of shear connection

Fig. 4.24 and 4.25 shows the effect of change in the rib depth on the load per stud versus the concrete strength relationships obtained from the FE analysis and the design rules with in the open trough and re-entrant trough profiled steel sheet. The curves were plotted for the 19 × 100mm and 22x100mm headed studs in the specimens of all group. G3 and G4 having the average rib width (b_o) of 146.15 mm and rib depths of 50 and 64 mm, respectively for open trough profiled steel sheeting and G7 and G8 having the average rib width (bo) of 140 mm and rib depths of 50 and 64 mm, respectively for re-entrant trough profiled steel sheeting. The design strengths calculated EC4 specifications were also plotted in Fig.4.24 and 4.25. It can be seen that the EC4 predictions were conservative and consider the effect of the change in rib depth from 50 to 64 mm on the strength of headed studs19 × 100 mm and 22x100mm in the specimens of groups G3, G4 and G7, G8 in Fig 4.24 and 4.25. respectively. That means, the EC4 took into consideration the effect of change in rib depth. The EC4 design strengths were conservative for the simulation specimens with rib depths of 50 and 64mm at different concrete cube strengths.

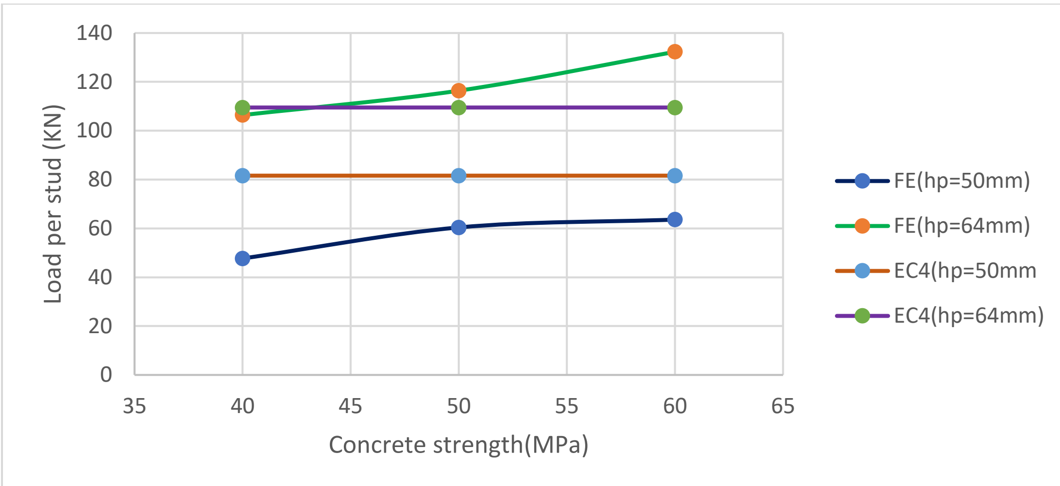


Figure 4. 24 Effect of change in ribs depth and width on the capacity of shear connection with OPTPSS

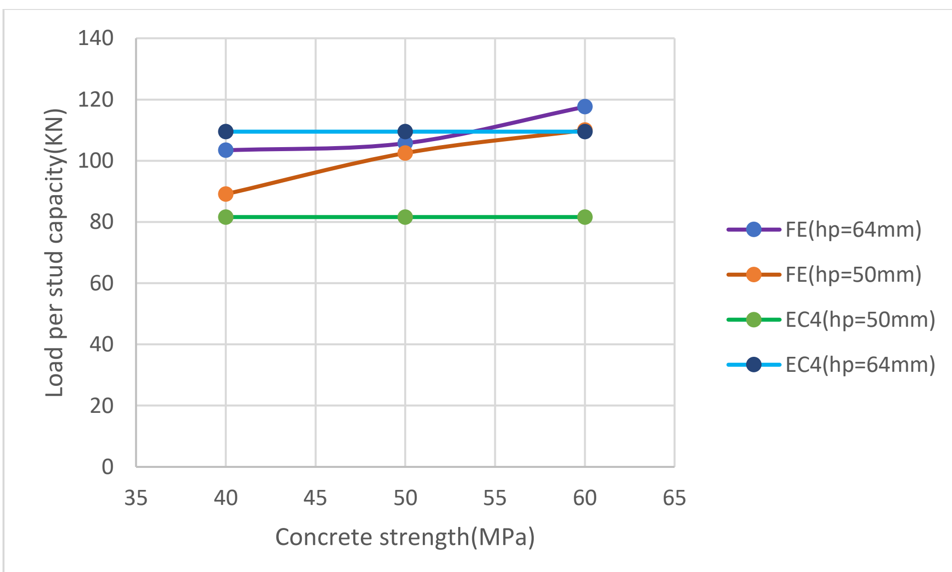


Figure 4. 25 Effect of change in ribs depth and width on the capacity of shear connection with RETPSS

4.15. Effect of change dilation angle on capacity of shear connection

Dilation angle plays a significant role in performance of shear connection in composite slab and steel beam with profiled steel sheet. As the values of the dilation angle increased from 36° to 40°

the capacity of connector also increased. See Figure 4.26 drawn on shear capacity versus the strength of concrete in the specimen. In some specimen since the angle of dilation increases the capacity of connector increase by 4.7% only. But in other case it remains constant.

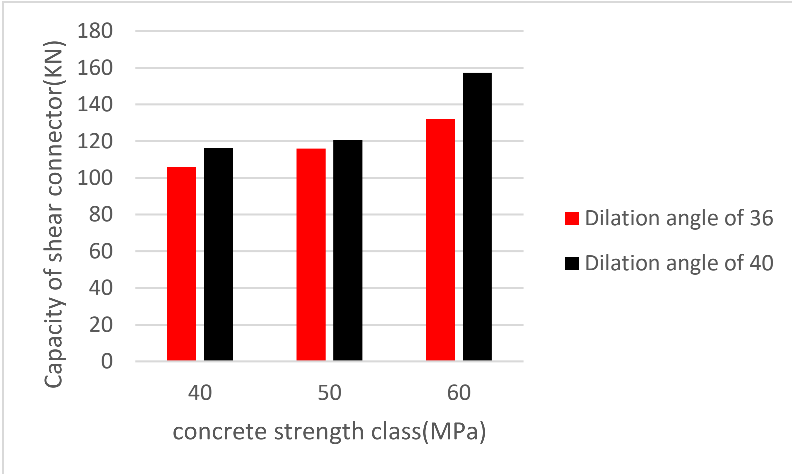


Figure 4. 26 Effects of dilation angles

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The strength of demountable shear connectors in composite structures was investigated in this thesis. The finite element analysis was also carried out for deeper understanding of the composite structures using in chapter 4. The main parameters investigated were the type of reinforcement, type of demountable shear connectors, diameter of connectors, number of connectors in each trough of profiled metal decking and concrete compressive strength. Accurate nonlinear finite element models have been developed to investigate the capacity of demountable shear connection in composite slab and beam with profiled steel sheeting perpendicular to the steel beam. The models consider the nonlinear material properties of the concrete, steel beam, profiled steel sheeting, reinforcement bars and headed stud shear connectors. The capacity of shear connection, load–slip behavior of headed stud and failure modes were predicted from the finite element analysis. An extensive parametric study of 24 specimens with open trough and re-entrant trough profiled steel sheeting different geometries, with vary demountable headed shear stud diameters as well as different concrete strengths was performed using the finite element models. Based on the Finite Element Analysis and parametric study the following conclusion were drawn:

