JIMMA UNIVERSITY

SCHOOL OF GRADUTE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

STRUCTURAL ENGINEERING STREAM

Comparative Analysis of Shear Reinforcement Shapes in Reinforced Concrete Beams

---

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

BY:

ETSUB MULUGETA

MARCH 2021

JIMMA, ETHIOPIA

JIMMA UNIVERSITY

SCHOOL OF GRADUTE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

STRUCTURAL ENGINEERING STREAM

Comparative Analysis of Shear Reinforcement Shapes in Reinforced Concrete Beams

---

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

BY:

ETSUB MULUGETA

MAIN ADVISOR:

ENGR. ELMER C. AGON, Asso. Prof.

CO-ADVISOR:

ENGR. IBRAHIM KEDIR (MSc.)

MARCH 2021

JIMMA, ETHIOPIA

JIMMA UNIVERSITY

SCHOOL OF GRADUTE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

STRUCTURAL ENGINEERING STREAM

Comparative Analysis of Shear Reinforcement Shapes in Reinforced Concrete Beams

ETSUB MULUGETA ZERIHUN

for Engr. Abinet Alemseged Chair holder structural Engineering

APPROVED BY BOARD OF EXAMINERS

1. Engr. Elmer C. Agon ____________ 22/03/2021

   Main advisor — Signature — Date

2. Engr. Ibrahim Kedir ____________ 22/03/2021

   Co-advisor — Signature — Date

3. Dr. Binaya Patnaik ____________ 16/03/2021

   External Examiner — Signature — Date

4. Engr. YehamleshetMenberu ____________ 22/03/2021

   Internal Examiner — Signature — Date

5. Engr.Haymanot G/silassie ____________ 22/03/2021

   Chairperson — Signature — Date

DECLARATION

I, the undersigned, declare that this thesis entitled "*Comparative Analysis of Shear Reinforcement Shapes in RC beams*" is my work. This thesis has not been present and submit to any other university, and that all sources of material used for this thesis have been duly acknowledged.

Etsub Mulugeta	____________	22/03/2021
Name	Signature	Date

Place: Jimma University, Jimma Institute of Technology

Advisors

Engr. Elmer C Agon (Asso. Prof.)	____________	22/03/2021
Main Advisor	Signature	Date
Engr. Ibrahim kedir (MSc.)	____________	22/03/2021
Co- Advisor	Signature	Date

ABSTRACT

Reinforced concrete beams subjected to high shear force, exhibit a diagonal tension crack, which tends to widen propagate towards the point of loading. Shear failure in reinforced concrete beam is usually sudden without sufficient warning. This type of failure made it necessary to explore more ways that are effective to design of these beams for shear. The two types of reinforcement account for shear are vertical or inclined stirrups. In vertical stirrups, the spacing between stirrups reduced at the supports to resist high shear stresses. However, the congestion near the support of reinforced concrete beams increases the cost and time required for installation.

This study focuses on reinforced concrete beam with different shear reinforcement shapes using finite element analysis to identify the most efficient shear reinforcement shapes considering three-span length of reinforced concrete beams having 2m differences (2m,4m, and 6m).

Twenty-eight reinforced concrete beams subjected to monotonic loading were compared based on shear strength using different shear reinforcement shapes and angles. The shear reinforcement shapes used in reinforced concrete beams are vertical stirrups considered a control sample, inclined stirrups, swimmer bars, continuous rectangular spirals, and warren truss shapes. Twelve reinforced concrete beams designed considering four-angle of shear reinforcement having 15^0 inclinations differences (45^0,60^0, $75^0$and 90^o) and compared based on shear strength values obtained manually and by using Tekla Tedds 2019 calculation software based on EN 1992-1-1code [5]. The shear carrying capacity of sixteen reinforced concrete beams using different inclined shear reinforcement shapes was analyzed under four-point loading conditions and compared with reinforced concrete beams having vertical stirrups considering the same shear reinforcement spacing using ABAQUS V6.14-5 Software.

Reinforced concrete beams with 45^o inclined shear reinforcements showed higher shear strength compared to the beams having other angles of inclination based on EC2 design. The increased shear capacity of reinforced concrete beams using swimmer bars, inclined stirrups, warren truss, and rectangular spirals as shear reinforcement shapes compared to reinforced concrete beams with vertical stirrups along 2m, 4m, and 6m span length showed averagely 35%, 30%, 24%, and 17% respectively considering the same shear reinforcement spacing. From the analytical results obtained, the uses of inclined links are an effective technique to enhance the shear capacity of reinforced concrete beams specially using swimmer bar shapes.

