“Comparative study of Headed Stud shear connector and U-shaped shear connector through numerical modelling”

A Dissertation Submitted in Partial Fulfilment of the Requirement for the Award of the Degree of

Master of Engineering

in

Structural Engineering

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July 2022

DECLARATION

I hereby declare that this is a bonafide work which is presented in this seminar report entitled **"Comparative study of Headed Stud shear connector and U-shaped shear connector through numerical modelling"** as per the requirements for the award of Master of Engineering in Structural Engineering, submitted in the Department of Civil Engineering, Thapar Institute of Engineering and Technology (TIET), Patiala. This work is carried out under the guidance of **Dr. Prem Pal Bansal** and **Mr. Abhishek Sharma**. It is declared that this work is original and has not been submitted anywhere else for the award of any other degree or certificate.

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This is to certify that, to the best of my knowledge and belief, the aforementioned assertion made by the concerned student is true.

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ABSTRACT

Shear connectors are one of the most important elements required for the composite construction. Composites structure usually fail at the shear between two structural elements i.e. steel and concrete. Hence, many researches has been carried out to enhance its properties so that the bond between two composites can be strengthened. In addition to increasing the strength of concrete, cost of shear connector also needs to be minimized hence stud connector has been modified into U-shaped connector.

In this project, a 3-D FEM model of pushout test of Headed stud and U-shaped shear connector has been simulated in ABAQUS and analysis has been carried out under gradual loading. The obtained result of headed stud is compared with the experimental value. The same material properties of headed stud is allotted to U-shaped connector and the analysis was carried out. The output of both connectors were contrasted.

It was found that the load bearing capacity of U-shape connector is 20% more than that of headed stud shear connector. The load-slip behaviour of headed stud and U-shape connector are found to be similar. The stress pattern at different location on U-shape connector follows a symmetric pattern from mid-point where as in case of headed stud it is maximum at roots and decreases while moving towards the other end.

From the economic point of view U-shape connector can be more economical than that of headed stud connector as former can provide more bearing capacity as compared to the latter one on same mass. From the optimisation of U-shape connector it can noted that 36 percent less volume of steel is required than that of headed-stud connector for the same peak load bearing capacity.

ABBREVATIONS

2-D	: Two Dimensional
3-D	: Three Dimensional
AASTHO	: American Association of State Highway and Transportation Officials
AISC	: American Institute of Steel Construction
CDP	: Concrete Damage Plasticity
FEA	: Finite Element Analysis
FEM	: Finite Element Model
FRC	: Fiber Reinforced Concrete
IRC	: Indian Roads Congress
LRFD	: Load and Resistance Factor Design
MPa	: Mega Pascal
mm	: millimetre
N	: Newton
kN	: Kilo Newton

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Chapter 1
Introduction

1.1 Background

Composite construction made of steel and concrete is a rapid, cost-effective, and environmentally friendly approach that has been widely used in high-rise buildings, medium-span bridge decks, and other structures. The transmission of longitudinal shear amid the beam of steel and reinforced slab (concrete) can be accomplished using variety of mechanical devices known as shear connectors during this form of construction.

In composite construction, the seismic response of the structure can be significantly affected by shear connectors among concrete and steel elements. They can be employed to transport the slab's substantial horizontal inertial forces to the structure's primary lateral load resisting parts (Figure 1-1) and bring about the necessary shear connection for composite flexure response. During an earthquake, these shear connectors experiences cyclic loading in reverse direction, according to Hawkins [8]. Shear connections assist in the creation of a composite action by promoting shear transmission between the concrete and steel interface.

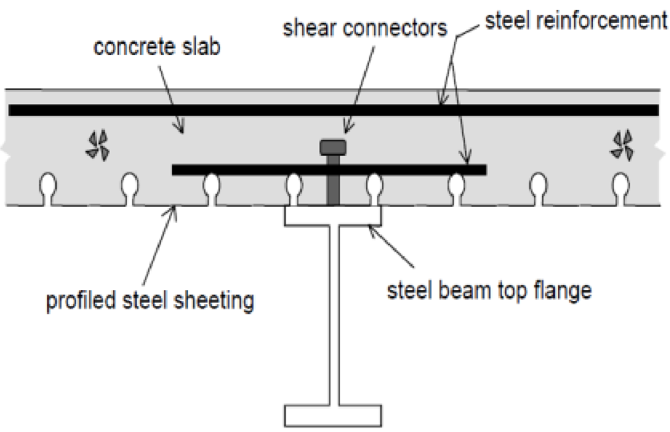

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Figure 1- 1 Shear connector in composite beam [33]

Before the 1970s Floor systems used to encompass steel girders and non-composite slab (concrete) in the development of bridges. But, the present loading condition couldn't be attained through old system of bridges construction. Hence, many bridges built through such construction process is in need to be strengthened or sometimes replaced. Providing a connection between steel girders and concrete slab may be a potential way to reinforce the strength of floor systems because it enables them to behave as a composite structure. As against the initial non-composite situation, composite behaviour enables the present concrete slab and steel girder to perform as an individual entity whereas, the concrete slab and the steel girders performs individually in flexure in case of non-composite girder. Thus, the use of shear connectors for attaching the two different components of structure enhance the bearing capacity of the girders by over 50% with respect to non-composite girder. Attachment of the slab of concrete to the steel girder, helps in transferring of shear forces at the interface of steel-concrete. This help in achieving benefit of composite action in structure. Before the concrete slab are casted, shear connectors and steel girder are connected through the means of welding so as to induce composite action between steel and concrete.

There are various forms of shear connectors among which stud-type connectors are most ordinarily utilized in construction of composite structure. They are experiences axial forces as well as flexural while resisting forces developed at interfaces using dowel action. The stiffness as well as strength of different elements of the shear connector are determining factor for force transfers in it. Hence, the design parameters such as stiffness and shear strength of stud connectors needed to be determined before constructional uses. This work is achieved through push-out experiments.

1.2 Objectives

The objective of the thesis has been to understand the method of finite element analysis using ABAQUS also investigate the mechanism of force so that a reliable, safe and practical design method can ensured through the numbers of simulations.

The main aims of the research are.

- To validate the experiment data of headed shear connector using the numerical simulation in ABAQUS.

- To compare the above obtained data of headed shear connector with Omega shear connector with the help of numerical simulation. (The mass of shear connector used in model are kept constant).
- Optimisation of U-shape shear connector through the change in its parameter to equal the peak load of headed stud.

1.3. Application of Shear connector

Shear connector has many uses in modern construction era where two or more than two different material are used in the field of construction of bridges, buildings, etc. to achieve the balance between economy and strength. Some of the application of shear connector are given below

- It can be used for joining a concrete foundation to a upright steel column/beams.
- Connection between composite slab to steel beams can be achieved by shear connectors. (mostly used in composite bridges)
- In case of supporting the interior ceiling by metal roofing, shear connector plays a vital role.
- Shear connector is also used to join load bearing beams to concrete flooring.

Figure 1- 2 Application of shear connector [32]

Chapter 2
Literature Review

2.1. History

Since 1920, regular-weight concrete were being used for developing composite structures and with the increase of research in field of composite construction bridges were preferred to be made up of composite structures from 1950 [36]. AISC specification had introduced essential design provision that leads the involvement of composite structures in the construction of building. During 1965, these provisions were further expanded by Dristroll and Slutter.

In 1971 Headed stud shear connector's behaviour in lightweight and regular weight concrete has been studied by J.G Ollgaard, J.W Fisher, and R. G. Slutter. Young modulus and strength in compression of concrete determines the shear strength of headed stud enclosed within regular-weight and light weight concrete. Hawkins, N. M., (1973) has provided research on the push-out specimens' strength. Later in 1987, Oehlers, R. P., & D. J. and Johnson have suggested some empirical equations. It explains the changes on the shear connector strength due the change in strength of material used. Pushout test has been carried out to determine the change in shear strength of headed stud connector due to transverse reinforcement by Lloyd, Wright, H. D. and R. M., in 1990. It's been reported that rigidity of the transverse reinforcement, rather than its strength, shown to have an impact on shear strength of connector. Static push-out experiments were performed to figure out the strength and ultimate slip of headed stud having large diameter by Shim, C. S.et.al in 2004. A report on the approximate value of strength of stud connector having larger diameter is given in (DD ENV 1994-1-1).

It is reported by Oehlers, D. J. and Park, S. M., (1992) that the transverse reinforcement produces confinement to the concrete surrounding the proximity of studs instead of contributing towards the strength. In 1996 Li, A., and Cederwall, K., had studied the behaviour of headed stud connector as well as their shear stiffness, shear strength and other factors are figured out in both case of high strength concrete as well as normal strength concrete. Headed studs exhibited ductile behaviour in normal strength concrete because the force acting was gradual and slower with respect to headed studs embedded in concete of high strength.

Research work of Bro, M., and Westberg, M., (2004) has used pushout test to explore shear connector on the static loading as well as fatigue loading. Pallares, L. and Hajjar, J. F., (2010) have reported an analysed compilation of experimental studies on push-out samples. After analysing the sample, an adjustment on the equation proposed in AISC, (2005) has been suggested to the adhered experimental practice.

The development of shear connectors in primitive form has been started over 50 years. With the advancement of technology, shear connectors are available in various forms as per requirement. The research in shear connectors has come a long way to reach the stage where it’s utilized in many construction practices such as bridges, buildings and other structures. It’s made the development and maintenance of structures very effective.

2.2 Types of shear connectors

Shear connector design plays a significant role while designing composite beam. Shear connector are often classified on the basis of their function and shear allocation corresponding to the force and deformation. Some of them are listed below.

2.2.1. Rigid shear connectors

IRC:22-2015 has defined, rigid connectors as angles, channels and other form of steel welded to steel section to form composite behaviour, as shown in Fig. 2-1. Adjacent concrete induces the shear on the connector. These type of connector shows minimum deformation when subjected to shear transfer and hence are not suggested for uses.

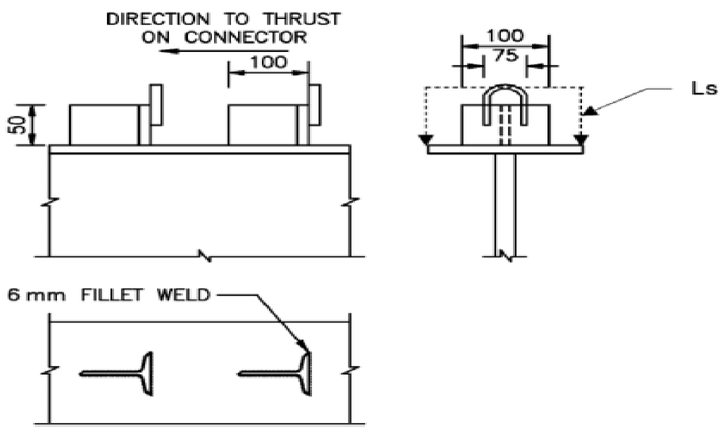
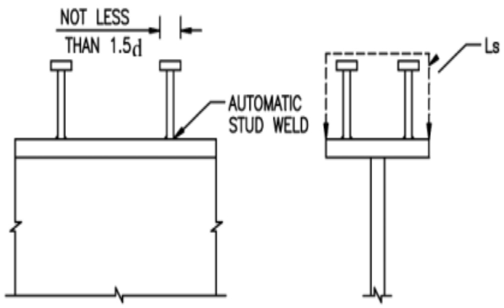


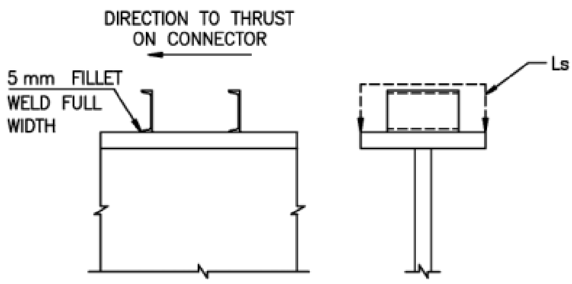
Figure 2- 1 Typical rigid shear connector (IRC:22-2015)

2.2.2. Flexible shear connectors

IRC:22-2015, has defined flexible connectors as of studs, channels and other steel form which are welded to steel section to form elemental action(behaviour). Shear resistance is produced due to bending of shear connectors, preventing girder flange to be lifted from the concrete slab by means of anchorage action. These connector are more ductile than that of rigid connectors and hence does not fail as easily as the rigid connectors.



(a) Stud connector



(b) Channel connector

Figure 2- 2 Typical flexible shear connector (IRC:22-2015)

Many experimental test has been done so far, to understand the behaviour of different shear connector. A review on some of the commonly used shear connector has been enlisted below.

2.2.3. Headed Studs

Headed studs shear connectors are the most ordinarily used in construction. These connectors not only helps in shear transfer but also prevents the uplift of concrete, because it is connected by means of welding to steel and it behaves like resisting connector having a head. The practice of these connector has few constraint when structure is subjected to fatigue loading. For the installation of these connector in site, welding equipment and power generator to operate the welding equipment must be available. In case of high strength concrete (more than M30) the strength of the composite section is constrained by headed stud cross section strength. So, higher grades of concrete won’t be beneficial for this shear connector.

Various equation has been postulated for determining the strength of headed shear connector after many researches by Viest in 1956. Pushout specimens test of different size and stud gap were carried out in full scale during the initial research on this connector meted by Viest. For gauging out the shear capacities of stud two test i.e. composite beam and pushout have been employed. Later in 2002 Ellobody develop a model based on finite element with non-linear behaviour to research on the behaviour of headed shear connector. The research was again continued by Ellobody and Lam in 2005. The result obtained were computed and compared with the data used in the then code of practice like BS5950 (Standard, 1994) and AISC (AISC, 2005a) by Lam and EI-Lobody in 2001. The data obtained from the analysis of finite element are compared with results of the experiment administered by the authors. The result shows that the capacity of shear connector, mode of failure as well as load slip behaviour of similar to that of experiment value and so was concluded that finite element model can also provide proper result with accuracy.

Again in 2002, to replicate the structural behaviour of shear connector (headed stud), the linear and non-linear behaviour of material was also taken into consideration to develop another finite element model by Ellobody. Parametric studies on the model has been carried out by varying the concrete strength and diameter of stud.