- ❖ Similar to the other types of shear connectors, demountable shear connectors have two main failure modes: connector fracture and concrete crushing.
- ❖ The concrete strength class does affect the capacity of demountable headed stud shear connection. The capacity of shear connector increases by 26% and 20% with open trough and re-entrant trough profiled steel sheeting respectively from concrete strength class C40 to C60 which is the consistent with the increase of young’s modulus of elasticity of concrete by 10% from C40 to C60.
- ❖ Geometry of profile steel sheeting effects on the performance of composite structures resist the shear forces. It appears that the ultimate shear resistance increased with the increase depth of sheets. In this the capacity of composite structures increase by 10.6% and 10.06% with in the open trough profiled steel sheet depth(hp) from 50mm to 64mm and re-entrant trough profiled steel sheet depth(hp) from 50mm to64mm respectively. But types of

profiled steel sheet did not have any effect on the ultimate shear resistances of the composite structures.

- The parameters that have the large influences on the resistance of the demountable shear connection with profile steel sheet are the diameters of stud connectors. The shear capacity of connectors increases 26% and 17% from diameter of stud 19mm to 22mm with open trough profile sheeting and re-entrant profile trough steel sheet respectively.
- The comparison of shear connection capacities obtained from the finite element analysis and European Code have shown that the design rules specified in the European Code were generally conservative, except for some cases that overestimated the capacity of shear connection with a maximum value of 1%.
- Since the dilation angle in the concrete damage plasticity was increased in the model from 36° to 40° the capacity of shear connection in composite slab and steel beam with profiled steel sheeting also increased by 4.7% with in high concrete strength.

5.2 Recommendation

Based on the results obtained from this study, the following recommendations are proposed for future work.

- Finite element models, in which the threaded parts of the collar connector will be included and the long-term effect on the relaxation of torque (nut) with should be created for more accurate results.
- Other parameters like the position of demountable shear connection with longitudinal distances and contact surfaces of demountable shear connector parts with others parts in models should be checked if they affect the capacity of shear connections.
- In the push-off work performed in the experiment those present in literatures, attention should be paid on the re-entrant trough profiled steel sheeting in order to interact composite structures.
- The type of sheet considered in this study were open and re-entrant trough profile steel sheeting. In other parameters like as smoothing profile steel sheet and sheet with embossment type can be considered for future work

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A. ANNEX - INPUT DATA FOR ABAQUS

In chapters 4 in Finite Element Analysis, detailed material models were used. The input data for the ABAQUS material models were obtained from equation and formulas thus discussed in chapter 3 of all sections.

A.1 Concrete

Concrete C40

Table A. 1 Density and Elastic for C40

Density	2.50E-06
Elasticity	
Young's modulus	Poisson's ratio
35000	0.2

Table A. 2 Concrete Damaged Plasticity for C40

Dilation angle	Eccentricity	fb_o/fc_o	k	Viscosity parameter
36	0.1	1.2	0.6	0.001

Table A. 3 Concrete Compression Hardening for C40

Yield stress	Inelastic strain
19.2	0
32.6	0.00053
42.7	0.00105
47.6	0.00158
48	0.00175
47.3	0.00199
45.1	0.00223
41.1	0.00247
35.4	0.00271
27.5	0.00295
24.19	0.00317
20.98	0.00385
17.88	0.00496
14.95	0.00653
12.25	0.00854
9.83	0.011
7.72	0.0139

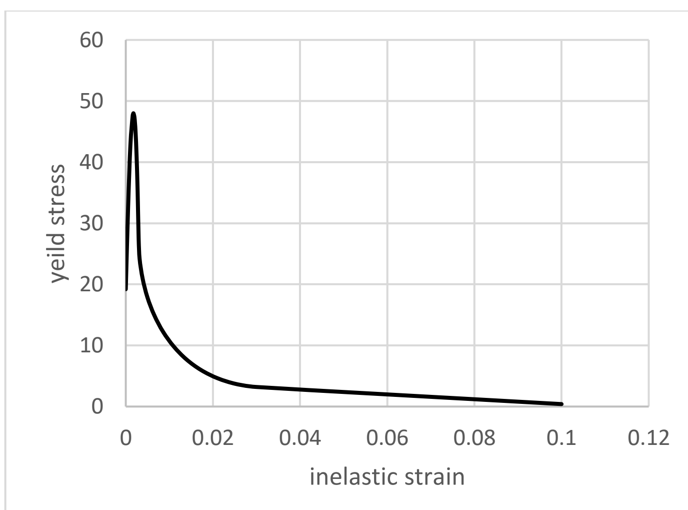

Figure A. 1 Concrete compression hardening for C40

Table A. 4 Concrete Compression Damage for C40

Damage parameter	Inelastic strain
0	0
0	0.00014
0	0.00038
0	0.00076
0	0.00093
0.0148	0.00119
0.0613	0.00149
0.1428	0.00184
0.2635	0.00225
0.4279	0.00272
0.496	0.00303
0.563	0.00379
0.6275	0.005
0.6885	0.00665
0.7447	0.00874
0.7952	0.01127
0.8391	0.01423
0.8754	0.01764
0.9036	0.02147
0.9231	0.02575
0.9333	0.03046
0.9917	0.10054