Keywords: *Inclined stirrups, Swimmer bars, continuous rectangular spirals, warren truss*

ACKNOWLEDGMENT

First, I thank God for giving me the courage, strength, and patience to complete this thesis. Then, I would like to thanks my advisor Engr. Elmer C. Agon (Asso. Prof) and co-advisor Engr. Ibrahim Kedir for their guidance, suggestions, and valuable comments throughout this thesis work.

Finally, I must express my very profound gratitude to my parents and my friends for providing me continuous encouragement and assistance in writing this thesis, despite their busy schedules with unfailing support. Thank you.

TABLE OF CONTENT

Contents Page No.

LIST OF TABLES

LIST OF FIGURES

ACRONYMS AND SYMBOLS

A_{SW}	Cross-sectional area of the shear reinforcement
$A\upsilon$	Area of one stirrup
av	Distance between the applied force and the support of beams
A_{sl}	Area of tensile reinforcement
b	width of the section
bw	Minimum width between tension and compression chords
CAE	Complete Abaqus Environment
CDP	Concrete damaged plasticity
CDPM	Concrete damage plasticity model
d	Effective depth of cross-section
EC2	Euro Code 2
FEA	Finite Element Analysis
FEM	Finite Element Method
GUI	Graphical user interface
Fcd	design value of the concrete compression force in the longitudinal member axis
fck	Compressive strength of concrete
Ftd	design value of the tensile force in the longitudinal reinforcement
f_{ywd}	Design yield strength of the shear reinforcement
PL	Point Load
RC	Reinforced Concrete
RCB-1	Reinforced Concrete beam with 45^{o} Inclined shear reinforcement
RCB-2	Reinforced Concrete beam with 60^{o} Inclined shear reinforcement
RCB-3	Reinforced Concrete beam with 75^{o} Inclined shear reinforcement
S	Spacing of shear reinforcement
ULS	Ultimate Limit State
V	Applied shear force
υ	A strength reduction factor for concrete cracked in shear

$V_{Rd,c}$ Design shear resistance of the member without shear reinforcement

$V_{Rd,s}$ Design value of the shear force which sustained by the shear reinforcement

WIB Welded, inclined bars

XC1 Carbonation - Dry or permanently wet (interior of buildings with very low air humidity, permanently submerged in water)

z Lever arm of internal forces

α Angle between shear reinforcement and the beam axis perpendicular to the shear force

a_{cw} A coefficient taking account of the state of the stress in the compression chord

θ Angle between concrete compression strut and beam axis perpendicular to the shear force

σ_{cp} Concrete compressive stress at the centroid axis due to axial loading

CHAPTER ONE

INTRODUCTION

1.1 Background of the Study

Beams are structural members used to carry loads primarily by internal shears and moments. Reinforced concrete beams used to transfer loads from slabs to the column. It must have an adequate safety margin against bending and shear forces so that it will perform effectively during its service life.

The design of a reinforced concrete member, flexure usually considered first that leading to the size of the section and the arrangement of reinforcement to provide necessary resistance for moments. The designers considered in the design of reinforced concrete beam members are safety, durability, and costs. The structural design codes usually emphasize safety as a priority taken when designing steel and concrete structures. For safety reasons, limits are set on the amounts of flexural reinforcement to make ductile failure.

Many reinforced concrete structures encountered shear problems due to various reasons such as mistake in design calculations and improper detailing of shear reinforcement; construction faults or poor construction practices; changing the function of a structure from a lower service load to a higher service load, and reduction in or total loss of shear reinforcement steel area causing corrosion in service environments.

Shear failure in the reinforced concrete beam is usually sudden without sufficient advanced warning. As a result, shear failure is more dangerous than the flexural failure. Reinforced concrete beams subjected to high shear force, exhibit a diagonal tension crack tends to widen propagate towards the point of loading. The shear failure mechanism varies depending upon the cross-section, loading type, and properties of reinforced concrete member. Monotonic loadings in the beams of a high level of shear reduce ductility and cause brittle failure.

Effectively anchored reinforcement in reinforced concrete beam able to resist the shear forces to a certain extent. The beam ductility increase and sudden failure reduce using Shear reinforcements. Shear reinforcement provided in reinforced concrete beams whenever the actual shear stress values exceed the permissible shear stress values.