The function of stud’s root is to convey the horizontal shear force engaging at the interface of concrete and steel, whereas the function of head is to restrain the uplift occurring in the slab. Shear strength and ultimate shear strength of headed stud is determined by the cross section area of the stud. It is also affected by young modulus and compressive strength of concrete.

IRC:22-2015, has stated that the design resistance, Qu of stud shear connectors needs to be calculated as described below:

$$Q_u = \frac{0.8 f_u . \pi . d2/4}{\gamma_v} \leq \frac{0.29 \alpha . d^2 \sqrt{f_{ck(cy)} . E_{cm}}}{\gamma_v}$$

where,

$$\alpha = 0.2\left\{\frac{h_s}{d}+1\right\} \text{ for } 3 < \frac{h_s}{d} < 4 \quad \text{and} \quad \alpha = 1.0 \text{ for } \frac{h_s}{d} \geq 4$$

E_{cm} = Secant modulus of elasticity of concrete

Q_u = design strength of stud in newton (N)

γ_v = partial safety factor for stud connector = 1.25

d = diameter of the shank of the stud in millimeters (mm)

f_u = ultimate tensile strength of the stud material $\leq$ 500 N/mm^2

$f_{ck(cy)}$ = Characteristic cylindrical compressive strength of concrete = 0.8 *fck*

h_s = nominal height of stud in millimeters (mm)

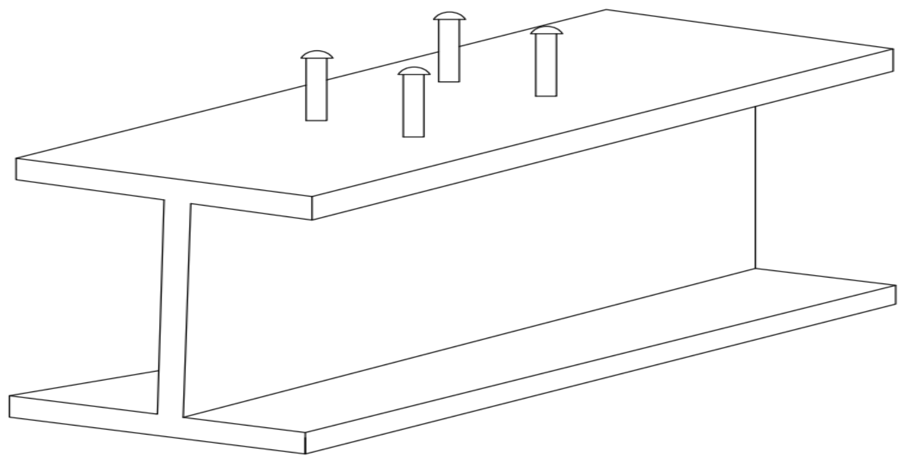


Figure 2- 3 Headed stud shear connector (Mahdi et al.,2012)

The elastic modulus along with the strength in compresson of the concrete is considered as critical variables that governs the capacity of the stud shear connector. These two elements sum up the weld's overall strength. This equation is constrained by the connector material's tensile strength stated by Kwon in 2009.

Field staff may have safety issues due to the dense allocation of shear connectors given the limited area on the top flange. Using large studs with a diameter of above 25 mm can therefore provide a variety of benefits and amenities for composite bridges as illustrated by Lee in 2005. On studs larger than 25 mm, several studies are carried out. Some of them include Hanswille and Sedlacek's push-out tests on 25 mm studs performed in 2003. It was concluded that it is safe to use the equation provided in the AASHTO LRFD bridge design specification to calculate ultimate strength from the push-off tests performed on 31.8 mm studs by Badie in the year 2002. Topkaya conducted research on the use of shear stud strength at young concrete ages in 2004. A study was conducted on the stud shear connection in high - performance concrete by Li and Krister; An and Cederwall in 1996.

Stud connections are affected by a variety of factors. The stud's height, the diameter of the shank, the concrete's compressive and tensile strengths, elastic modulus, and casting direction are among the factors that are most crucial. The material qualities of the filling material and the bedding height must also be taken into consideration when examining the precast deck bridges' shear connection's structural performance as stated by Shim in 2000 & 2001.

2.2.3.1. Benefits of stud connectors

The benefits of shear connectors with headed stud are frequently summed up as follows: good anchorage in concrete, quick welding, ease of placing reinforcement bars through the slab, resistance to concrete slab uplift from the conventional sized head, simplicity of large-scale production, and suitability for use in steel deck slabs. As suggested by Lungershausen in 1988, factors that should be taken into account for the load-bearing capability of studs include the concrete behind the weld collar, the friction forces in the interface of the composite section, and the tensile force acting within connector shaft and shear and bend force resistance in the connecting end of stud.

2.2.3.2. Drawbacks of stud connectors

The large slip in the interface caused by the impact load cannot be sustained by this type of connector, which exhibits non-ductile behaviour in these sort of connectors. Severe concrete crushing at the connector's root actively lowers the concrete's modulus. Concrete crushing or stud shearing failure are also potential causes of shear connection failure. The Connector’s strength is decided by the tension created by the applied load on the steel flanges and the concrete slab's bottom surface shattering restraint, as well as the restriction on concrete expansion caused by the transverse reinforcement.

2.2.4. Channel connectors

Due to the very reliable standard welding technology employed to weld these connectors, channel connectors may not require inspection and testing like bending tests of headed studs. Load bearing capability of channel shear connector is greater compared to shear connector with headed stud. As result of this, many headed studs can be substituted with fewer channel connectors as stated by Bagheri and Maleki in 2008. The test findings of full size pushout samples were reported by Viest et al. in 1952. This fundamental study's primary goal was to observe the behaviour of the channel shear connection and evaluate whether using channels as shear connectors was feasible or not.

In 2006, an experimental examination employing specimens with various channel length and size under monotonic loading was carried out to confirm the accuracy of the design code equations for the strength of channel shear connections by Pashan. According to an experimental study conducted under monotonic and low-cycle fatigue loading, a number of researchers such as Maleki et al., 2008; Güney et al., 2011; Jumaat et al., 2011; Shariati et al., 2010; 2011; predicted the behaviour of channel shear connectors enclosed in a solid concrete material slab. They also suggested an effective numerical model based on the finite element method to simulate the push-out test of channel connector

Push-out samples were constructed of unreinforced concrete, reinforced concrete, fibre reinforced concrete (FRC), and designed cementitious composite were subjected to testing (ECC). When compared to the reversed cyclic shear strength of many of these samples, it was discovered that their monotonic strength is reduced by between 10 and 23%. The outcomes also showed a slight influence of the polypropylene fibres on the specimens' changes the load-displacement behaviour and shear strength of the samples. (FRC specimens). The use of polyvinyl alcohol fibres (ECC specimens) by Maleki and Bagheri in 2008, however, led to a notable improvement in the ductility and strength characteristics of channel shear connectors.

The shear capacity of channel shear connectors encased in a solid reinforced concrete slab exposed to monotonic stress based on the finite element model was further investigated by comparison with experimental test results and verification against data provided in North American design codes. Parametric investigations utilising the nonlinear model were carried out to assess the changes in concrete strength, the direction of the channel, and channel dimensions. The analysis found that in order to determine the ultimate strength of channel shear connectors, important factors such as the concrete's strength, the channel length, and the thicknesses of the channel's web and flange are needed (as opposed to the earlier assumption that the height of the channel section is sufficient). Maleki and Bagheri found in 2008 that the shear connector's stiffness and ultimate strength can alter depending on how the channel is oriented.

Using both analytical and experimental methods, Maleki and Mahoutian also looked at the ability of channel shear connectors contained in regular and polypropylene concrete in 2009. Prior to being able to forecast the shear capacity of channel connectors in polypropylene concrete, a thorough parametric research was conducted. It was suggested to employ an equation in design codes to determine the shear capability of these connections used in polypropylene concrete.

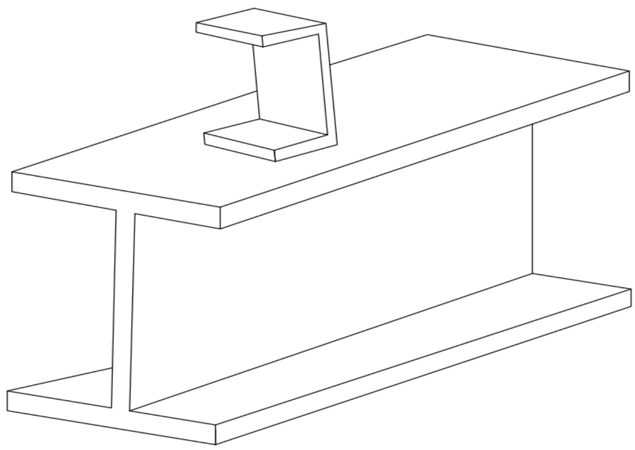


Figure 2- 4 Channel shear connector

IRC:22-2015 has stated that the design resistance of the channel connector needs to be calculated as described below:

$$Q_n = \left[20b.(h)^{3/4}.(f_{ck(cy)})^{1/3}\right]/\gamma_v$$

where,

Q_u = design strength of channel in newton (N)
b = length of the channel in millimeters (mm)
h = height of the channel in millimeters (mm)

When employing channel shear connectors, you should keep the following advice in mind.

1. The channel's height, h, must not be larger than 150 mm, or 20 times the web's thickness, whichever is minimum.
2. The width of the channel, b, must not be greater than 300 mm.
3. The top flange of the channel's underside should be 30 mm or more clear of the bottom reinforcement.
4. No more than half the thickness of the plate should be used as the leg length of the weld joining the channel to the plate.

2.2.5. Perfobond Rib

In response to the unsatisfactory behaviour of shear studs caused by fatigue issues brought on by live loads on composite bridges, Leonhardt, Andra, and Partners developed a special type of connector known as the perfobond rib as stated in (Leonhardt et al., 1987). A welded steel-plate with several holes makes up this connector (Figure 6). (Ahn et al., 2010). Dowels are formed as concrete flows through the rib openings, creating resistance in both the horizontal and vertical orientations. According to the results of the conducted experimental experiments, this shear connector is a workable replacement for the headed stud connector as stated by Ahn in 2010, Kisa in 2011, and Jumaat in 2011. Ferreira in 1998 claimed that this connector had been originally utilized in building structures. The concrete-steel connection provided by Perfobond rib and its promotion of a superior anchorage of the internal columns hogging moment have encouraged its implementation.

Concrete is poured through the perforations in the perfobond web to secure the perforated ribs, or they can simply be placed on top of the transverse reinforcing bars that are installed on them.. According to a study by Zellner in 1987, 24 numbers of 19 mm diameter studs put in three lines or 18 numbers of 22 mm diameter studs placed in two lines make up a 1 m length of perfobond connector.

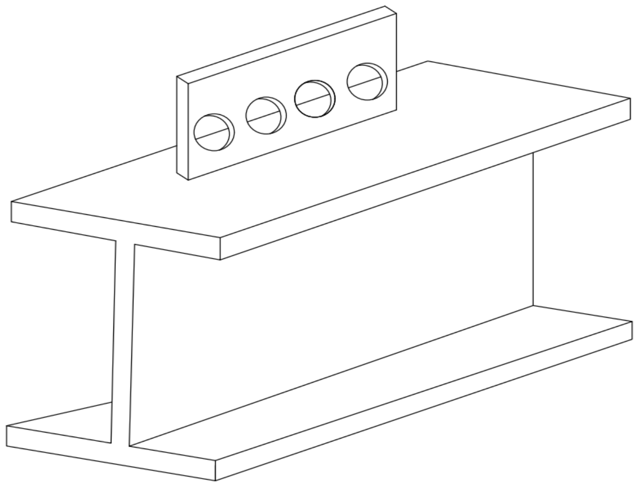


Figure 2- 5 Perfobond-rib shear connector

The concrete slab that is subjected to shear is connected by concrete dowels that pass through perforations in the perfobond ribs and is reinforced transversally. Design resistance is given by:

$$q_u = 0.590tA_c\sqrt{f_c'} + 1.233\, A_{tr}f_y + 2.871\, nd^2\sqrt{f_c'}$$

In which, f'_C denotes the strength of the concrete in compression ;

The yield strength of steel is f_Y.;

A_C is shear area of concrete;

A_{tr} refers to the transverse reinforcing area that runs through the holes;

d represent the perfobond rib hole diameter.

n is the total number of perfobond rib holes.

The number of holes, percentage of transverse reinforcement provided, length, height, and thickness of the plate, as well as the compressive strength of the concrete slab, all have an impact on the perfobond connector's structural reaction.

2.2.5.1. Disadvantages

This type of shear connector's most significant drawback is the challenging location of the transversal bottom slab reinforcementThese connectors have strong capacities for both fatigue resistance and shear resistance.

2.2.5.2. Advantages

By utilising concrete dowels, transverse rebars in the rib holes, and a concrete end bearing zone, this connector is able to sustain vertical uplift forces and horizontal shear at the steel-concrete interface as stated by Leonhardt in 1987. The recommendation of such connectors is based on the fact that it not only ensures the link between concrete and steel but also encourages a stronger anchorage of the interior columns' hogging moment reinforcing bars. By inserting the reinforcing bars through the perfobond web holes, anchorage on such bars can be created. Similarly, anchorage might be provided by merely superimposing these bars on the transverse reinforcing bars that are utilised on them.

2.2.6. T- Rib Connector

A different connector for headed studs dubbed the T-perfobond was introduced by Vianna in 2009 as part of a study on perfobond connectors. The behaviour of these connections and a select few T-perfobond connectors were compared by the author in another investigation. This connector was developed from the perfobond connector by adding a flange to the plate, which serves as a block. The invention of this T-perfobond connector was driven by the requirement to unite the high strength of a block-type connector with some ductility and uplift resistance resulting from the holes at the perfobond connector web. The T-rib connector detail should reduce the influence of the prying action to prevent an early loss of stiffness in the connection as stated by Ferreira in 2000. The T-rib connectors can be made from residual rolled sections, which could lower the cost and require less welding.