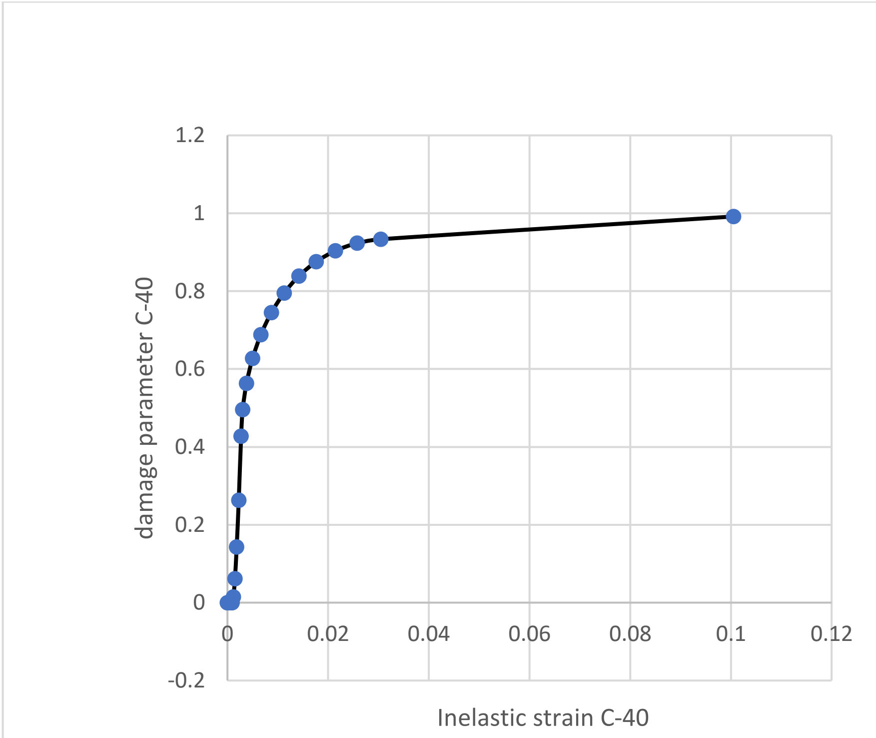

Figure A. 2 Concrete compression damage for C40

Table A. 5 Concrete Tension Stiffening for C40

Yield Stress	Cracking Strain
3.5	0
2.97	0.0001
2.45	0.0002
1.96	0.0003
1.51	0.0004
1.11	0.0005
0.77	0.0006
0.5	0.0007
0.31	0.0008
0.2	0.0009
0.18	0.001

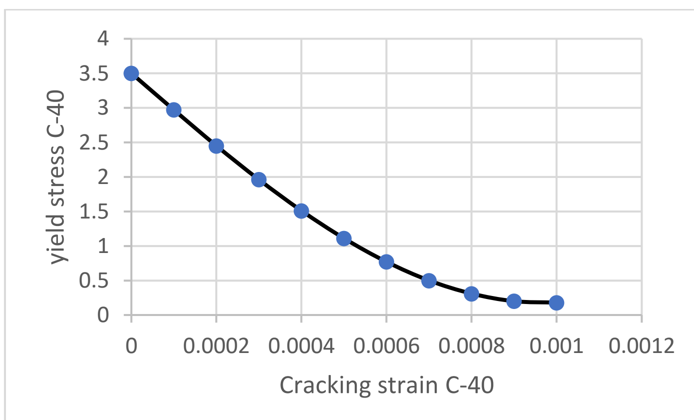

Figure A. 3 Concrete tension stiffening for C40

Table A. 6 Concrete Compression Damage for C40

Damage Parameter	Cracking Strain
0	0
0.1514	0.00011
0.299	0.00022
0.439	0.00033
0.5678	0.00044
0.6821	0.00055
0.779	0.00066
0.856	0.00076
0.9111	0.00087
0.9427	0.00097
0.95	0.00107

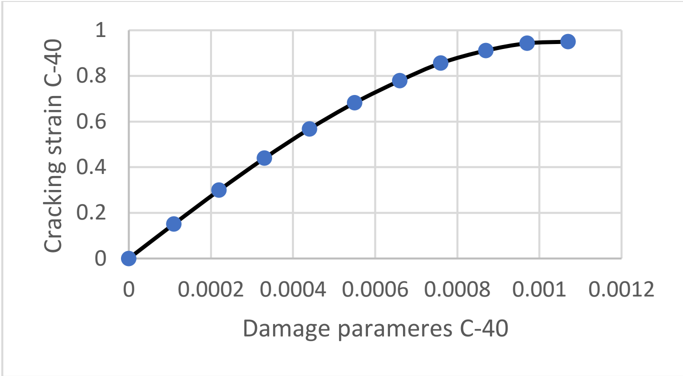

Figure A. 4 Concrete compression damage for C40

Concrete C50

Table A. 7 Density and Elastic for C50

Density	2.50E-06
Elasticity	
Young's modulus	Poisson’s ratio
37000	0.2

Table A. 8 Concrete Damaged Plasticity for C50

Dilation angle	Eccentricity	fb_o/fc_o	k	viscosity parameter
36	0.1	1.2	0.6	0.001

Table A. 9 Concrete Compression Hardening for C50

Yield stress	Inelastic strain
23.2	0
39	0.00055
51.1	0.00109
57.5	0.00164
58	0.00182
57.3	0.00203
55.1	0.00224
51	0.00245
44.8	0.00266
36.1	0.00287
31.8	0.00294
27.54	0.00326
23.43	0.00393
19.55	0.00504
15.97	0.00665
12.75	0.00883
9.95	0.01164
7.62	0.01511
5.81	0.0193
4.55	0.02425
3.87	0.03
0.4	0.1

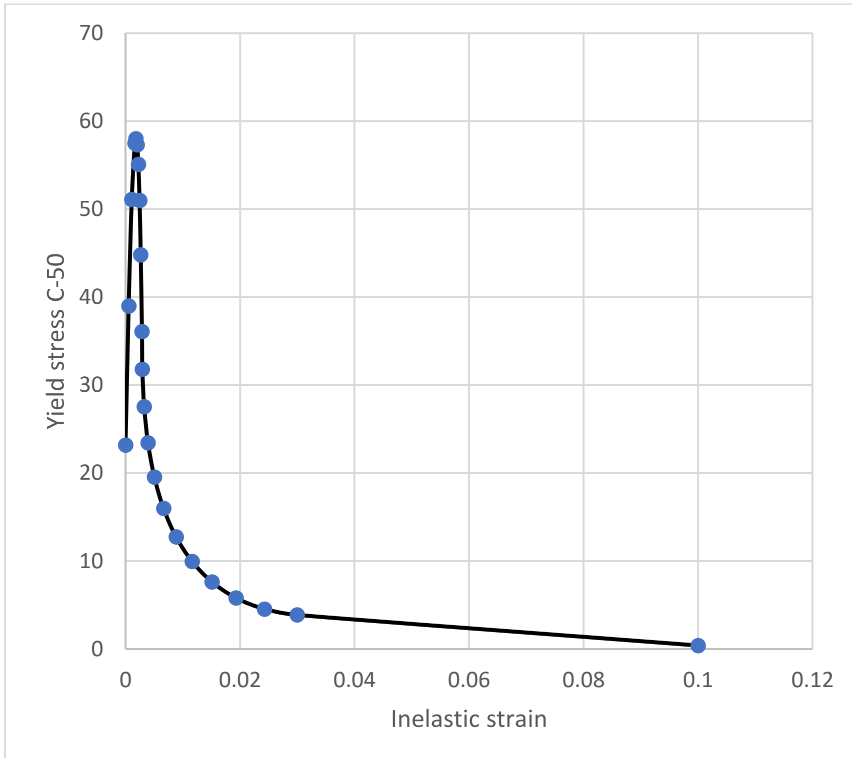

Figure A. 5 Concrete compression hardening for C50

Table A. 10 Concrete Compression Damage for C50

Damage parameter	Inelastic strain
0	0
0	0.00012
0	0.00034
0	0.00071
0	0.00088
0.012	0.00111
0.0507	0.00138
0.1205	0.0017
0.2269	0.00208
0.377	0.00252
0.4517	0.00271
0.5252	0.00314
0.596	0.00392
0.6629	0.00513
0.7247	0.00685
0.7802	0.00912
0.8285	0.01199
0.8687	0.01553
0.8999	0.01977
0.9216	0.02475
0.9333	0.03052
0.9931	0.10062