Practically, the shear reinforcements are vertical stirrups, inclined stirrups, bent-up bars, a combination of stirrups, and bent-up bars. The other shear reinforcements that have not used widely in the construction industry are inclined stirrups, swimmer bars, rectangular spiral, warren truss shapes as shear reinforcements, and so on. Vertical stirrups are the conventional shear reinforcements used widely in the construction industry to increase the shear capacity of RC structural members. Form the inclined stirrups; a Swimmer bar system is a new type of shear reinforcement, which is an inclined stirrup with both ends bent horizontally for a short distance. The implementation of rectangular spiral shear reinforcement in reinforced concrete beams is a new technology that estimated and enhance the capacity and performance of these reinforced concrete members. One of the main benefits when using rectangular spiral stirrups is materials savings because there are not required end hooks for each section to close the stirrup and ensure proper structural behavior against the stirrup opening. The warren truss shape as shear reinforcements taken from the truss member shapes, using these concepts of truss members shape throughout the beam length, which lead to positive results as shear reinforcement in reinforced concrete beams.

The shear reinforcements form an angle α of between 45^0 and 90^0 to the longitudinal axis of the structural element according to EC2. The stirrup's contribution highly increases beam strength according to EC2, with a precise selection of concrete strut inclination angleθ, much more than other methods. Moreover, a high range selection in θ (from 21.8° to 45°), can change shear capacity significantly.

1.2 Statement of the Problem

Many factors affect the shear failure of reinforced concrete beams. These are the proportion and shapes of beam, structural restraints, and interactions of beam with other components in the system, compressive and transverse reinforcements, load distribution and load history, placement of concrete and curing, and the surrounding environment.

In the case of static loads, the principal tensile stresses are much more inclined where shear forces are significant, so the inclined shear reinforcements along the direction of these stress is much more effective. Reinforced concrete beam that has a vertical stirrup, the spacing between stirrups reduce at the supports to resist high shear stresses. Although, a high apply shear force requires shear reinforcing bars of large diameter place at closer spacing and the congestion of these shear reinforcing bars near the support of reinforced concrete beams increases the cost and the time for installation.

Shear failure in reinforced concrete beam is usually sudden without sufficient warning. This type of failure made it necessary to explore more shear reinforcements shapes along the inclined direction to reduce sudden failure and maximize the shear capacity of beams.

The uses of inclined stirrups, swimmer bars, rectangular spiral reinforcements, and warren truss shapes, as shear reinforcement in reinforced concrete beams, have not used widely in the construction industry as compared to reinforced concrete beams having vertical stirrups due to not well addressed in the construction industry and the time required for implementation in large construction of reinforced concrete beams. Using these shapes of shear reinforcement in reinforced concrete beams analyzed and compared with vertical stirrup of reinforced concrete beam based on EC2 subject to monotonic loading.

1.3 Research Question

Regarding the use of other alternatives of shear reinforcement shapes with a different angle of inclination instead of vertical or conventional stirrups in reinforced concrete beams, the research answers the following questions: -

1. What is percentage improvement in the shear strength of reinforced concrete beams using inclined stirrups, swimmer bars, rectangular spiral reinforcements, and warren truss as shear reinforcement shapes comparing to reinforced concrete beams with vertical stirrups?
2. Which one of reinforced concrete beams with different shear reinforcement shapes has high shear performance compare to a reinforced concrete beam with conventional stirrup considering load- deflection of the reinforced concrete beams?

1.4 Objective of the study

1.4.1 General Objective

The general objective of this study was to compare the performance of reinforced concrete beams with different shear reinforcement shapes using finite element method.

1.4.2 Specific Objectives

The specific objectives of this study are:

- To compare the effectiveness of reinforced concrete beams that have inclined shear reinforcement shapes to reinforced concrete beams that have vertical stirrups.
- To validate the nonlinear FE analysis software results with experimental results.

- To determine maximum deflection of reinforced concrete beams using different shear reinforcement shapes.
- To identify the most efficient shear reinforcement shapes from inclined links, swimmer bars, rectangular spiral reinforcement, and warren truss shapes comparing to vertical stirrups by considering the same spacing and diameter of shear reinforcements.

1.5 Significance of the Study

The vertical stirrups are widely used as shear reinforcement in reinforced concrete beams in most of the construction industry. However, the inclined shear reinforcements using different shapes or configurations in reinforced concrete beams have a large effect in improving the shear performance of the reinforced concrete beams compared to reinforced concrete beams having vertical stirrups. Therefore, this study is significant to the designers and researchers to know how reinforced concrete beams with different shear reinforcement shapes and angles behave, and which reinforced concrete beams have the most efficient shear reinforcement configuration to resist shear force.