The fabrication of T-rib connectors involves the following four phases: (i) initial profiling, (ii) creating web holes, (iii) flange holes, and (iv) opposite flange saw cut. T-perfobond connectors are more strong and resistant than perfobond connectors for longitudinal plate designs that are identical to them. In complement to all of this benefit, the use of T-perfobond connectors has advantages in terms of labour and material savings because they are made from regular laminated I or H sections.

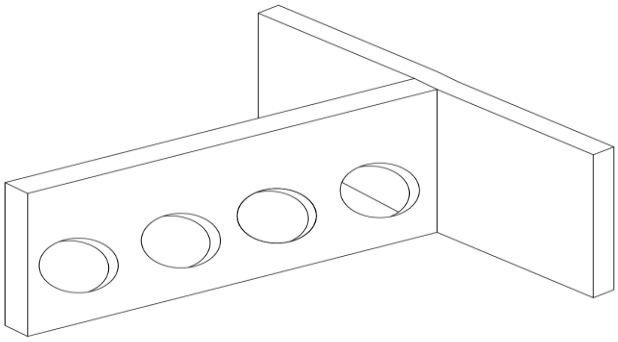


Figure 2- 6 T-rib connector

2.2.7. Oscillating Perfobondstrip

This kind of connector has a higher load capability compared to headed studs and T-shape connectors. However, the performance of this connector in the situation of ordinary strength and normal weight concrete is somewhat unsatisfactory because of the quick reduction in load capacity after the peak. However, the oscillating perfobond strips connectors (Figure 2.7) function effectively because this behaviour doesn't occur when they are used with lightweight concrete, concrete with fibres, or high strength concrete as explained by Rodera in 2008. For oscillating perfobond strip connectors, it is important to analyse the differences between the failure mechanisms for concrete with lower and higher strengths. Steel fibres were added to the concrete, and the results were very favourable.

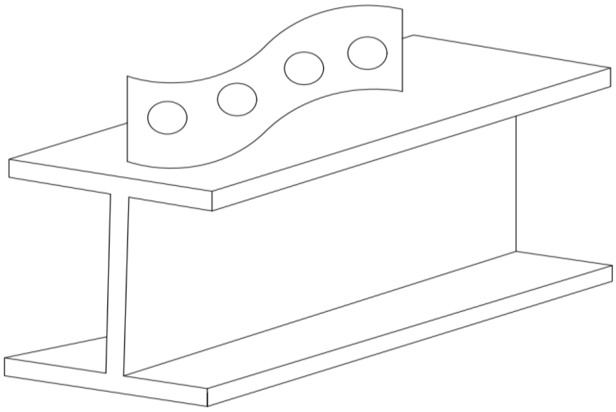


Figure 2- 7 Oscillating-perfobond strip shear connector

2.2.8. Pyramidal shear connectors

A steel plate-concrete composite slab with pyramidal shear connections is expected to have enough bending strength and flexural rigidity for loads during and after construction. Another one of them would be a TSC composite slab, which connects concrete and steel deck lower end using pyramidal shear connectors (Lee and Han, 1998). When such a TSC composite slab is put to a bridge deck susceptible to traffic loads, the fatigue problem should be taken into consideration during design. In particular, the welding of shear connectors may lower the thin bottom plate's fatigue strength (Matsui, 1984).

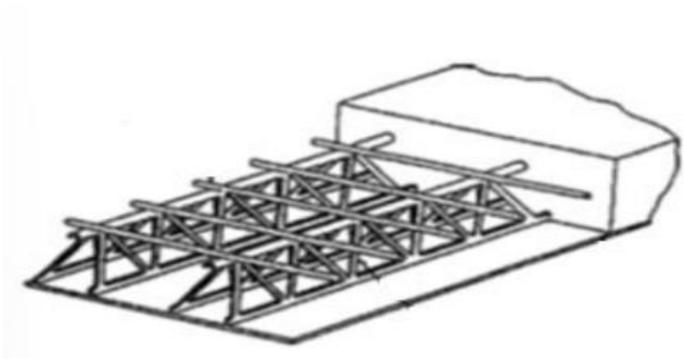


Figure 2- 8 Pyramidal shear connector

2.3 Finite Element Analysis

By using FEA, we can effectively test and forecast how structures will behave while being subjected to static and dynamic loading conditions, which allows us to tackle challenging structural engineering challenges. In many physical problems where the governing differential equations are known, the finite element method (FEM) has been used to numerically solve differential or integral equations. The 3D finite element modelling of stud shear connectors is the focus of a few studies. It is well known that experiments in lab takes time, cost a lot of money, and in some circumstances, may even be unfeasible. The finite element approach, on the other hand, has developed into a strong and practical instrument for the study of a variety of engineering issues.

To replicate the load-slip characteristic of the shear stud without a head in a solid RC slab, Lam and El-Lobody (2005) created a finite element model. The seismic performance of moment-resisting frames made of steel-concrete composite beams with the shear connection was examined by (Bursi et al., 2005). In order to evaluate various modelling assumptions and local effects, 3-D FEM of the substructures were created using the ABAQUS code. Non-linear spring elements were used to track the behaviour of shear connectors as well as the bond-slip between the slab and reinforcing bars.

The modelling of composite beams with full and partial shear connections was the topic of (Queiroz et al., 2007). The shear connections are represented by nonlinear springs in a three-dimensional model that includes all of the major structural characteristics and related nonlinearities (concrete slab, steel beam, and shear connectors). In order to investigate the structural behaviour of simply supported composite beams with large rectangular web holes, Wang and Chung (2008) developed two-dimensional finite element models made up of plane stress elements. The models include both vertical and horizontal springs to represent shear connectors with nonlinear deformation properties.

The long-duration behaviour of composite beams at the limit state of serviceability was modelled using a uniaxial non-linear finite element method presented by Sakr and Sakla (2008). The proposed finite element procedure took into account the creep, shrinkage, and cracking of concrete slabs as well as the nonlinear load-slip relationship of shear connectors. This study presented a mathematical model to simulate composite beams' time-dependent behaviour. Nguyen and Kim (2009) created a precise nonlinear finite element model of the push-out specimen to research the capability of substantial stud shear connectors implanted in a solid slab. The finite element model took into account of the material nonlinearities of headed studs, steel beams, concrete, and rebar.

Individual Finite element model was produced for each connections that are examined in lab by (Julander, 2009). To further understand the connection breaking behaviour, the model findings and test data were compared. Additionally, a preliminary FEM was developed that provides more details on cracking behaviour of each connection when compared to the findings of the tests. Steel plate, welded rebar, and shear studs were modelled using link elements, which are line elements with three degrees of freedom in translation. (Abbu et. al, 2013) created a number of FE models of a laboratory specimen by utilising various ANSYS software techniques. Rigid link elements enhanced the models with full interaction, providing a good depiction for shear connectors.

A total of 9 Pushout test were carried out on mixed shear connectors consisting of perfobond and headed stud by Shuangjie Zheng et al. in 2019. Analytical analysis and a parametric investigation were conducted in light of the experimental findings. To investigate the impacts of connection dimension and material qualities, nineteen push-out tests has been simulated. Increases in stud diameter, rebar diameter, hole diameter, stud strength, rebar strength, and concrete strength all improve the mixed shear connector's shear capacity.

2.3 Pushout Test

The Pushout Test has been one of the preferred lab test carried out for comprehending composite structure and their exclusive characteristics such as shear strength, composite structure's ultimate strength. According to Bro and Westberg; 2004, the push out test assesses the shear connector's static and residual strength, and also fatigue endurance. Depending on how the load is applied over the specimen, the Push out test configuration may be horizontal or vertical. A composite specimen made of a steel piece, a concrete slab, and other materials is included in the test setup. The test apparatus consists of a composite specimen made up of steel sections, concrete slabs, and stud connectors; a loading mechanism made up of a load frame, a load cell, and a loading machine; and a measuring unit made up of dial gauges, LVDTs, strain gauges, and a computerised data acquisition system.

Figure 2- 9 Pushout test arrangement in vertical direction (Vianna et al 2009)

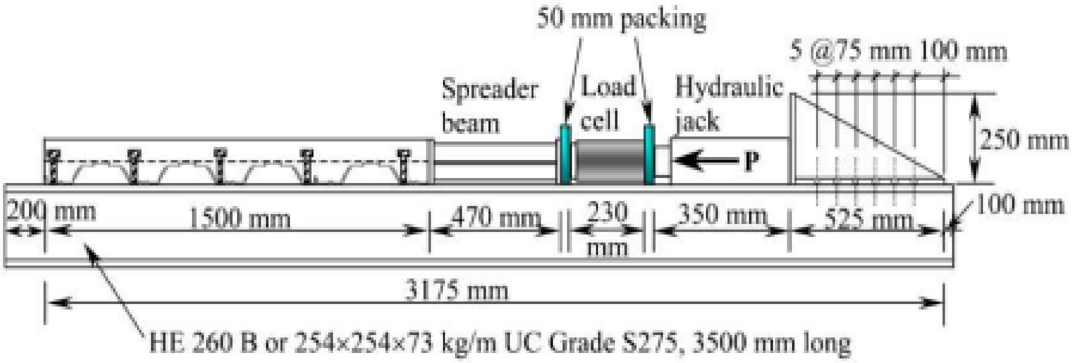


Figure 2- 10 Push-out test arrangement in Horizontal direction (Qureshi et al, 2011)

2.3 Voids in Research

Many researches has been carried out on shear connector till date, especially on headed stud connector. It is necessary to have an economic and efficient design of shear connector so that it could be used in engineering applications. The parametric studies carried out in the researches has significantly increases the shear strength of connector with the increase of diameter of shear connector, length of shank in stud connector, composition of material. But from the literature review, it was observed that to increase the shear strength, cost of the shear stud is also increased.

No researches has been carried out on the economic basis design to change the shape and size of shear stud connector. Hence, an approach is made to increases the strength of the shear connector by changing the design of stud (i.e. an omega shaped shear connector was designed but later it was modified to U-shaped shear connector considering easy implementation in civil engineering field) in this project.

Chapter 3
Methodology

3.1. General

For the motive of achieving the objective we have considered the FEM analysis on two sample; they are 'sample 1' consisting of headed stud as a shear connector and 'sample 2' consisting of U shaped share connector.

The following step are carried out in succession in this project.

I. The modelling of headed stud embedded in the concrete and steel is carried out with the required material property and interaction and a simulation is performed on ABAQUS.
II. The obtained result from step I is validated against the experimental result of [39].
III. The u shaped shear connector is then designed with the same material property and interaction as that of headed stud in ABAQUS. The volume (mass) of u shaped connector is kept same as that of headed stud to make a single parameter constant for comparative analysis.

Modelling and analysis of structure and its elements is carried out in following steps in ABAQUS.

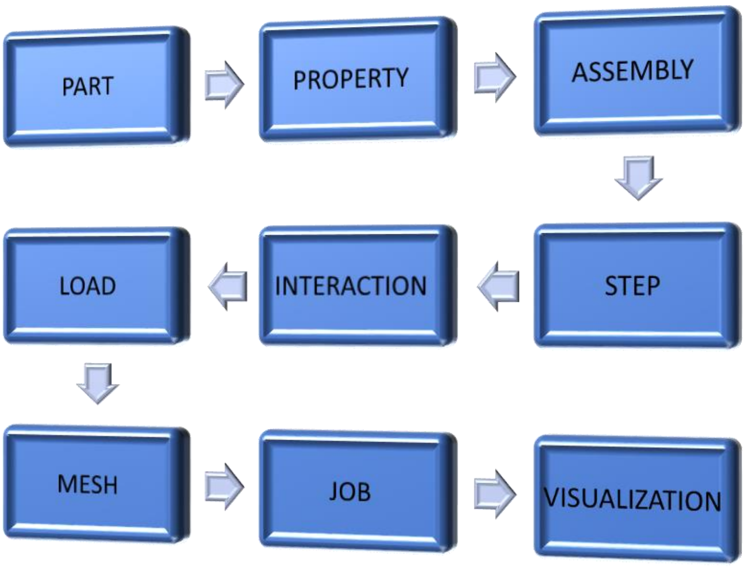


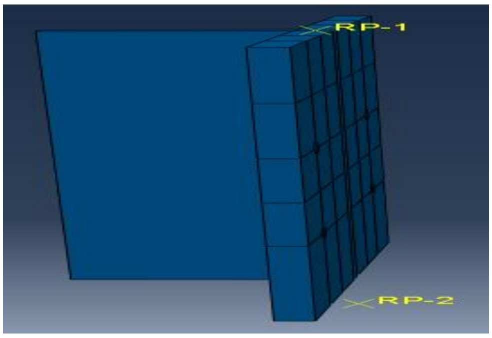
Figure 3- 1 Steps involved in ABAQUS

Half portion of pushout-test from its symmetric axis is considered so that number of elements can reduce leading to the reduction in analysis time without hampering the accuracy of result.

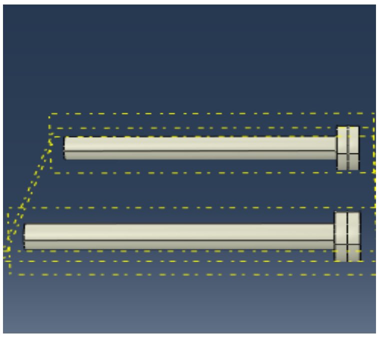
3.2. Sample 1

3.2.1. General Configuration and Assembly

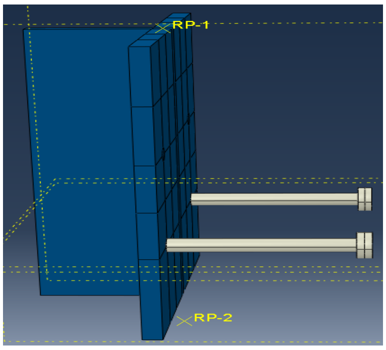
Sample 1 consists of concrete, steel beam, headed-stud, reinforcement bar assembled as shown in the figure below (3-2, 3-3). Concrete has the dimension of 460*560*400 mm^3, steel beam T-section has flange dimension of 460*460*20 mm^3 and web dimension of 20*460*130 mm^3, diameter of reinforcement bar is 12 mm, diameter of stirrup is 8mm. the length of main bar and dimension of stirrups is adjusted such that minimum 50mm cover can be provided to the reinforcement from concrete. In case of headed-stud, shank length is provided as 185mm and head depth is provided 15mm, similarly diameter of head is 42mm and shank diameter is 22mm.