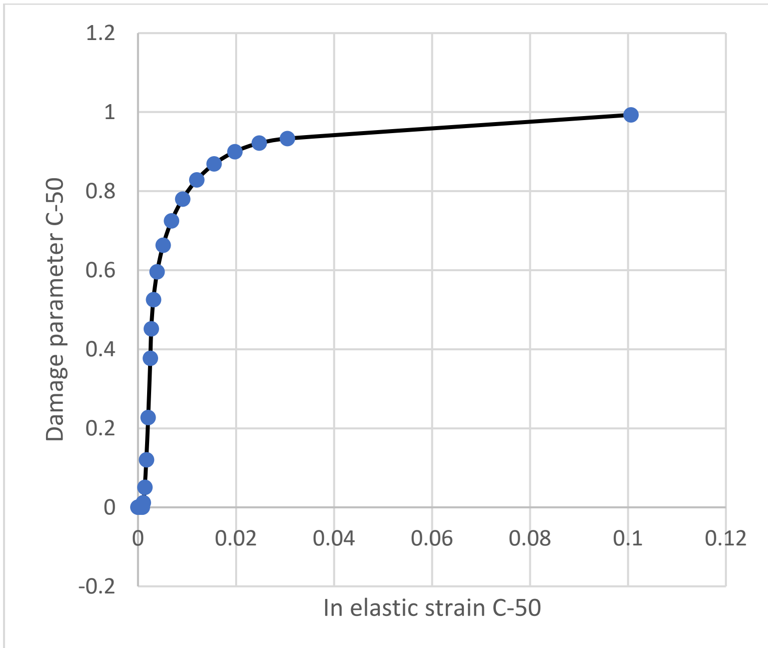

Figure A. 6 Concrete compression damage for C50

Table A. 11 Concrete Tension Stiffening for C50

Yield Stress	Cracking Strain
4.1	0
3.48	0.0001
2.87	0.0002
2.3	0.0003
1.77	0.0004
1.3	0.0005
0.91	0.0006
0.59	0.0007
0.36	0.0008
0.23	0.0009
0.21	0.001

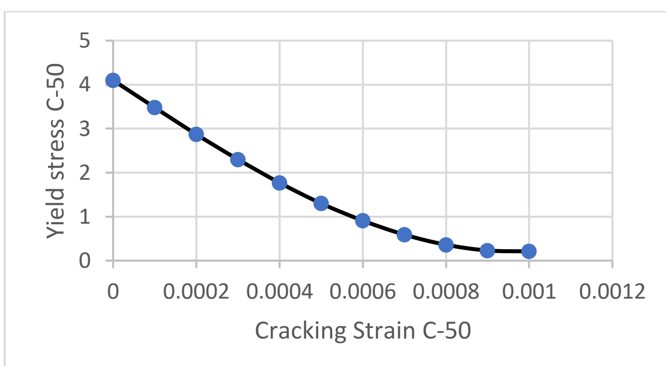

Figure A. 7 Concrete tension stiffening for C50

Table A. 12 Concrete Tension Damage for C50

Damage Parameter	Cracking Strain
0	0
0.1514	0.00012
0.299	0.00023
0.439	0.00035
0.5678	0.00046
0.6821	0.00058
0.779	0.00069
0.856	0.00079
0.9111	0.0009
0.9427	0.001
0.95	0.00111

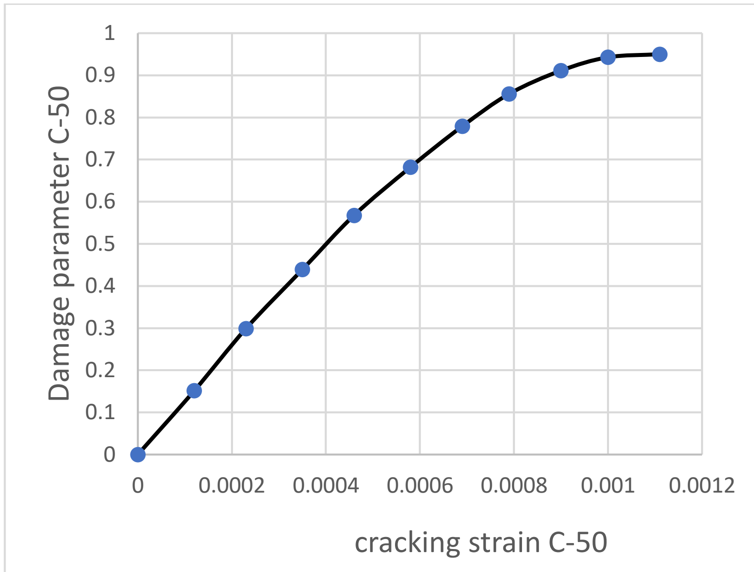

Figure A. 8 Concrete tension damage for C50

Concrete C60

Table A. 13 Density and Elastic for C60

Density	2.50E-06
Elasticity	
young's modulus	Poisson’s ratio
39000	0.2

Table A. 14 Concrete Damaged Plasticity for C60

Dilation angle	Eccentricity	fb_o/fc_o	k	viscosity parameter
36° &40°	0.1	1.2	0.6	0.001

Table A. 15 Concrete Compression Hardening for C60

Yield stress	Inelastic strain
27.2	0.000709
49	0.001397
61.2	0.001923
67.5	0.00243
68	0.0026
67.3	0.002791
65.1	0.00298
61	0.003171
54.8	0.003359
46.1	0.003542
41.8	0.003615
37.54	0.00368
33.4	0.003737
29.5	0.003786
25.97	0.003827
22.75	0.003863
19.95	0.003893
17.62	0.003916
15.82	0.003934