The results of data analysis and information obtain from this study; helps the stakeholders to use other alternatives of shear reinforcements shapes in the construction industry by comparing the shear performance of reinforced concrete beams with the beam, which has a vertical stirrup. The students also obtain knowledge about the effect of different shear reinforcement shapes using different angles of inclination in reinforced concrete beams by relating design and analysis result of reinforced concrete beams subject to four-point loading based on EC 2.

1.6 Scope and limitations of the Study

The scope of this study is to find out the suitability of adopting shear reinforcement shapes considering shear performance of the reinforced concrete beams, which is better than the usual way of providing a vertical stirrup reinforcement system in reinforced concrete beams based on EN 1992-1-1 code.

Twenty-eight reinforced concrete beams subjected to monotonic loading were compared based on shear strength using different shear reinforcement shapes and angles. The reinforced concrete beams grouped in to three based on the effective lengths such as 2m, 4m and 6m. From each groups, the one reinforced concrete beam has a vertical stirrup consider as a control sample.

From the total number of beams, twelve reinforced concrete beams designed considering four-angle of shear reinforcement having 15^0 inclinations differences (45^0,60^0, $75^0$and 90^o) and compared based on shear strength values obtained manually and by using Tekla Tedds 2019 calculation software based on EN 1992-1-1code [5]. The shear carrying capacity of sixteen reinforced concrete beams using different inclined shear reinforcement shapes was analyzed under four-point loading conditions using ABAQUS V6.14-5 Software and compared with reinforced concrete beams having vertical stirrups considering the same shear reinforcement spacing obtained from the reinforced concrete beams with vertical stirrups designed along different span length.

For validation, a reinforced concrete beam was model based on experimentally tested and reported by Roslli Noor Mohamed at University Teknologi Malaysia [19]. Reinforced concrete beams with different shear reinforcement shapes used in this study shown in the figures below:

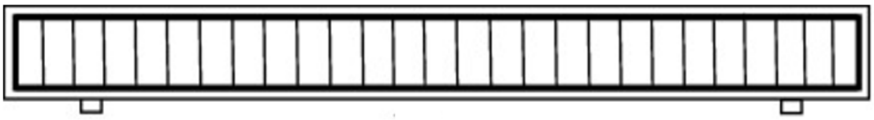


Figure 1. 1: Vertical Stirrups or Conventional Stirrups

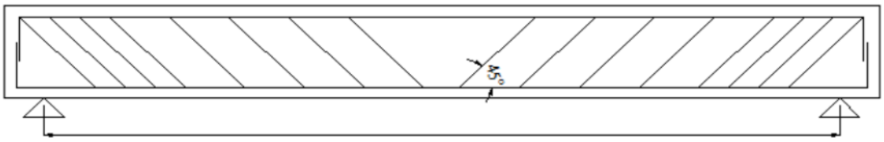


Figure 1. 2 Inclined Stirrups

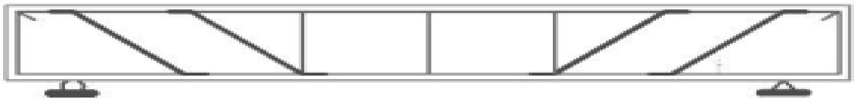


Figure 1. 3: Swimmer bars

Figure 1. 4: Rectangular spiral shear reinforcements

Figure 1. 5: Shape of warren truss girder

CHAPTER TWO

RELATED LITERATURE REVIEW

2.1 Theoretical Review

2.1.1 Shear Transfer Mechanism

In beams without the shear reinforcement, the applied shear (V) is transferred through a combination of shear in the compression zone (Vcz), dowel action (Vd), and the vertical component of aggregate interlock stresses (va) over the surface of the inclined crack as shown in Figure 2.1. These three factors are the sum of beam action. In addition to beam action, arch action also contributes to the shear resistance. The compression zone depth, the shear span to depth ratio, the crack width roughness, concrete strength, and other parameters, affects each of these three components. In addition to these three shear resisting mechanisms, using residual tension across the crack, some shear transferred; however, this component is relatively small, especially in the case of wide cracks. The factors assumed to be carrying shear force in cracked concrete to the supports when no shear reinforcement provides for the member, illustrated in the following free body diagram.