(a) Steel Section

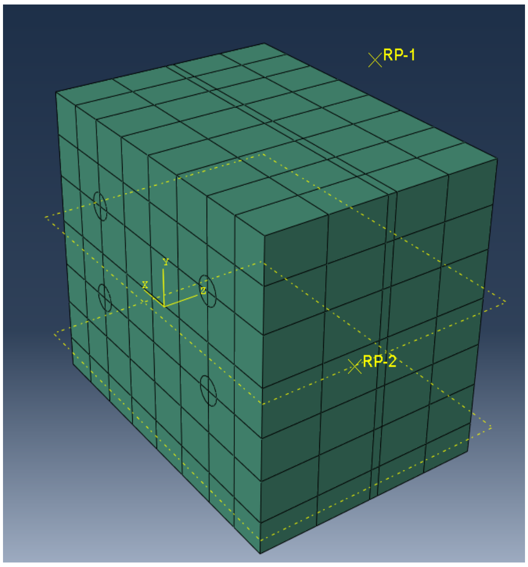


(b) Headed stud

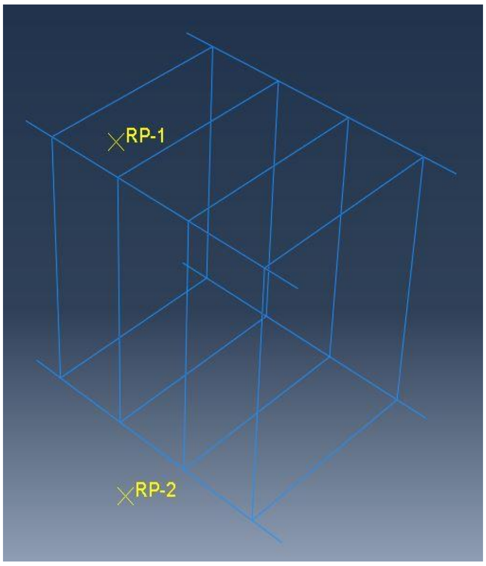


(c)

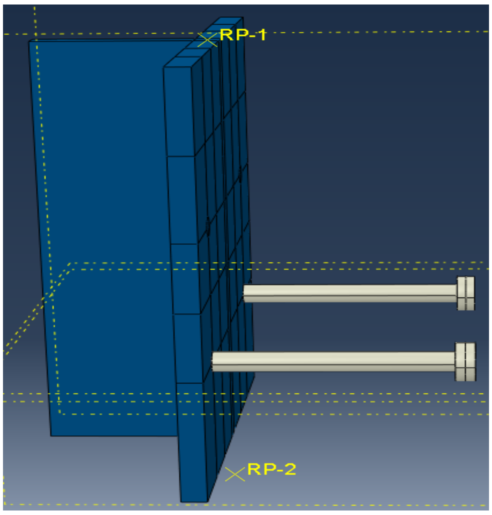
Figure 3- 2 Assembly of headed stud and steel section



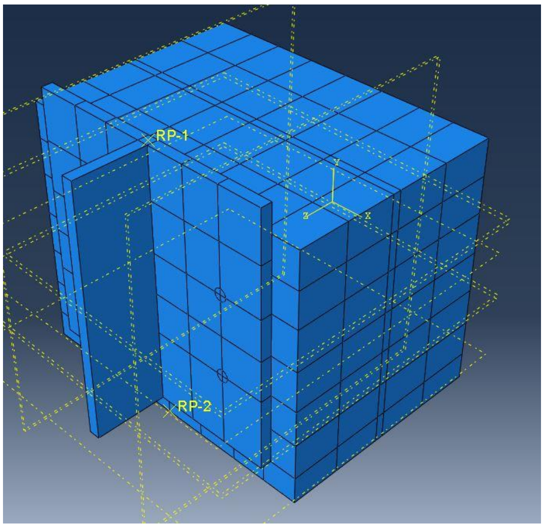
(a) Concrete



(b) Reinforcement



(c) Assembled stud and steel beam



(d) Final assembled Structure

Figure 3- 3 Assembly of sample 1

In modelling concrete, the plasticity hypothesis is frequently applied. However, this hypothesis only works well in the compression zone. Other models are used for tension zones, including the smeared crack model and the fictional crack model. The "Concrete Damage Plasticity" model, which Lubliner and Oliver (1989) created as an adjustment to the Drucker-Prager strength hypothesis and was further improved by Lee and Fenves. It was created as a result of the necessity to incorporate both concepts into a single model (1998).

Dilation angle (Ψ): A necessary variable to describe the flow potential capacity is the dilation angle. The flow potential capacity speaks of the volume change induced because of plastic deformations for solid material Oliver and Lubliner (1989). The most common dilation angle values for concrete range from 25° to 40°. Higher dilation angles result in a marginal increase in initial stiffness and an increase in ultimate load (Nasrin et al., 2018).

Eccentricity: Eccentricity describes the rate at which the asymptote of the flow potential function is reached. For concrete, the default eccentricity value is 0.1.

Poisson's ratio: The proportion of longitudinal extension strain to transverse contraction strain in the direction of the stretching force is known as poisson's ratio. Its value varies from 0 – 0.5 in most of the material.

Young's Modulus: It is defined as the ratio of stress applied to the corresponding strain. The compressive strength of concrete and its Young's modulus of elasticity are often controlled by the same elements, such as the amount of air in the concrete and the porosity of the cement paste, which is connected to the water-cement ratio.

Plasticity number (k): Plasticity number is given by

$$k = E_{c^*} \frac{E_{cp}}{f_c}$$

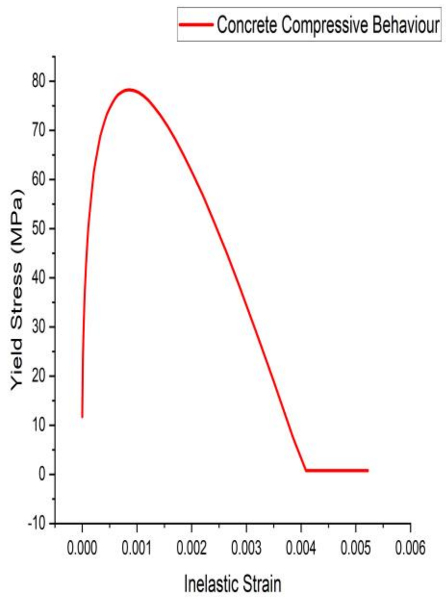
Where, E_c is young modulus of concrete; E_{cp} is strain at which yielding occurs; F_c is compressive strength of concrete.

Viscosity parameter: To avoid strain localisation and numerical instability, this parameter is utilised. The way that columns and beams behave actually determines how the entire structure behaves. Because of this, it is crucial for the safe design of structures that these parts exhibits nonlinear behaviour. The primary inputs needed by the plasticity model are the concrete's plastic, compressive, and tensile properties. By permitting strains to exist outside of the yield surface, this model (CPD) can be regularised using visco-plasticity.

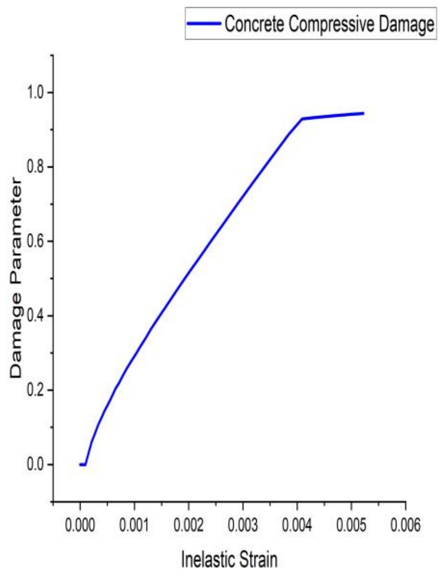
The property of concrete that has been used in simulation are tabulated below.

Density	2.40*E-009 g/mm^3
Young's modulus	41176.80489 N/mm^2
Poisson's ratio	0.2
Dilation Angle	38
Eccentricity	0.1 mm
Fb_o/Fc_o	1.16
K (plasticity number)	0.67
Viscosity Parameter	0

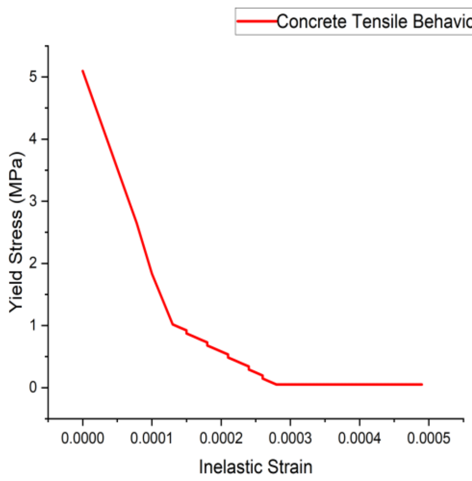
Table 1 Elastic and Plasticity value used in concrete modelling

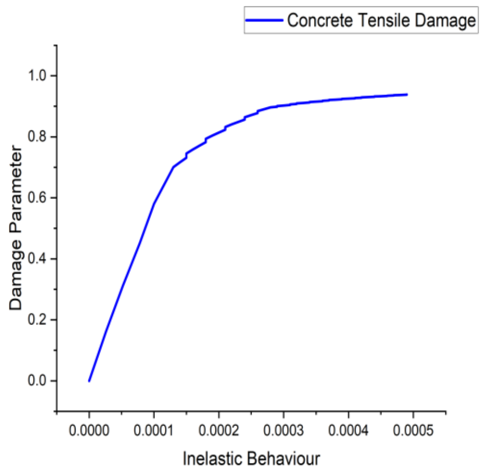


a) Concrete Compressive Behaviour



b) Concrete Compressive Damage





Concrete Tensile Behaviour Concrete Tension Damage

Figure 3- 4 Concrete Damage plasticity Properties represented graphically

The material properties that are used for headed stud, steel section, reinforcement bars and stirrups are enlisted in table below.

Plastic property of			
Steel Section and reinforcement bar		**Headed stud**	
Yield stress (Mpa)	**Plastic strain**	**Yield stress(Mpa)**	**Plastic strain**
410	0	369.0306	0
410	0.003528	380.6633	0.000361
430.66327	0.004704	395.2041	0.000795
447.19388	0.005769	415.5612	0.001532
462.34694	0.006706	433.0102	0.002125
476.12245	0.007676	447.551	0.002616
489.89796	0.008613	464.0306	0.003194
506.42857	0.009678	464.0306	0.003816
525.71429	0.010886	464.0306	0.004639
543.62245	0.012046	464.0306	0.00555
		464.0306	0.010927

Table 2 Plastic Properties of Headed stud, Steel section and reinforcement bar

	Steel section	Headed stud	Reinforment-bar and stirrups
Mass Density	7.85*E-009	7.85*E-009	7.85*E-009
Young’s Modulus	200000	200000	2000000
Poission ratio	0.3	0.3	0.3

Table 3 Elastic properties of steel section, Headed stud, and Reinforcement bar

3.2.2. Steps, Interaction and Loading

Dynamic explicit procedure was applied on the sample to replicate material geometry and non-linearity. Semi-automatic mass scaling was used and at the beginning of the steps,the target time increment of 0.0026 was taken. As a result, accuracy and time required to complete the analysis can be counterbalance. The rate of loading applied was 0.0001m/s.

Interaction between steel and concrete was considered frictionless, a hard contact was applied for the pressure-overclosure at right angle between them and separation was allowed after the contact. Penalty contact method was used with finite sliding in the interaction between headed stud and concrete and steel beam and concrete applying surface to surface contact interaction. Reinforcement are tied to the stirrups and this setup was embedded in the concrete. Tie interaction has been applied in between steel beam and headed stud. Reference point 1 has been coupled with steel beam top surface whereas reference point 2 has been coupled with base surface of concrete.

The reference point 2 has been fixed restraining the moment and rotation in all direction (U1= U2 = U3 = UR1 = UR2 = UR3 = 0). The displacement load has been applied to reference point1 with the total displacement of 10mm in vertically downward direction.

3.2.3. Meshing

The proper support constraint was used in the symmetric layer when designing the typical push-out test. To imitate a steel beam, headed stud, and concrete slab, an 8-node linear brick with decreased integration and hourglass control (C3D8R) was chosen. The meshing of the reinforcing bar was done using a 3-dimensional, 2-node truss element. For the headed shear connectors, a local mesh size of 7 mm was used, whereas a global mesh size of 20 mm was used for the reinforcement, 13mm for steel sections, and 20mm for concrete.