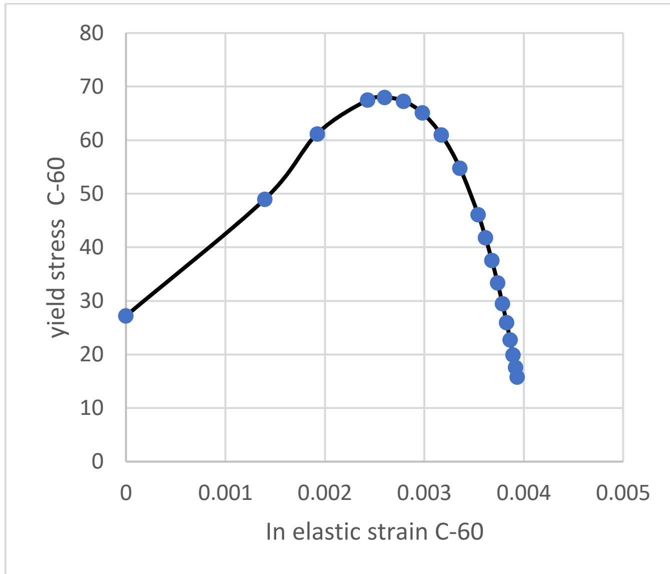

Figure A. 9 Concrete compression hardening for C60

Table A. 16 Concrete Compression Damage for C60

Damage parameter	Inelastic strain
0	0
0	0
0	0.000129992
0	0.000452517
0	0.000607769
0.010294	0.000819077
0.042647	0.001072231
0.102941	0.001384251
0.194118	0.001752996
0.322059	0.002191784
0.385294	0.002390764
0.447941	0.002580071
0.508824	0.002758163
0.566176	0.002921623
0.618088	0.003066543
0.665441	0.003196481
0.706618	0.003308014
0.740882	0.003399778
0.767353	0.003470113

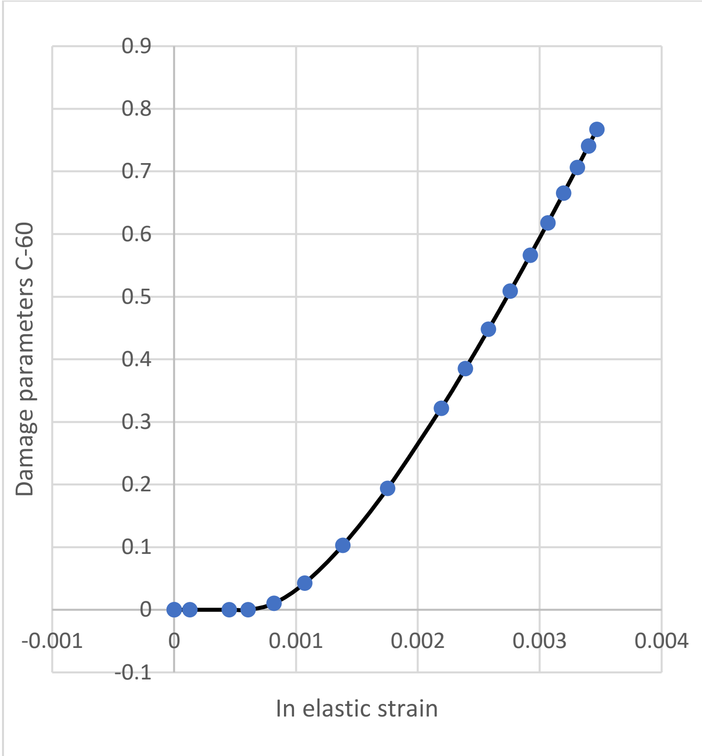

Figure A. 10 Concrete compression damage for C60

Table A. 17 Concrete Tension Stiffening for C60

Yield Stress	Cracking Strain
4.4	0.000113
3.58	0.000273
3.21	0.000338
2.933333	0.000387
2.5	0.000512
2.1	0.000627
1.65	0.000757
1.25	0.000872
1	0.000944
0.733333	0.001354
0.5	0.001506

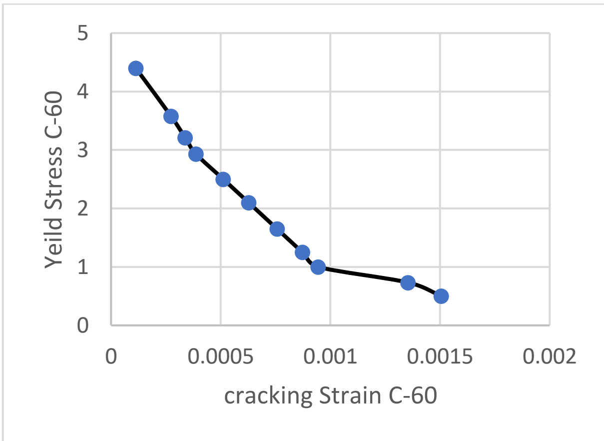

Figure A. 11 Concrete tension stiffening for C60

Table A. 18 Concrete Tension Damage for C60

Damage Parameter	Cracking Strain
0	0
0.186364	0.000181
0.270455	0.000256
0.333333	0.000312
0.431818	0.000448
0.522727	0.000573
0.625	0.000715
0.715909	0.00084
0.772727	0.000919
0.833333	0.001335
0.886364	0.001493

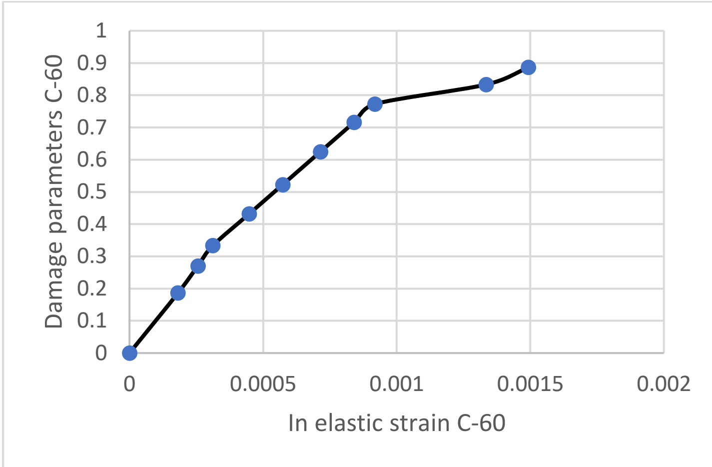

Figure A. 12 Concrete tension damage for C60

A.2 steel beam and reinforcement bar

Table A. 19 Density and Elastic for structural steel and reinforcement part

Density:	7.85E-06
Elastic: structural steel	
Young's modulus	Poisson's ratio
210000	0.3

Table A. 20 Plastic for structural steel

Yield Stress	Plastic strain
530	0
540	0.059852
550	0.064229
560	0.068838
570	0.073687
580	0.078785
590	0.084139
600	0.089758
610	0.095649

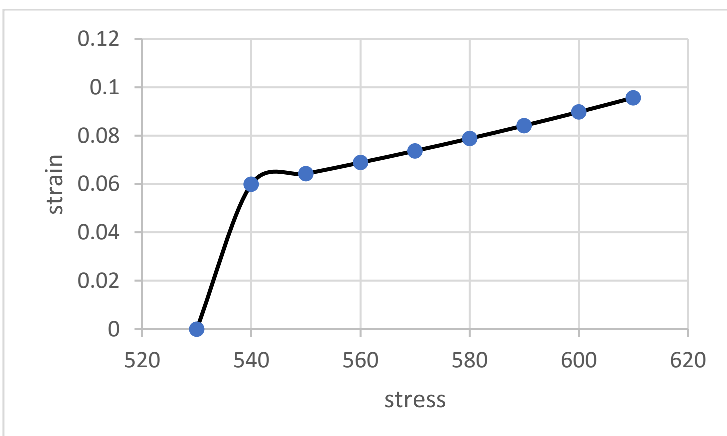

Figure A. 13 Plastic for structural steel

A.3 profile steel sheet

Table A. 21 Density and Elastic profile steel sheet

Density:	7.85E-06
Elastic: structural steel	
Young's modulus	Poisson's ratio
210000	0.3