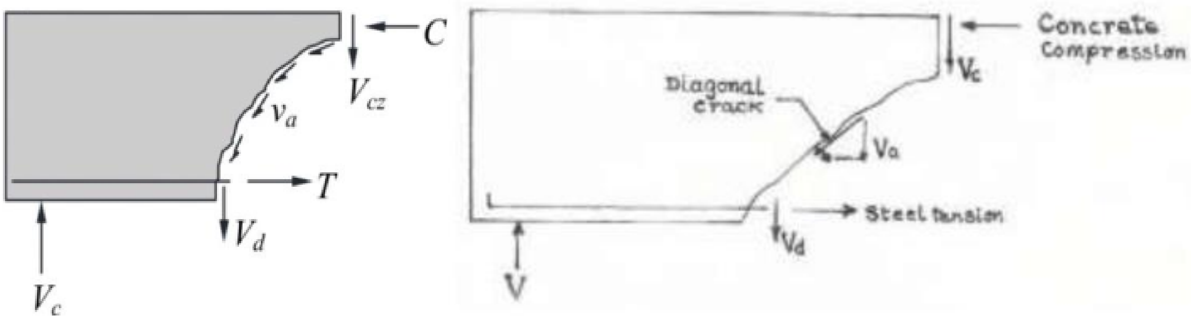


Figure 2. 1: Shear transfer mechanisms in RC beams without shear reinforcement [1]

The total shear force can then have expressed as:

$$V_c = V_c + V_{ay} + V_d \tag{2.1}$$

The Shear reinforcements provide to resist the sudden failure of concrete in shear failure mode. In the case of reinforced concrete beams with shear reinforcement, there is a vertical force (Vs) due to the presence of stirrups; this is contributing to shear resistance. The free-body diagram shown in Figure 2.2 illustrates the fundamental mechanisms of shear transfer in reinforced concrete beams.

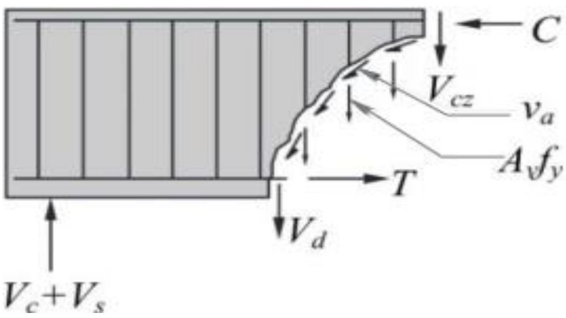


Figure 2. 2: Shear transfer mechanisms in RC beams with shear reinforcement [1]

Gradually, inclined cracks widen in the concrete, the shear resistance from Va decreases while Vc and Vd increase. Finally, when the aggregate interlock reaches failure, a large shear force transfers rapidly to the compression zone cause sudden and often explosive failure to the beam when action contribution is low.

Shear resistance caused by dowel action increases as the shear reinforcement decreases. Consequently, it has a significant effect on members where no shear reinforcements provided. When inclined cracks cross the longitudinal reinforcing bars, forces act on the dowel due to deflection of the bar at the face of beams crack shown in the Figure below.

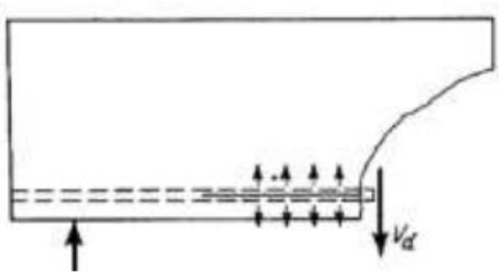


Figure 2. 3: Dowel action [2]

Generally believed that aggregate interlock (Va) transfers a large part of the total shear force to the supports. Width of the cracks, aggregate size, and concrete strength are the most useful variables. When the longitudinal reinforcement ratio increased with added bars to the beam, the width of the flexural cracks gets smaller due to increased shear resistance; consequently, the contribution of Vd decreases. When reinforced concrete beams develop a flexure-shear interaction, the shear resistance consists of two different mechanisms, beams and arch mechanisms. When the arch action begins to contribute more than beam action, the member can achieve considerably more load than at diagonal cracking.

2.1.2 Mode of Failures

Shear failures in the concrete beams occur due to the combined shear forces and bending moments. The diagonal crack characteristics near the support of the beams are small deflections and ductility loss.

Naiem M. Asha et al. [3], he finds out the failures mode of the beam. According to him, the diagonal cracks due to excessive applied shear forces are the mode of shear failure near the supports of reinforced concrete beams. The concrete beam fail immediately upon the formation of critical cracks in the high-shear region near the beam supports. The inclined shear cracks start at the middle height of reinforced concrete beam near support at approximately 45 degrees, which extends towards the compression zone.