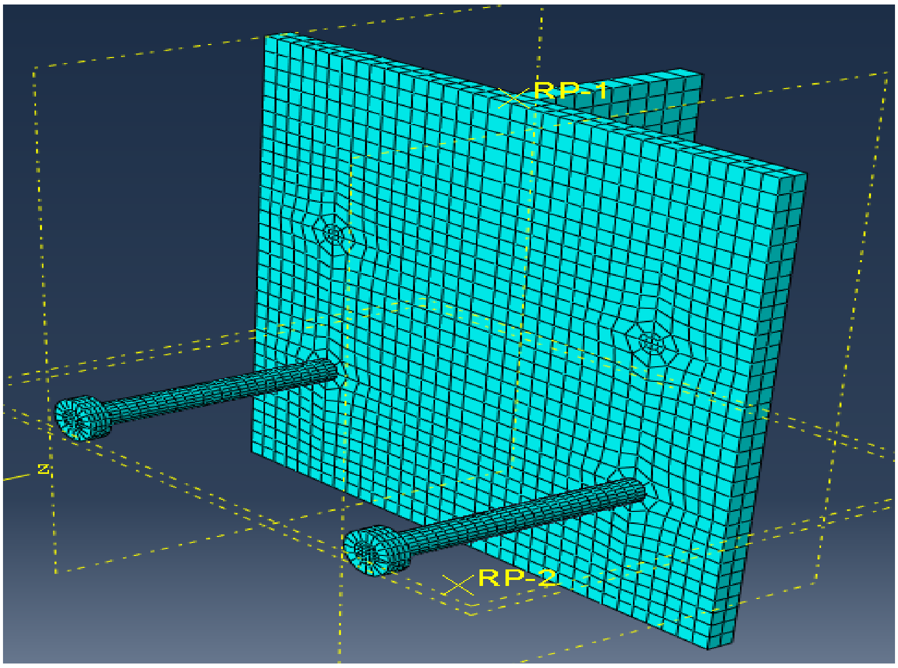


Figure 3- 5 Meshing of Stud and steel beam

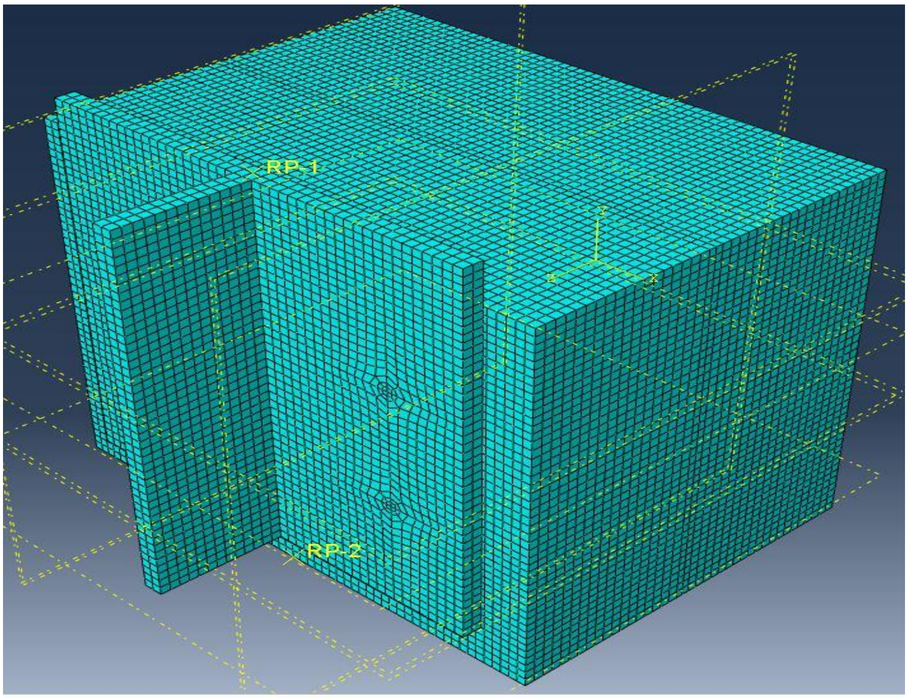


Figure 3- 6 Meshing of assembled structure of sample 1

3.3. Sample 2

3.3.1. General Configuration and Assembly

The assembly setup of sample 2 is shown in the figure (3-6 & 3-7). It consists of U shape shear connector, concrete, steel beam and reinforcement bar. The configuration and material property of steel beam, reinforcement bar concrete is same as that of ‘sample 1’. In case of U-shaped connector, it is configured such that the mass/volume of the U-shaped connector is equal to that of headed stud.

Volume of 1 Headed stud (v) = Volume of shank + volume of head

= 3.1415* 11 * 11* 185 + 3.1415 * 21* 21 * 15

= 91103.5 mm3

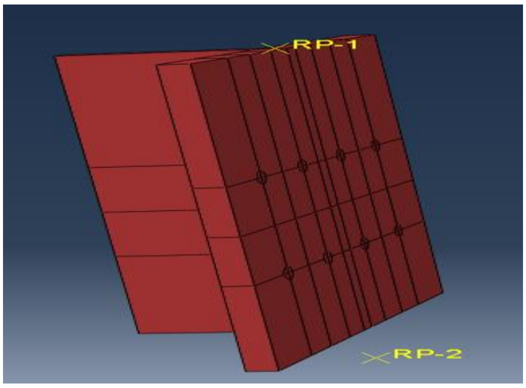
Again,

Volume of 1 U- shaped connector = volume of straight portion + volume of semi-circular portion

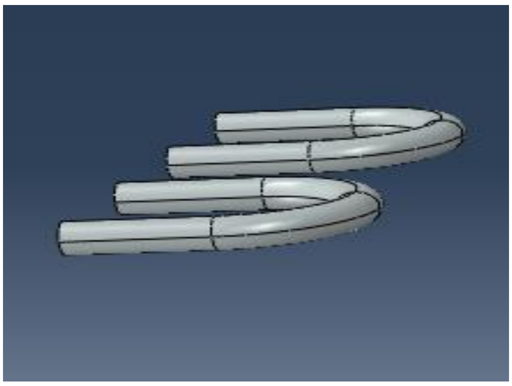
= 2* 3.1415*10*10*66 + 3.1415 * 50 * 3.1415 * 100

= 90812.91 mm^3

In U shaped connector Length of straight portion is 66mm, while length of of semi-circular portion is 157 mm and diameter of bar is 20mm.The material property of the U shape connector is same to that of headed stud used in ‘sample 1’.



(a) Steel beam



(b) U-shaped shear connector

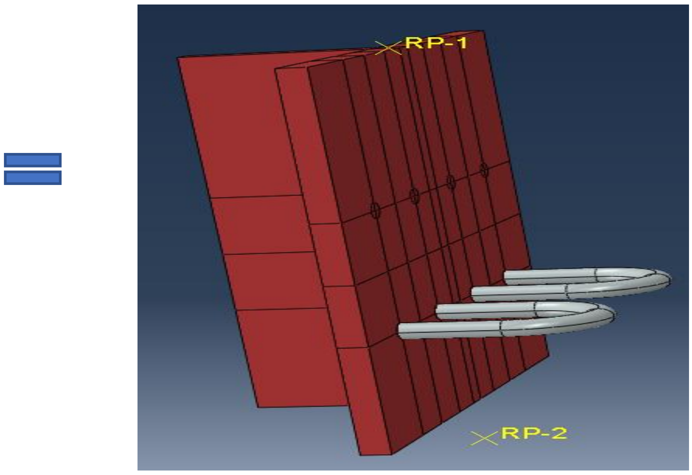
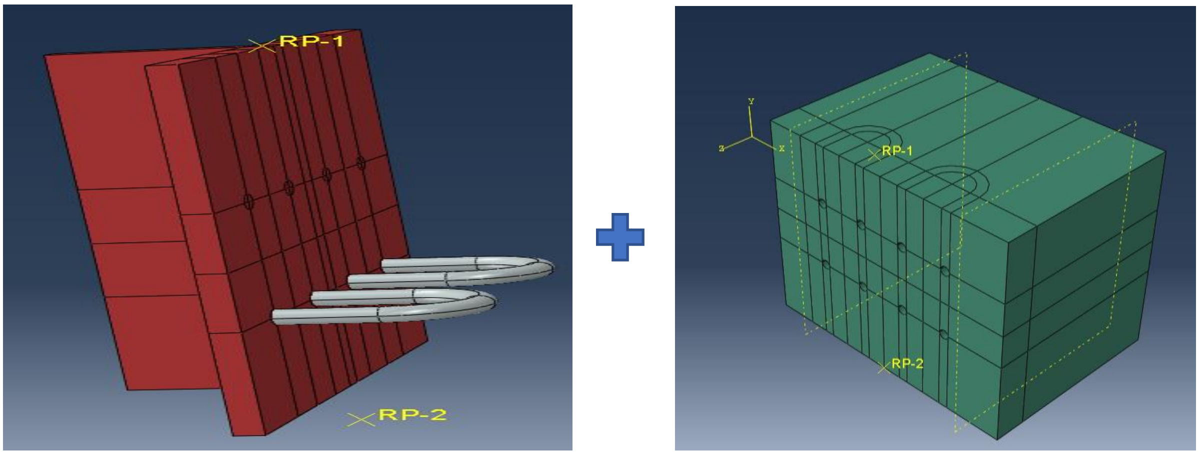
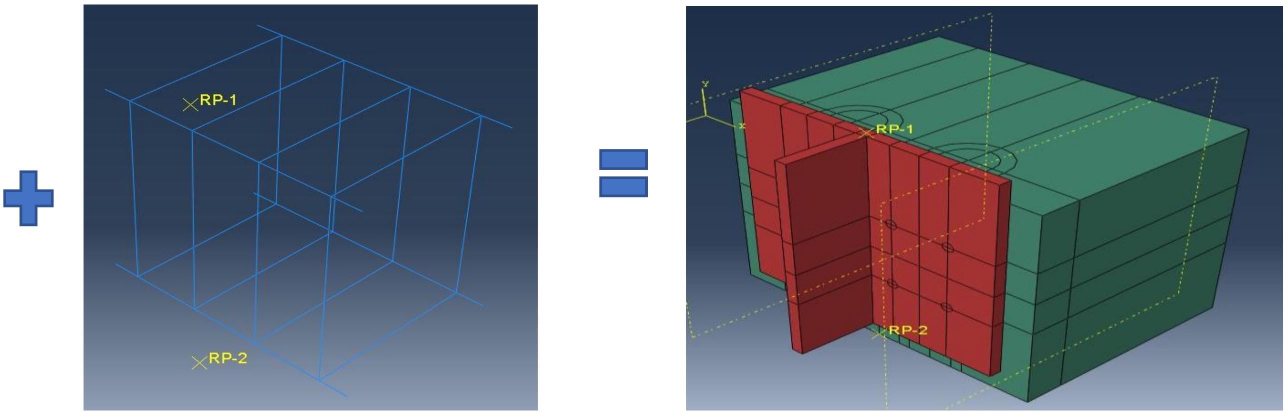


Figure 3- 7 Assembly of U-shaped shear connector and steel beam



(a) Assembled U connector and steel beam

(b) Concrete



(c) Reinforcement

(d) final Assembled strucutre

Figure 3- 8 Assembly of Sample 2

3.3.2. Steps, Interaction and Loading

For the purpose of comparative studies, same procedure of dynamic explicit was applied on the sample to replicate material geometry and non-linearity to that of sample 1. Semi-automatic mass scaling was used and at the beginning of the steps, the target time increment of 0.0026 was assigned. The rate of loading applied was 0.0001m/s.

Similarly, interaction between steel and concrete was considered frictionless, a hard contact was applied for the pressure-overclosure at right angle between them and separation was allowed after the contact. Penalty contact method was used with finite sliding in the interaction between U-shaped connector and concrete; and steel beam and concrete applying surface to surface contact interaction. Reinforcement are tied to the stirrups and this setup was embedded in the concrete. Tie interaction has been applied in between steel beam and U shape connector. Reference point 1 has been coupled with steel beam top surface whereas reference point 2 has been coupled with base surface of concrete.

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3.3.3. Meshing

The proper support constraint was used in the symmetric layer when designing the typical push-out test. To imitate a steel beam, U shaped connector, and concrete slab, an 8-node linear brick with decreased integration and hourglass control (C3D8R) was chosen. The meshing of the reinforcing bar was done using a 3-dimensional, 2-node truss element. For the headed shear connectors, a local mesh size of 7 mm was used, whereas a global mesh size of 20 mm was used for the reinforcement, 13 mm for steel sections, and 18mm for concrete.

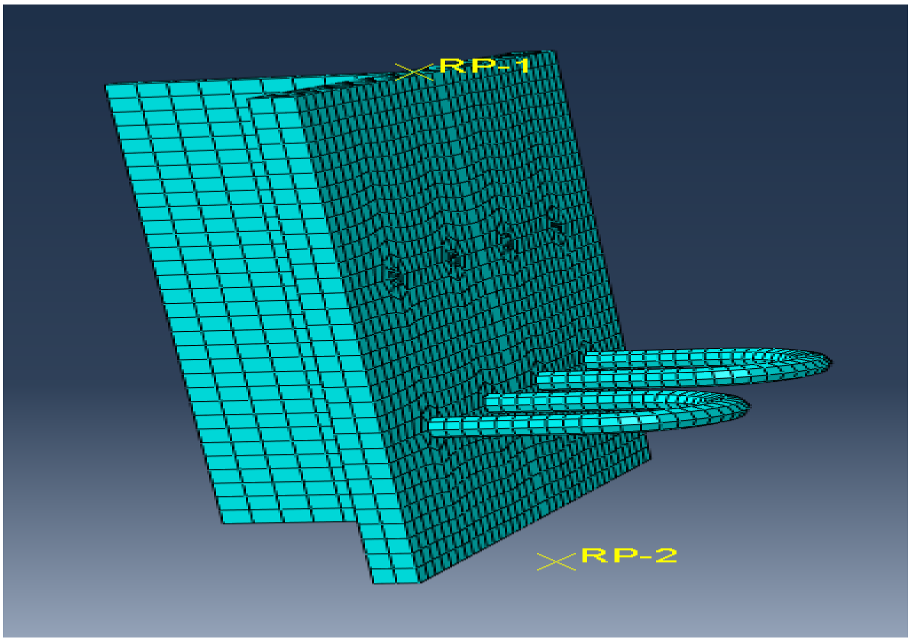

Figure 3- 9 Meshing of U-shear connector and steel beam

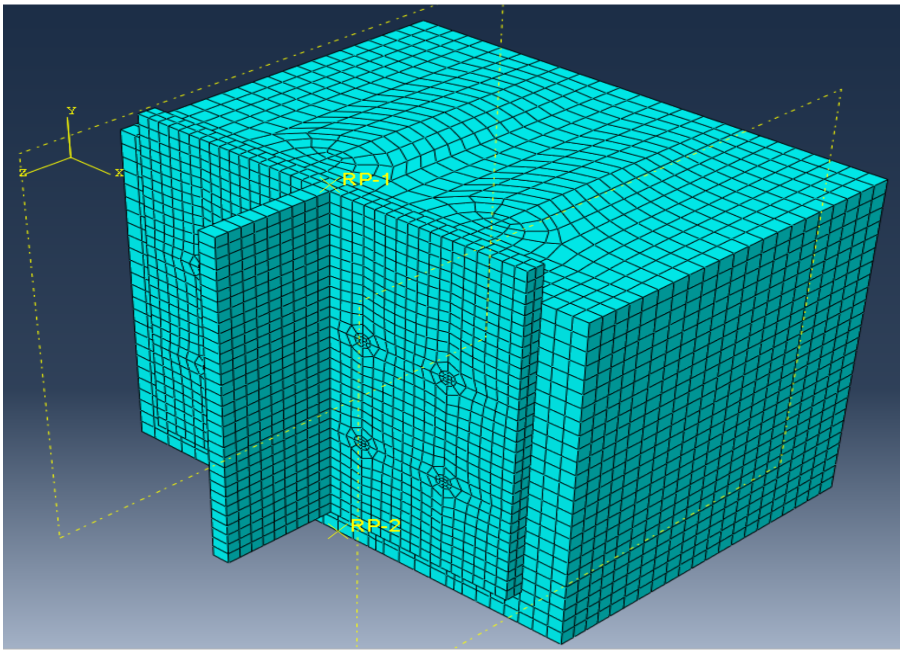

Figure 3- 10 Meshing of assembled structure of sample 2

Chapter 4
Results and discussion

We have considered the experimental result from the paper Zheng, S., Zhao, C., & Liu, Y. (2019).