Table A. 22 Plastic for profile steel

Yield Stress	Plastic strain
355	0
360	0.029225
370	0.032473
380	0.03598
390	0.039761
400	0.043827
410	0.048194
420	0.052874
430	0.057882
440	0.063233
450	0.068942
460	0.075023
470	0.081493
480	0.088366
490	0.09566

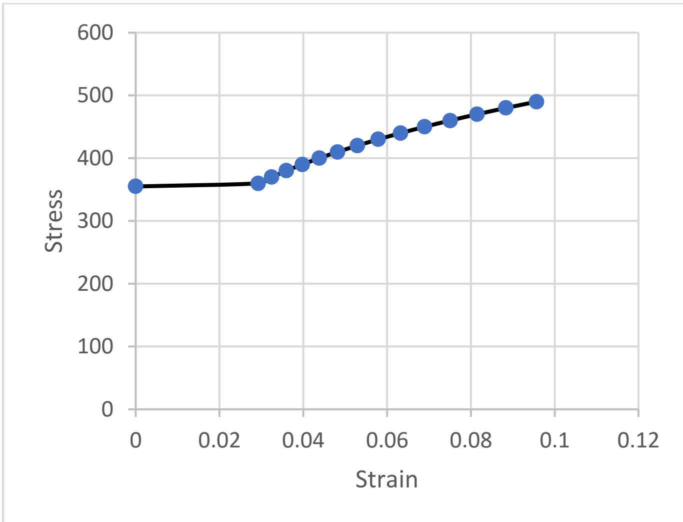

Figure A. 14 Plastic for profile steel

A.4 demountable shear connector

Table A. 23 Density and Elastic for connector

Density:	7.85E-06
Elastic	
Young's modulus	Poisson's ratio
210000	0.3

Table A. 24 Plastic for connector

Yield Stress	Plastic strain
325.7	0
370.4	0.0015
391.5	0.0054
422.2	0.0341
451	0.0686
465.2	0.0985
475.3	0.1199
497	0.1646
523.2	0.216
546.5	0.2596
582.8	0.3239
636	0.4113
639.2	0.4162

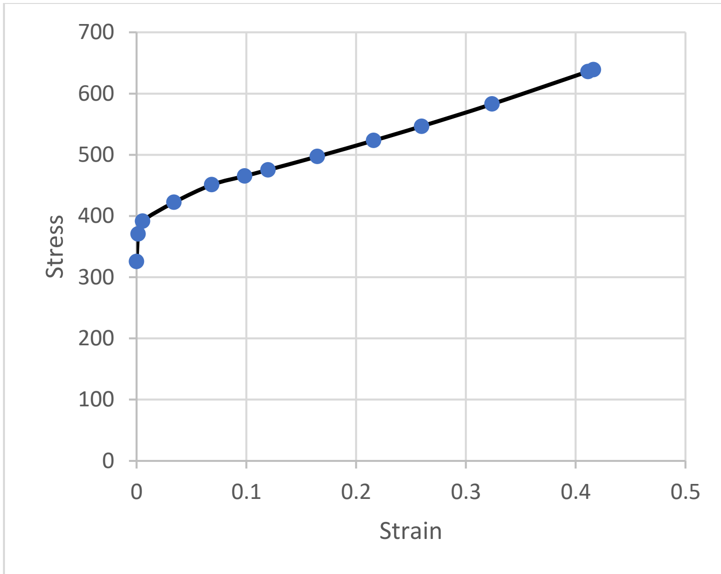

Figure A. 15 Plastic for connector

Table A. 25 Ductile Damage for connector

Fracture Strain	Stress Triaxiality	Strain Rate
0.7278	-1	1
0.2678	-0.33333	1
0.1624	0	1
0.1398	0.1	1
0.1203	0.2	1
0.0985	0.33333	1
0.0767	0.5	1
0.0362	1	1
0.0081	2	1

Table A. 26 Ductile Damage Evolution for connector

Damage Variable	Displacement
0	0
0.008	0.0492
0.063	0.1521
0.153	0.2702
0.238	0.3704
0.354	0.5185
0.492	0.7194
1	0.7308