Diagonal Failure

Many types of structural concrete members other than beams failed due to shear distress or diagonal failure examples slabs, foundation, columns, corbels, and shear walls. The shear transfer mechanism is very similar or the same in all the cases, but the cracking pattern may differ. Shear reinforcements provided to resist the sudden failure of concrete in shear failure mode [2].

When the applied load is far away from the support, such as the slender flexural element, a truss action Figure 2.4 activated, and it has a longitudinal compression chord on top through the concrete and a tension chord at the bottom formed by the longitudinal tension reinforcement. The shear forces transferred up and down the beam depth by inclined compressive forces, which can carry by the concrete and vertical ties that formed by the shear reinforcement. In a flexure dominated reinforced concrete (RC) member such as reinforced concrete beam, shear stresses cause diagonal cracks to develop, and the concrete divides into a series of diagonal concrete struts. The cracked member acts like a truss having parallel longitudinal chords, and a web composed of diagonal concrete struts and transverse steel ties.

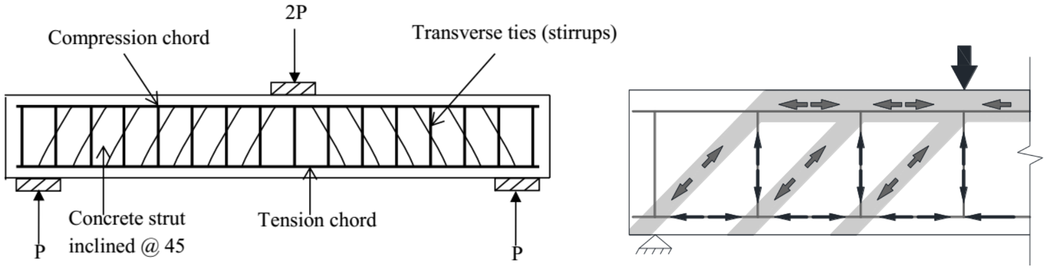


Figure 2. 4: Truss mechanism

Generally, there are different types of diagonal failures in the beams. These are the diagonal tension, shear tension, compression, and flexural failures.

In diagonal tension failure, the diagonal crack initiates from the last flexural crack formed. The diagonal tension failure occurs in beams when the ratio a/d is approximately 2.5 – 6.0 in the shear span. The crack propagates through the concrete beam until it reaches the compression zone. When the beam reaches a critical point, it will fail because of the splitting of the compression concrete. Often this happens almost without an advanced warning, and the failure becomes sudden and brittle.

Figure 2. 5 : Diagonal tension failure [2]

Shear tension failure is similar to diagonal tension failure but applies to short beams. The shear crack propagates through the concrete beam but does not cause the beam failure on its own. Secondary cracks travel along the longitudinal reinforcement from the last flexural crack can cause a loss of bond between the bars and the concrete or anchorage failure (Figure 2.6). When the beam reaches a critical point, it will fail as a result of the splitting of the compression concrete Ziara, [2].

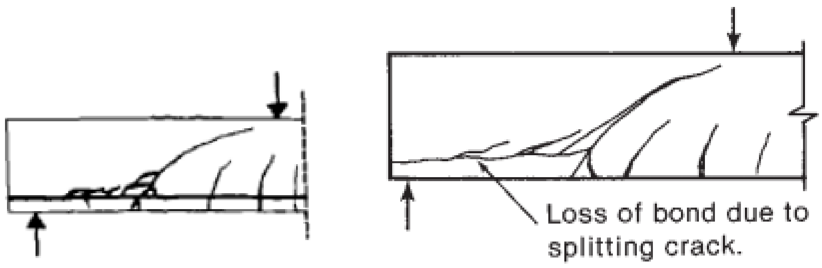


Figure 2. 6: Shear tension failure [1, 2]

On the other hand, if the diagonal shear crack propagates through the beam, causing failure when it reaches the compression zone without any sign of secondary cracks as is described in shear tension failure, it referred to a shear compression failure as shown in Figure 2.7. This failure mode applies to short beams.

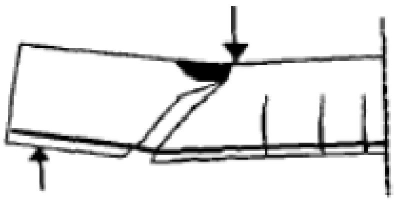


Figure 2. 7: Shear compression failure [2]

Flexural cracks are mostly moment dependent in long beams. Consequently, the cracks develop where the maximum moment is in the concrete beam. When the shear stress in the concrete reaches its tensile strength, the cracks develop. The flexural failure usually occurs in the concrete beams with a shear span to depth ratio more than 6. In this case, cracks are mainly vertical in the middle third of the beam span and perpendicular to the line of principal stress.