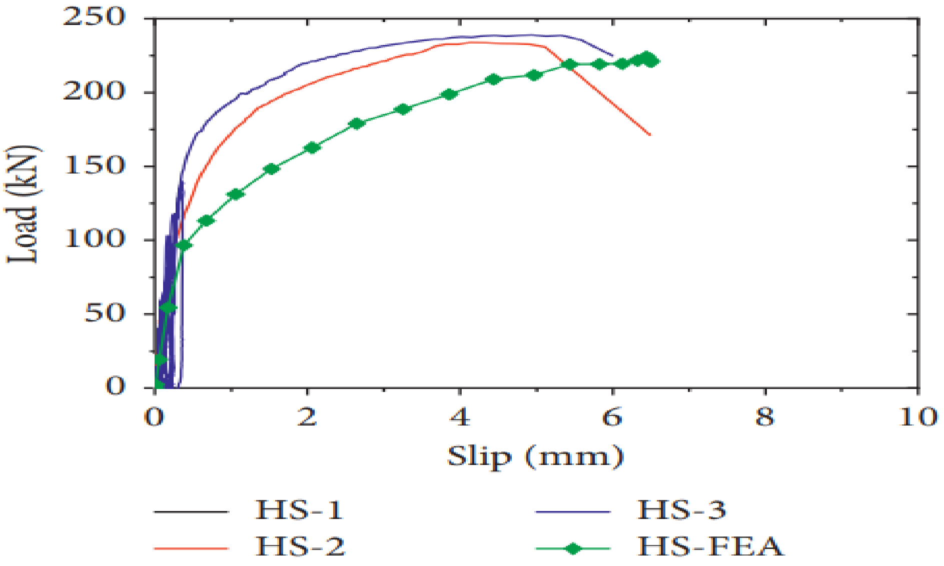

Figure 4- 1 load-slip behaviour of referenced paper [39]

In the figure 4-1 HS refers to headed stud and HS-FEA refers to the Finite element analysis of headed stud. We have computed the result of the paper with the numerical modeled result obtained from ABAQUS.

4.1 Headed stud Connector

4.1.1 Load-slip Responses

The curves obtained from the finite element analysis responded similarly to that of the push-out test outcomes. The analysed shear from finite model was found to be 3% -4% less than that of test result. The peak load value of the headed stud shear connector was determined as 226.86kN. The load-slip figure obtained from analysis consists of stiffness similar to that of experimental result at the beginning stage of loading as in figure 4-2 the stiffness decreases at later stage.

Samples	Tested Shear capacities		Analysed Shear capacity	Comparison	
	V(kN)	V_{avg} (kN)	V_{fea} (kN)	V_{fea}/V	V_{fea}/V_{avg}
1	233.9	236.45	226.86	0.97	0.965
2	239			0.96	

Table 4 Comparison of experimented and FEA analysed Headed stud connector

$R^2_{FEA} = 0.8122$ $R^2_{sample1} = 0.8147$ $R^2_{Sample\ 2} = 0.8796$

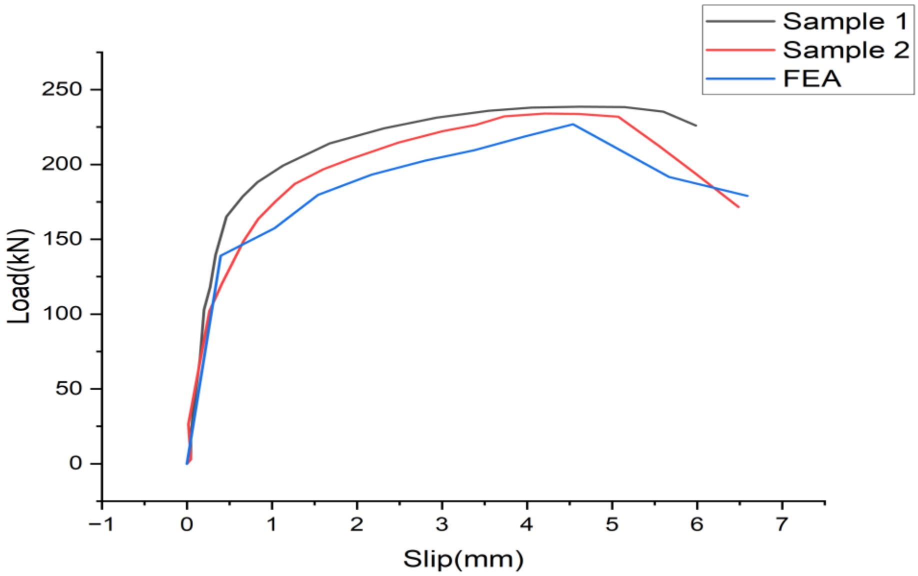

Figure 4- 2 Load slip Behaviour of experimented and FEA analysed headed stud

From the table 4 and the figure 4-2, it can be summarised that the result obtained from experiment behaves similarly as the result obtained from finite element analysis of headed stud as value of R^2 is same in sample 1 and FEA result. The FEA result shows the decrease in bearing capacity after the slip of 4.5mm whereas in experiment result shows the decrease in value of bearing capacity after the slip of 5mm. The R^2 value is determined based on second order of non-linear analysis as this analysis takes into account of the materials' non-linear deformation properties and is conducted on the geometry of the deformed structure.

4.1.2. Mode of Failure

From the figure 4-3, it can be visualised that the failure in the headed stud has occurred due to fracture of shanks of headed stud. The fracture of shank has started from the root (intersection of headed stud and steel section) of headed stud.

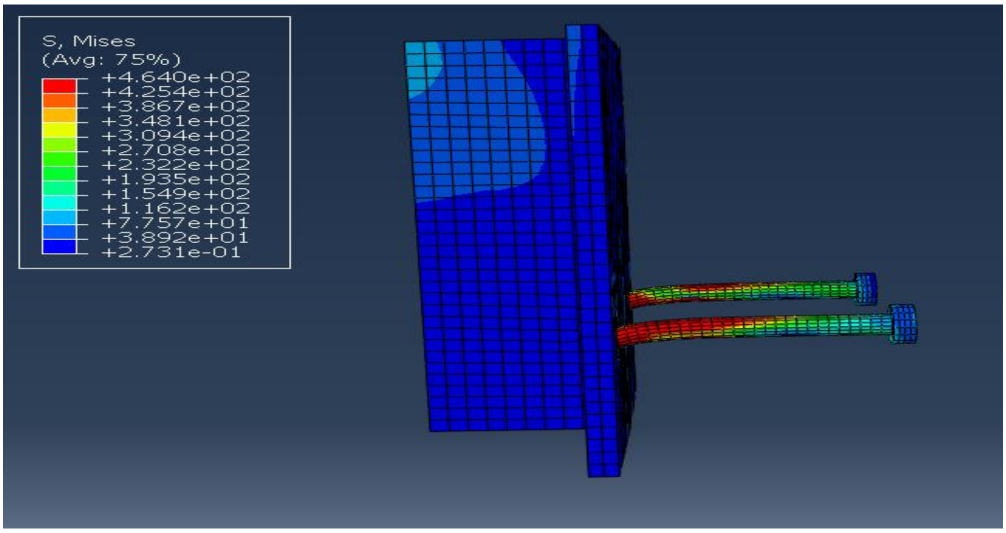

Figure 4- 3 Failure of headed stud

Similarly, it is evident that the concrete fails by crushing below the root of the headed stud which can be visualised through the figure 4-4. The cracking starts from the immediate bottom of headed stud and spread in the direction of applied force.

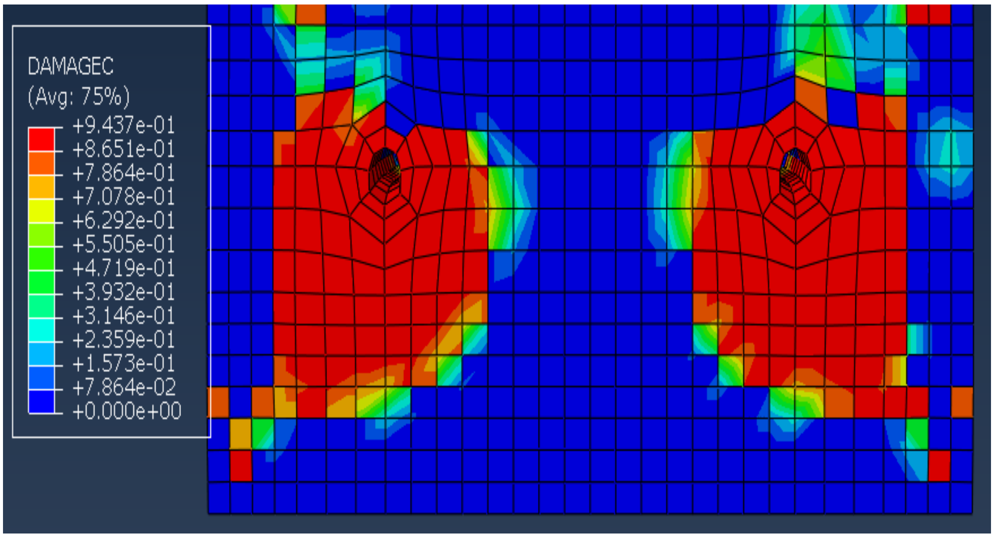

Figure 4- 4 Concrete Damage Pattern after FEA analysis

4.1.3. Response of local Stress

The figure 4-5 describes the local response stress at different length of the headed stud starting from the root of headed stud. The top section represent the highest point on the headed stud in the direction of applied force whereas the bottom section represents the lowest point of the headed stud in the direction of applied force.

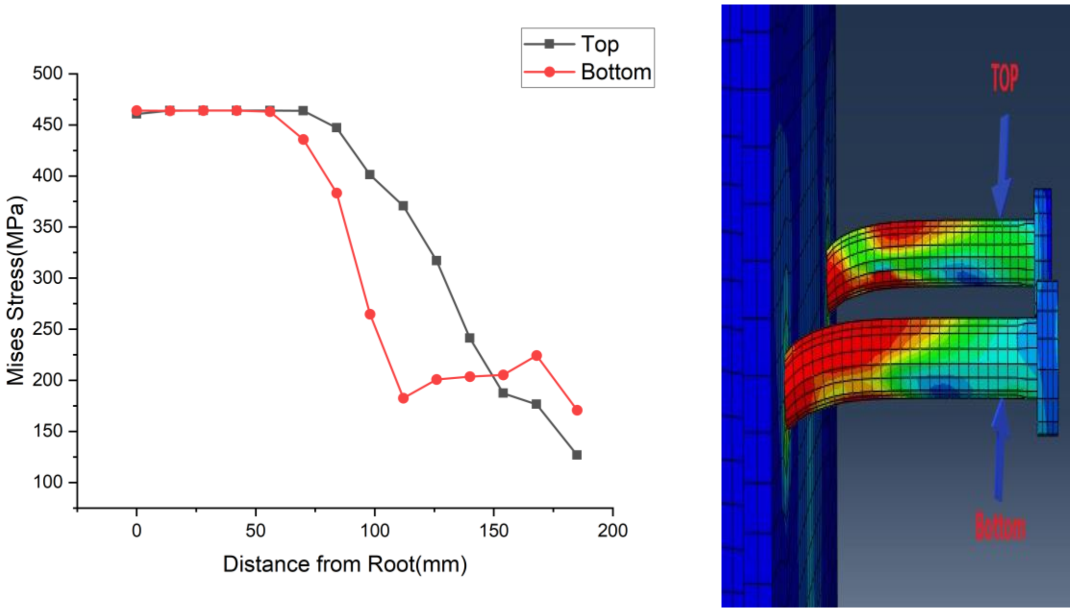

Figure 4- 5 Stress at the different points of headed stud

The mises stress is maximum that is 464 MPa at the root of headed stud and till 60 mm length of shank for both top and bottom point. After 60 mm bottom section, mises stress decreases in uniform pattern for the length upto 120 mm and then has slow rises in stress for length upto 160 mm and decreases abruptly after it.

Whereas, in case of top section, the maximum stress occurs upto 80mm length and has drop in stress till 160 mm. The rate of drop is less than that of bottom section. After 160mm the drop of stress in abrupt as in bottom section.

To summarise the response of stress, it could be said that the stress is maximum at the root of headed stud and minimum at other end, as concrete compression zone is formed at root. There is more stress at the top section compared to the bottom section of headed stud. The fluctuation of stress at bottom section of stud between 110mm-160mm is due to formation of compression zone in the area as stud is taking support in that region when downward force is acting on it. The headed region of stud also provides anchorage action in the composite section.