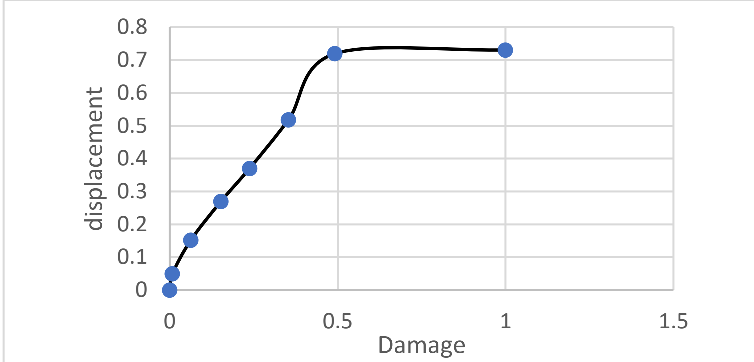

Figure A. 16 Ductile damage evolution for connector

Table A. 27 Damage evolution

Type:	Displacement
Softening:	Tabular
Degradation:	Multiplicative

Table A. 28 Shear Damage for connector

Fracture strain	Shear Stress Ratio	Strain Rate
0.08	1.732	0.1

Table A. 29 Shear Damage Evolution for connector

Displacement at Failure load	Exponential Low Parameter
0.3	0.7

Table A. 30 Damage evolution

Type:	Displacement
Softening:	Exponential
Degradation:	Multiplicative

B. ANNEX - OUTPUT DATA FROM ABAQUS

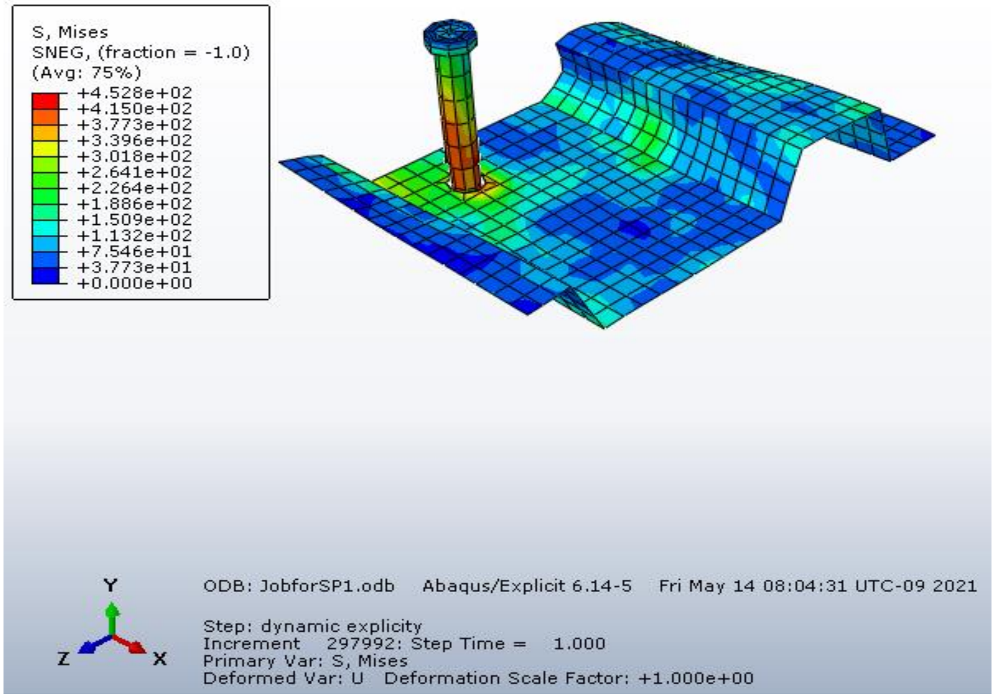