In the beginning, a very few fine vertical cracks start to develop in the mid-span area at about 50% of the flexure failure. The additional cracks develop in the central region, and the initial cracks widen and deeper towards the neutral axis when the external loads increase continuously. Cracks initiates with few vertical flexural cracks at mid-span and stop propagating as the destruction of the bond occurs between longitudinal bars and the concrete at support region. Next, an inclined crack steeper than in the diagonal tension case will develop to propagate towards the neutral axis shown in Figure 2.8. The rate of its propagation is slower with the crushing of concrete in the top of compression fibers; it will cause the failure to occur without warning.

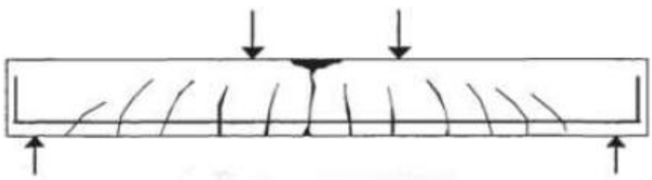


Figure 2. 8: Flexural failure [2]

2.1.3 Shear strength of beams according to Euro Code 2

Shear reinforcement is not normally required when the design ultimate shear force VEd does not exceed VRd,c. The shear resistance depends on several factors, such as the amount of flexural steel, compression strength of concrete, types of aggregate and effective depth of the section. The shear strength capacity of a concrete section without shear reinforcement given by the empirical expression in EC2 [5] as follows:

$$V_{Rd,c} = \left[C_{Rd,c}k(100\rho_1 f_{ck})^{1/3} + K_1\sigma_{cp}\right]b_w d \quad (2.2)$$

With a minimum value of $V_{min} = [0.035k^{3/2}{f_{ck}}^{1/2} + K_1\sigma_{cp}]b_w d$ (2.3)

Where,

$$C_{Rd,c} = {}^{0.18}/_{\gamma_m}, \gamma_m = 1.5, C_{Rd,c} = 0.1 \quad (2.4)$$

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \text{ With d in mm} \quad (2.5)$$

$$\rho_1 = \frac{A_{sl}}{b_w d} \leq 0.02 \quad (2.6)$$

$$A_{sl}, which\ extends\ \geq (l_{bd} + d)\ beyond\ the\ section\ considered \quad (2.7)$$

$$\sigma_{cp} = \frac{N_{Ed}}{A_c} < 0.2f_{cd} \quad (2.8)$$

For members with loads applied on the upper side within a distance $0.5d \leq a_v \leq 2d$ from the edge of a support (or center of bearing where flexible bearings used), the contribution of this load to the shear force V_{Ed} may multiply by $\beta = a_v/2d$. This reduction may apply for checking $V_{Rd,c}$. This only valid provided that the longitudinal reinforcement fully anchored at the support. For $a_v \leq 0.5d$ the value a_v = 0,5d should be used. The applied shear force V_{Ed}, calculated without reduction byβ, should however always satisfy the condition

$$V_{Ed} \leq 0.5b_w d v f_{cd}, \quad (2.9)$$

$$\text{Where } v = 0.6\left[1 - \frac{f_{ck}}{250}\right]$$

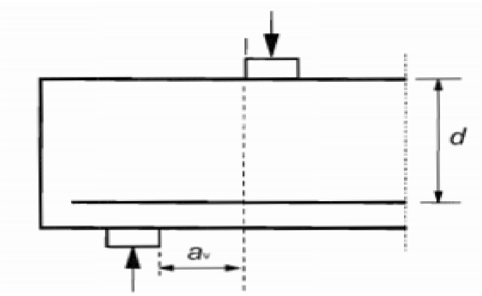


Figure 2. 9: Beam with direct support [5]

The Euro code approach to the design of members with shear reinforcement based on a truss model. The internal force state identified with the truss shown in Figure 2.10, where the top chord represented by compressed concrete, and the bottom chord represented by longitudinal reinforcement subjected to tension [10]. Between the horizontal forces Fcd and Fctd, there is a shear zone containing compressed concrete struts divided by cracks, inclined to the horizontal axis by an angle of θ and the shear reinforcement inclined by an angle of α. The distance between the top and bottom chords assumed approximately equal to d.