4.2. U-shaped Connector

4.2.1. Load-slip Responses

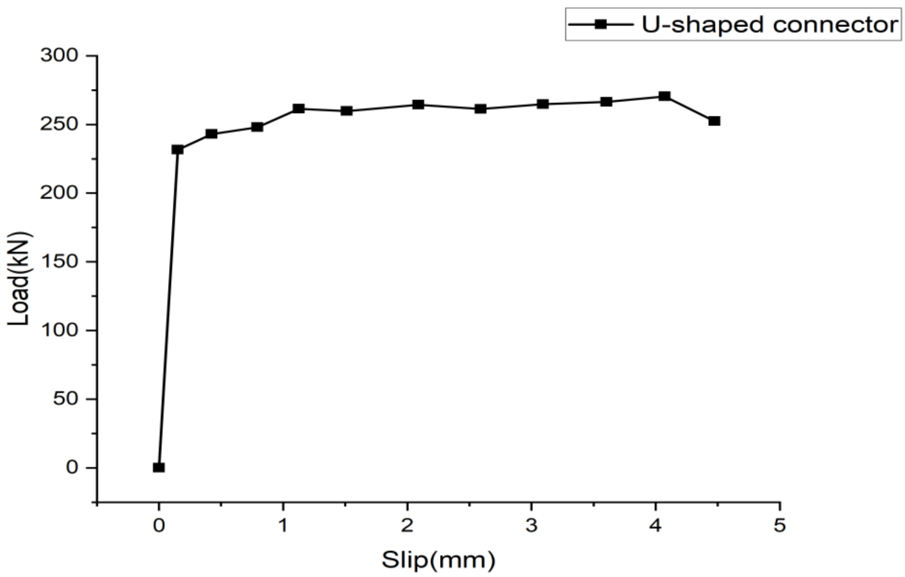

Figure 4- 6 Load slip responses of U-shape stud

In case of U-shaped connector, load-slip behaviour is same as that of headed stud. From the figure 4-6, it is noted that load has increased drastically from 0 to 231 kN with slip of 0.15 m and then it has follow a pattern of slow increase of load upto the length 4mm slip, reaching the maximum load of 270.36 kN. After reaching the slip of 4 mm, there is gradual decreasing in load for the increase in slip denoting the failure point of U-shaped shear connector.

4.2.2. Mode of Failure

In case of U- shape connector, the failure occurs at the root (intersection between U shear connector and steel beam) of the U-shape connector. From the figure it can be analysed that the failure occurs at the straight legs of U-shape connector rather than semi-circle part where the stress are minimum.

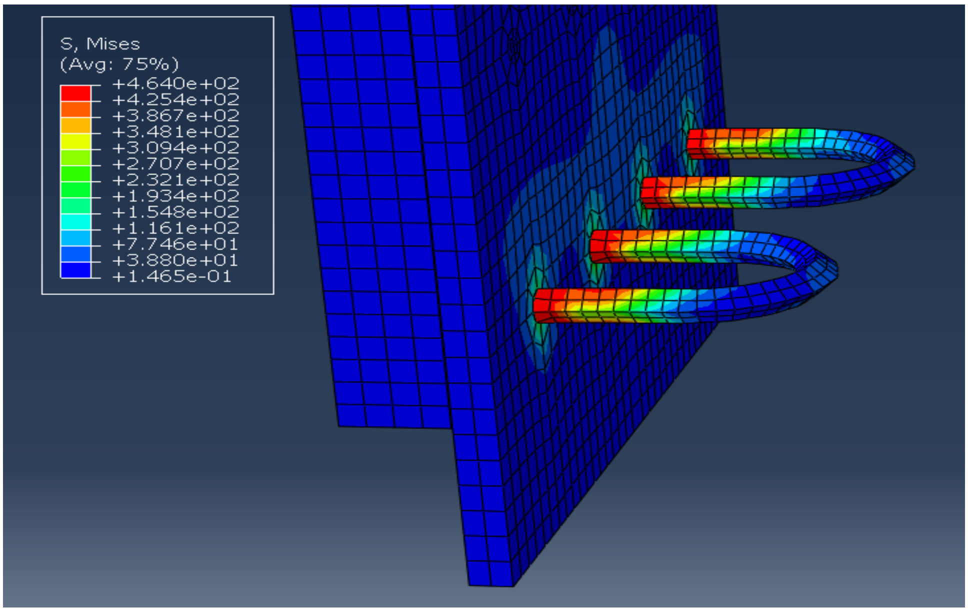

Figure 4- 7 Failure of U-shape connector

Similarly, it is evident that the concrete fails by crushing below the root of the U-shape shear connector which can be visualised through the figure 4-8. The cracking starts from the immediate bottom of U-shape shear connector and spread in the direction of applied force.

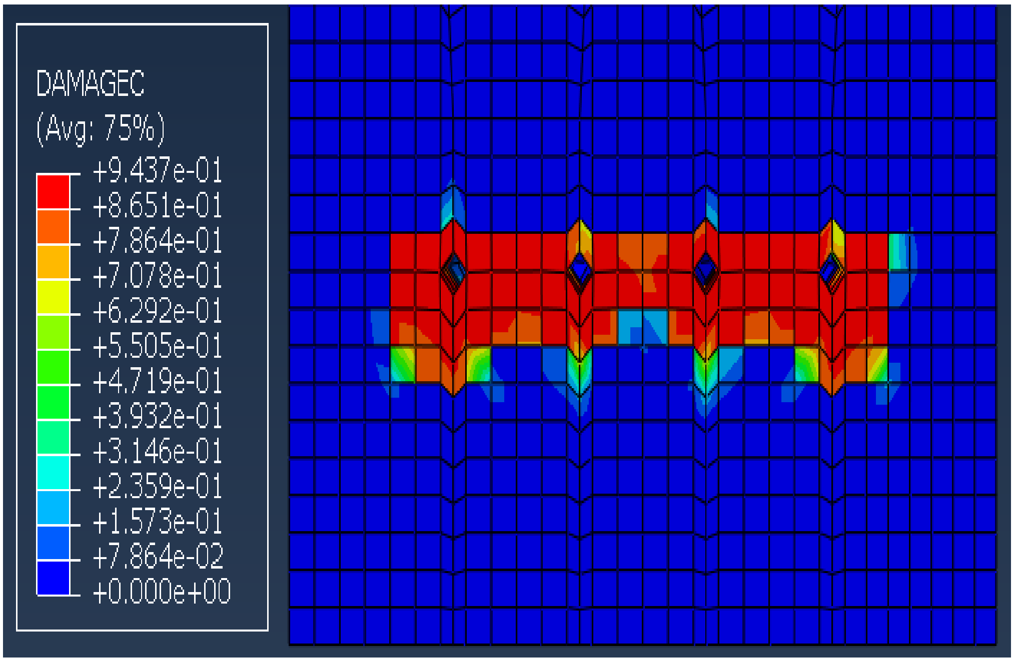

Figure 4- 8 Damage of concrete with U-shape connectors

4.2.3. Response of local Stress

The figure 4-9 describes the local response stress at different length of the U-shape shear connector starting from the root of it. The top section represent the highest point on the U-shape connector in the direction of applied force whereas the bottom section represent the lowest point on the U-shape connector in the direction of applied force.

The mises stress is maximum (i.e. 464 MPa) at the root of U-shape connector and till 60 mm length of shank for both top and bottom point. After 60 mm, mises stress decreases in uniform pattern for the length upto 110 mm for bottom section reaching the stress value of 112 kN and then has slow rises in stress for length upto 140 mm. The 140mm length marks the midpoint of U shape shear connector.

Whereas, in case of top section, the maximum stress occurs till 80mm length and has drop in stress till 126 mm. The rate of drop is greater that of bottom section. After 126 mm there is rise in stress till 210mm length and there becomes maximum i.e. 464 MPa till the end of U-shape connector.

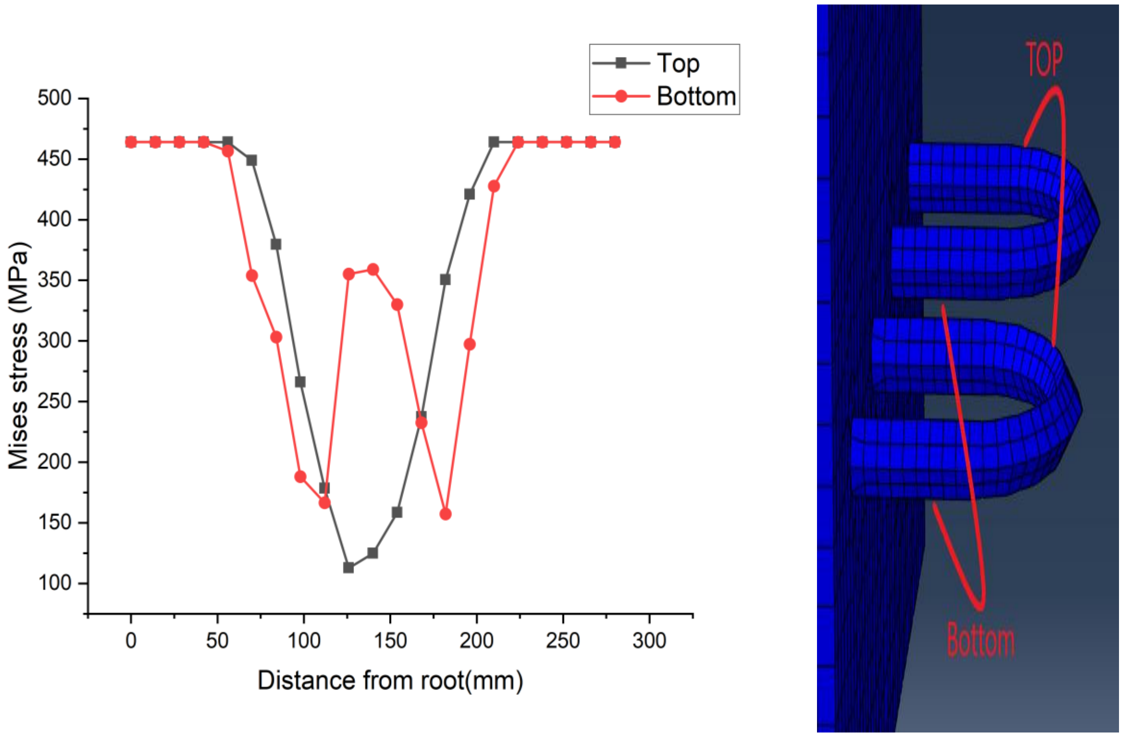

Figure 4- 9 Stress at the different points of U-shaped connectors

To summarise the response of stress, it could be said that the stress is maximum at the both roots of U-shaped connector and minimum at the middle of U-shape connectors. As U-shape connector is symmetrical, the stress distribution is also symmetrical along its midpoint. The stress is maximum at root because of concrete compression occurring at the intersection of concrete and stud. The stress is maximum upto the leg-length of U-shape connector and then starts to decrease in both top and bottom sections. Less force acting around periphery of concrete and stud at Top section compared to bottom section periphery due to force being acted on stud at downward direction. There is formation of compression zone at the middle of U-shape connector as it is taking support from the extreme end of its length leading to the rises of stress in the middle of shear connector. Stress decreases from the end of leg of U-shape connector becomes minimum and then increases when it is in the midpoint of stud because as there is formation of compression zone at the leg of U-shape connector and middle connector at connector is supporting itself while loading in this zone. Hence when we move away from the compression zone, stress decreases and becomes minimum and stress slowly rises while moving towards the compression zone again.

4.3. Comparison of U-shaped connectors and Headed stud connectors

4.3.1. Comparison of Load-slip Responses

The figure 4-10 compares load slip behaviour of U-shape connector and headed stud connector. The analysed U-shape connector shows greater stiffness than that of headed stud connector at initial loading condition. The peak load that a U-shape connector can resists is 270.36 kN whereas the peak load the headed stud connector can resists is 226.38 kN. The slip value of concrete for headed stud before yielding is 4.5mm whereas the slip value of concrete is 4mm for U-shape connector before yielding. The peak load that a U-shape connector can carry is 19.42% greater loadcompared to Headed-stud connector.

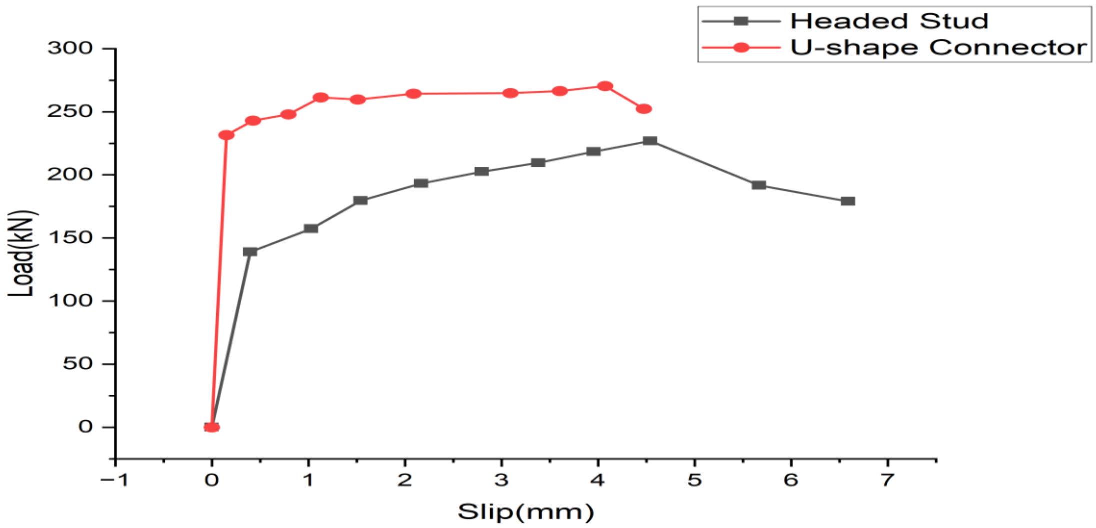

Figure 4- 10 Load-slip Behaviour of U-shape connector and Headed stud

4.3.2. Comparison of local Stress

The graph in the figure 4-11 represent the comparison of stress at top section of headed stud and U-shaped connectors. The stress are maximum at one root of headed stud and both roots of U-shape connector. With the increase in distance from the root of connectors the stress decreases gradually in both connector. In case of headed stud, the stress reaches minimum at the end of stud but in case of U shape connector the stress becomes minimum at the middle and rises to maximum while moving towards the other end of root from middle.