Figure B. 1 Failure modes of sheet and connector of specimen 2

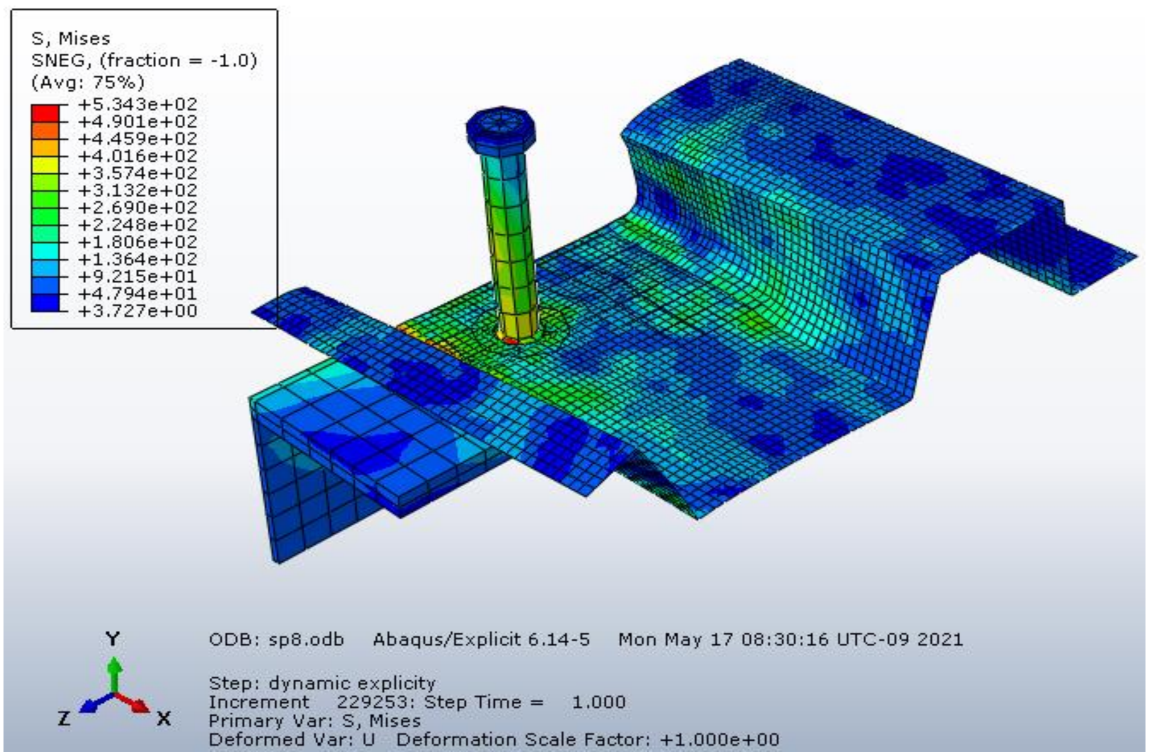

Figure B. 2 Failure modes of sheet and connector with beam of specimen -8

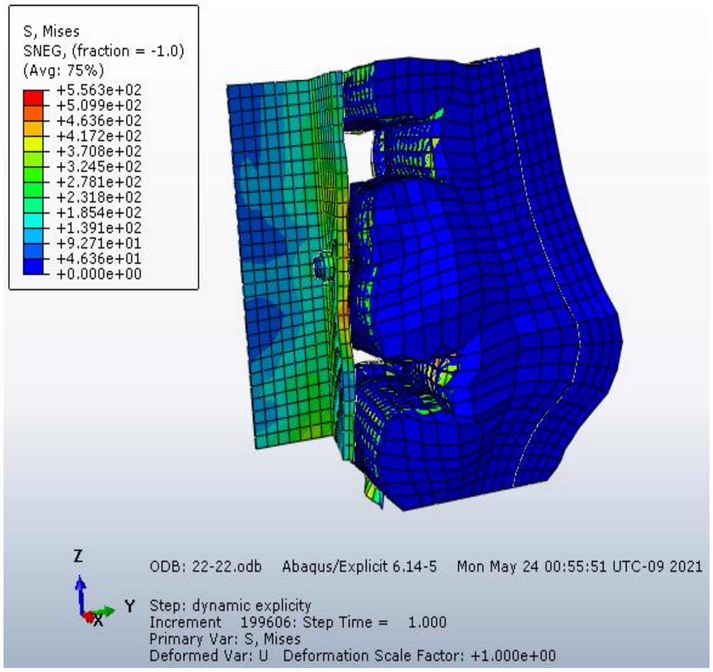

Figure B. 3 Failure modes models of specimen -22

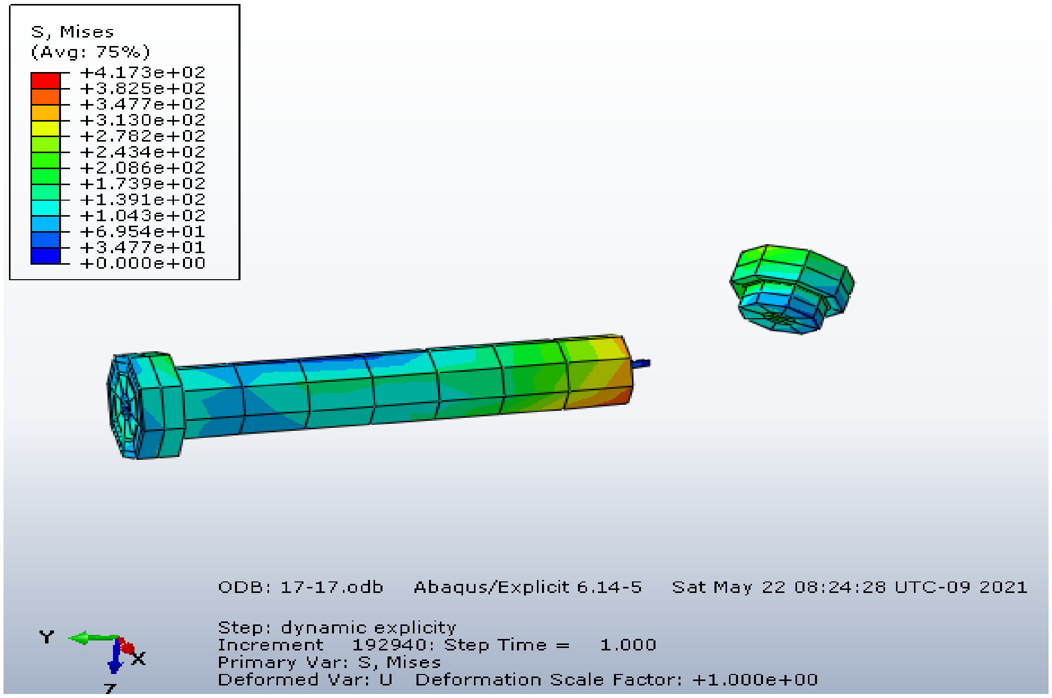

Figure B. 4 Failure modes of connector of specimen -17

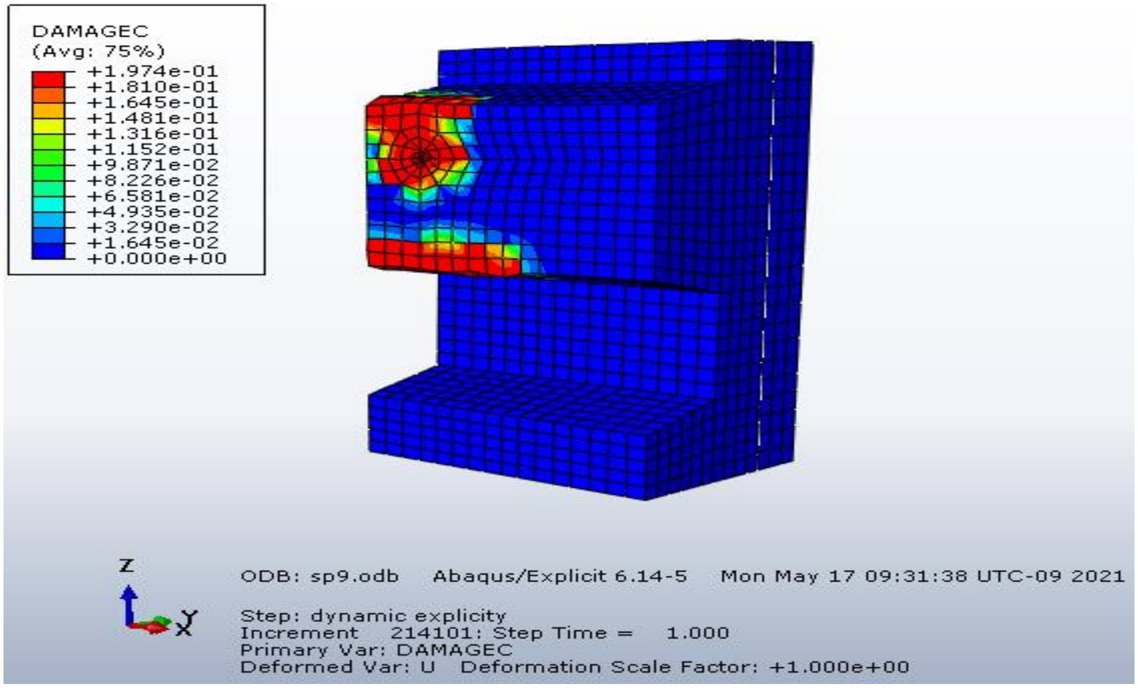

Figure B. 5 Crushing modes open trough shapes of specimen -9

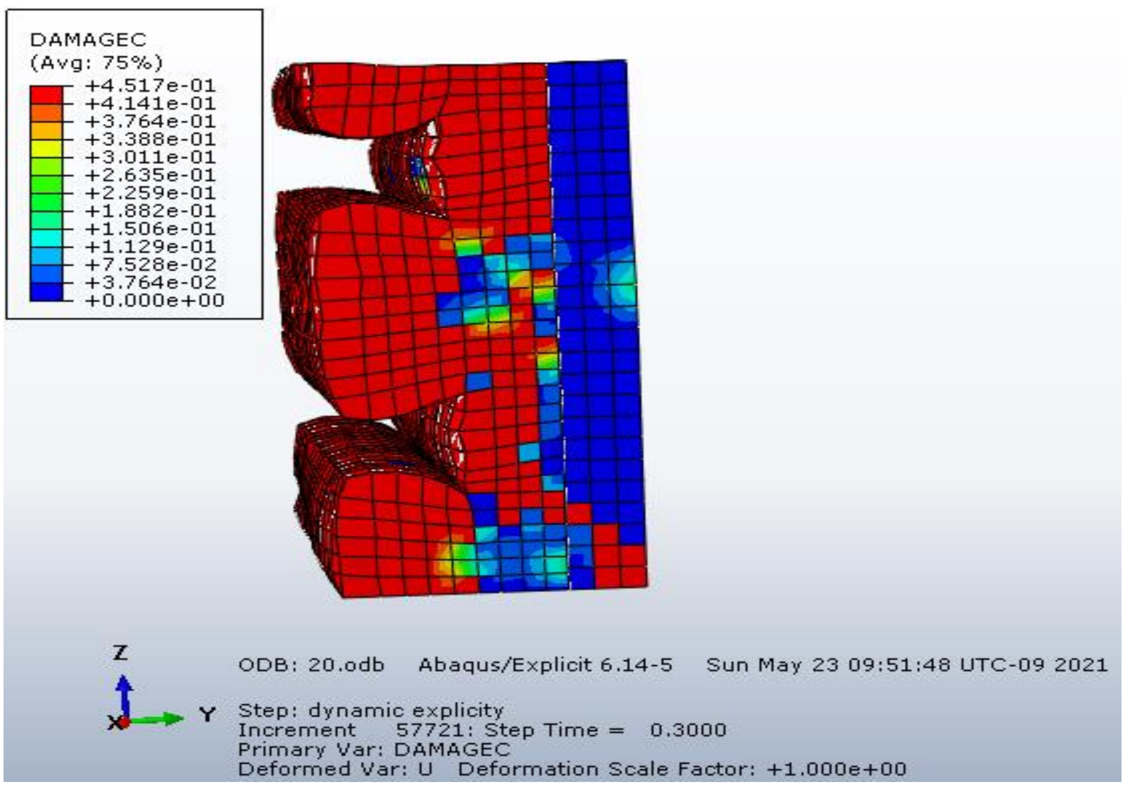

Figure B. 6 Crushing mode of re-entrant shapes of slab of specimen -19