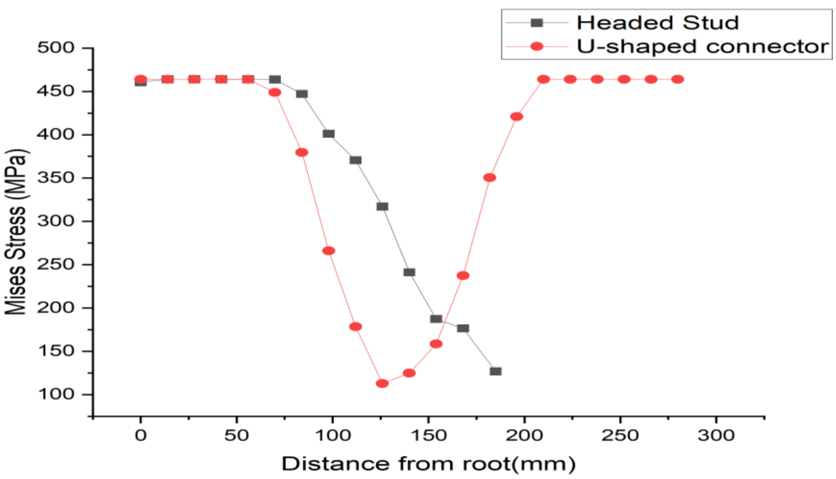

Figure 4- 11 Comparison of stress at top section of Headed Stud and U-shaped connectors

The graph in the figure 4-12 represent the comparison of stress at bottom section of headed stud and U-shaped connectors. The stress are maximum at one root of headed stud and both roots of U-shape connector. With the increase in distance from the root of connectors the stress decreases gradually in both connector. In case of headed stud, the stress reaches minimum at the end of stud but in case of U shape connector the stress fluctuates i.e. maximum stress at the ends of U shape connector decreases to the minimum value and then again increases towards the middle. The stress is symmetrical from the middle toward the end in U shape connector.

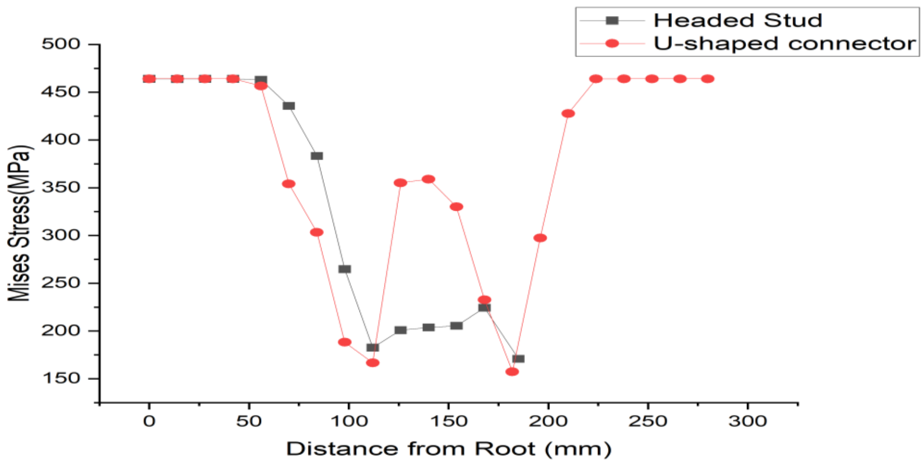

Figure 4- 12 Comparison of stress at Bottom section of Headed Stud and U-shaped connectors

4.3.2. Comparison of mode of failure

From the figure 4-3, 4-4, 4-7, 4-8 an analogy can be made that mode of failure of headed stud and U-shape connector are similar. The studs in both case fails at the root of shank. Since the number of roots in U-shape connector is greater than that of headed stud connector the force needed to fail the stud in U- shape connector is greater than that of headed stud.

Similarly the concrete also fails in both cases by crushing. Concrete crushing occurs beneath the root of shear connectors.

4.4. Optimisation of U-shape connector

Since U-shape shear connector has greater load bearing capacity. So, we have considered to optimise it with respect to the headed stud shear connector. In order to have the same peak value of load as of headed stud, optimisation on the basis of size is carried out (i.e. change in the diameter of U-shape connector). In this analysis the diameter of U-shape shear connector is reduced from 20mm to 16mm whereas other parameters such as length, material property etc. are kept same.

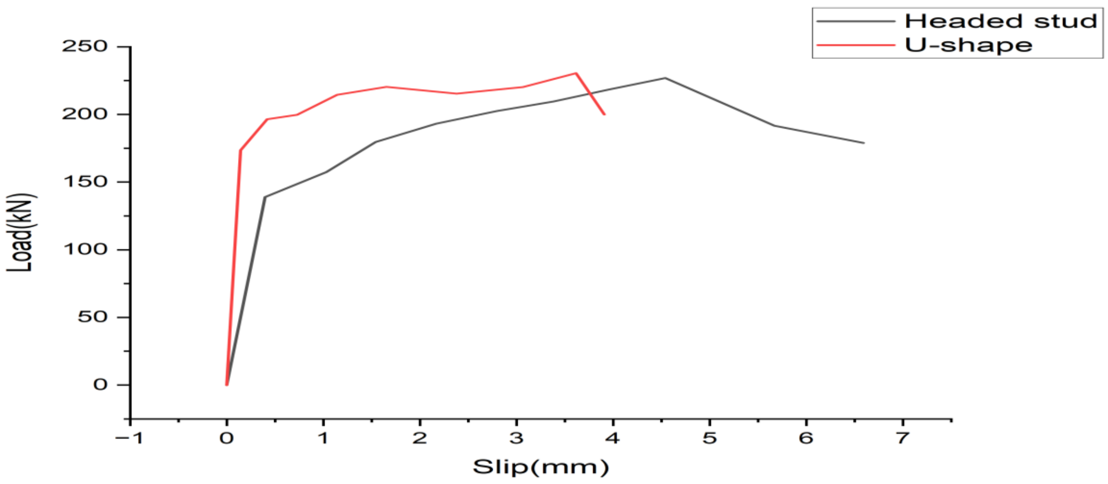

Figure 4- 13 Optimisation of U-shape connector with respect to headed stud connector

From the figure 4-13,value of peak load of U shape connector is 230.475 kN and the peak value of load in headed stud is 226.858 kN.

Sample	No of connectors	Volume of connector	Reduction in Weight	Percentage reduced
Headed stud	2	91103.5 * 2 =182207	-	-
U-shape	2	58120.26 * 2 =116240.5	65966.78	36.2%

Table 5 Comparison between optimised U-shape connector and headed stud connector

Peak load value of 22 mm headed stud connector is equivalent to the peak load value of 16 mm U-shaped connector. The volume of U-shaped connector is 36 percent less than that of headed stud connector for the same peak load value which can be analysed from Table 5.

Chapter 5
Conclusion

In this project, simulation of pushout test of headed stud and U-shaped connector has been carried out in ABAQUS. Following conclusion can be extracted from this project.

- For the same volume of material, U-shape shear connectors provide 20% more load bearing capacity compared to headed stud connectors.
- The load-slip characteristics of headed stud connector and U-shape connector are found to be similar.
- Although the load bearing capacity of both connector are same but the stress distributed within the connector are different. U-shape connector has maximum stress at roots and minimum at the middle whereas the headed stud connector has maximum stress at its root and reaching minimum while moving further form the roots.
- Mode of failure of U-shape connector and headed stud connector are similiar. The stud fails by fracture from its root in both cases and concrete fail by crushing below the root of shear connector.
- The cost of the projects can be reduced by replacing the headed stud with U-shaped connector as the load bearing capacity of U-shape is greater than headed stud within the same mass of stud. 36% of reduced cost is obtained by replacing the headed stud connector with U-shape connector.
- The FEA analysis through ABAQUS can be cost effective and time saving through which we can simulate model and made the model behaves like object in real life and their behaviour can be studied.

Annexure

The below table shows the value obtained from the graph given in figure 3-4 to figure 3-7 which was calaculated through concretedamagedplasticity.com

Concrete compressive behaviour		Concrete compressive Damage		ConcreteTensile behaviour		Concrete Tension Damage	
yield stress (Mpa)	**Inelastic strain**	**Damage parameter**	**Inelastic strain**	**yield stress (Mpa)**	**Inelastic strain**	**Damage parameter**	**Inelastic strain**
11.7391	0	0	0	5.0955	0	0	0
23.5695	1.2E-05	0	1.2E-05	4.28022	2.6E-05	0.16192	2.6E-05
26.4388	1.7E-05	0	1.7E-05	3.46494	5.2E-05	0.311714	5.2E-05
29.2691	2.3E-05	0	2.3E-05	2.64966	7.8E-05	0.450696	7.8E-05
32.0586	3E-05	0	3E-05	1.83438	0.0001	0.579995	0.0001
34.8052	3.8E-05	0	3.8E-05	1.0191	0.00013	0.70059	0.00013
37.5067	4.7E-05	0	4.7E-05	0.97069	0.00014	0.716455	0.00014
40.1605	5.8E-05	0	5.8E-05	0.92229	0.00015	0.73132	0.00015
42.7642	6.9E-05	0	6.9E-05	0.87388	0.00015	0.745275	0.00015
45.315	8.2E-05	0	8.2E-05	0.82547	0.00016	0.758403	0.00016
47.8099	9.6E-05	0	9.6E-05	0.77706	0.00017	0.770774	0.00017
50.2457	0.00011	0.0077908	0.00011	0.72866	0.00018	0.782453	0.00018
52.619	0.00013	0.0181819	0.00013	0.68025	0.00018	0.793494	0.00018
54.9261	0.00015	0.0286062	0.00015	0.63184	0.00019	0.803951	0.00019
57.1629	0.00017	0.0391152	0.00017	0.58343	0.0002	0.813867	0.0002
59.3251	0.00019	0.0497569	0.00019	0.53503	0.00021	0.823283	0.00021
61.4079	0.00021	0.0605761	0.00021	0.48662	0.00021	0.832237	0.0-0021
63.4063	0.00024	0.0716159	0.00024	0.43821	0.00022	0.840762	0.00022
65.3146	0.00027	0.0829183	0.00027	0.38981	0.00023	0.848887	0.00023
67.1268	0.0003	0.0945255	0.0003	0.3414	0.00024	0.856641	0.00024
68.8362	0.00033	0.1064797	0.00033	0.29299	0.00024	0.864048	0.00024

70.4356	0.00037	0.1188246	0.00037	0.24458	0.00025	0.87113	0.00025
71.917	0.00041	0.1316053	0.00041	0.19618	0.00026	0.87791	0.00026
73.2717	0.00045	0.1448696	0.00045	0.14777	0.00026	0.884405	0.00026
74.4899	0.0005	0.1586679	0.0005	0.09936	0.00027	0.890633	0.00027
75.5611	0.00055	0.1730543	0.00055	0.05095	0.00028	0.896611	0.00028
76.4735	0.0006	0.1880874	0.0006	0.05095	0.00029	0.89865	0.00029
77.214	0.00065	0.2038307	0.00065	0.05095	0.00029	0.90061	0.00029
77.768	0.00072	0.220354	0.00072	0.05095	0.0003	0.902496	0.0003
78.1193	0.00078	0.237734	0.00078	0.05095	0.00031	0.904311	0.00031
78.2496	0.00085	0.2560556	0.00085	0.05095	0.00031	0.90606	0.00031
78.1387	0.00093	0.2754135	0.00093	0.05095	0.00032	0.907747	0.00032
77.7633	0.00102	0.2959137	0.00102	0.05095	0.00032	0.909373	0.00032
77.0975	0.00111	0.3176752	0.00111	0.05095	0.00033	0.910944	0.00033
76.1116	0.00121	0.340833	0.00121	0.05095	0.00034	0.912461	0.00034
74.7719	0.00131	0.3655401	0.00131	0.05095	0.00034	0.913927	0.00034
73.0397	0.00143	0.3919714	0.00143	0.05095	0.00035	0.915345	0.00035
70.8703	0.00156	0.4203279	0.00156	0.05095	0.00036	0.916717	0.00036
68.2123	0.0017	0.4508417	0.0017	0.05095	0.00036	0.918045	0.00036
65.0057	0.00185	0.4837823	0.00185	0.05095	0.00037	0.919331	0.00037
61.1801	0.00202	0.5194652	0.00202	0.05095	0.00037	0.920578	0.00037
56.6528	0.00221	0.5582613	0.00221	0.05095	0.00038	0.921787	0.00038
51.3256	0.00241	0.6006108	0.00241	0.05095	0.00039	0.922959	0.00039
45.0809	0.00264	0.6470396	0.00264	0.05095	0.00039	0.924097	0.00039
37.7772	0.00289	0.6981815	0.00289	0.05095	0.0004	0.925202	0.0004
29.2419	0.00317	0.7548075	0.00317	0.05095	0.00041	0.926275	0.00041
19.2632	0.00349	0.8178655	0.00349	0.05095	0.00041	0.927317	0.00041
7.57745	0.00385	0.8885338	0.00385	0.05095	0.00042	0.928331	0.00042
0.7825	0.00409	0.9290053	0.00409	0.05095	0.00042	0.929317	0.00042
0.7825	0.00417	0.9302232	0.00417	0.05095	0.00043	0.930276	0.00043
0.7825	0.00424	0.9314	0.00424	0.05095	0.00044	0.931209	0.00044

0.7825	0.00432	0.9325378	0.00432	0.05095	0.00044	0.932117	0.00044
0.7825	0.00439	0.9336384	0.00439	0.05095	0.00045	0.933002	0.00045
0.7825	0.00447	0.9347037	0.00447	0.05095	0.00046	0.933865	0.00046
0.7825	0.00454	0.9357354	0.00454	0.05095	0.00046	0.934705	0.00046
0.7825	0.00462	0.9367349	0.00462	0.05095	0.00047	0.935524	0.00047
0.7825	0.00469	0.9377039	0.00469	0.05095	0.00047	0.936323	0.00047
0.7825	0.00477	0.9386436	0.00477	0.05095	0.00048	0.937102	0.00048
0.7825	0.00484	0.9395554	0.00484	0.05095	0.00049	0.937863	0.00049
0.7825	0.00492	0.9404404	0.00492	0.05095	0.00049	0.938605	0.00049
0.7825	0.00499	0.9413	0.00499				
0.7825	0.00507	0.942135	0.00507				
0.7825	0.00514	0.9429467	0.00514				
0.7825	0.00522	0.9437359	0.00522				

Table 6 CDP value of concrete used

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