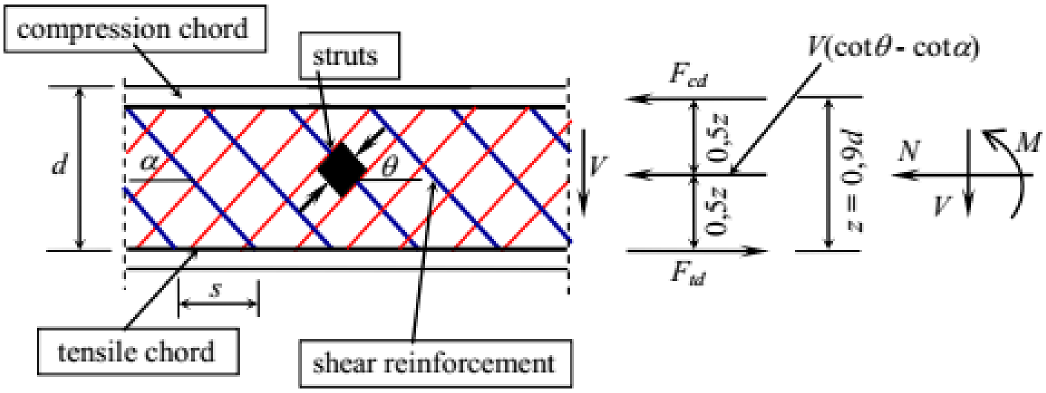


Figure 2. 10: Truss model based on Euro code 2[5].

In the shear analysis of reinforced concrete without axial force, the approximate value z = 0.9d may use. The values of $cot\theta$ for use in a country has its limits, and it may found in its National Annex. The recommended within:

$$1.0 \leq cot\theta \leq 2.5 \tag{2.10}$$

Vertical links have widely been used as shear reinforcement to carry the shear stresses in beams. The cracks form an angle of 45 degrees to the neutral axis, then the crack horizontal length nearly to d shown in the Figure below.

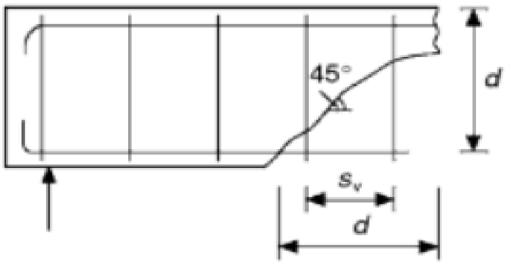


Figure 2. 11: Shear resistance of link

The crushing strength or the design value of the maximum shear force which can be sustained by the member,

$$V_{Rd,max} = \frac{\alpha_{cw} b_w \, z \, v_1 \, f_{cd}}{(cot\,\theta + tan\,\theta)} \tag{2.11}$$

Where: - $A_{Sw} = 2\frac{\pi \emptyset_s{}^2}{4}$ (2.12)

z = 0.9d (2.13)

The value of α_{cw} is 1 for non-pre stressed structures

The value of the strength reduction factor for concrete cracked in shear, v_1

For the design stress of the shear reinforcement is below 80 % of the characteristic yield

Stress f_{yk}, v_1 taken as

$$v_1 = 0.6 \quad for\ f_{ck} \leq 60MPa \tag{2.14}$$

$$v_1 = 0.9 - \frac{f_{ck}}{200} > 0.5 \quad for\ f_{ck} \geq 60MPa \tag{2.15}$$

$$f_{cd} = \frac{0.85 f_{ck}}{\gamma_c} \tag{2.16}$$

Shear strength due to stirrups strength, $V_{Ed} \leq V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} \cot\theta$ (2.17)

For V_{Ed} < $V_{Rd,\max \cot\theta}$ = 2.5, then we calculate area of shear reinforcement:

$$\frac{A_{sw}}{s} = \frac{V_{Ed}\, b_w}{f_{ywd} \cot\theta} \tag{2.18}$$

Checking the maximum spacing for vertical shear reinforcement:

$s_{l,max}$= 0.75d (2.19)

If V_{Ed} > $V_{Rd,\max \cot\theta}$ = 2.5, the we check V_{Ed} > $V_{Rd,\max \cot\theta}$ = 1.0

If it is greater than 1.0 or 22º < θ < 45º, then we determine θ from

$$\theta = 0.5 \sin^{-1}\left[\frac{V_{Ed}}{0.20 f_{ck}(1-(1-\frac{f_{ck}}{250})}\right] \tag{2.20}$$

The shear resistance for the members with inclined shear reinforcement is the smaller value of

$$V_{Rd,s} = \frac{A_{SW}}{s} Z f_{ywd}(\cot\theta + \cot\alpha)\sin\alpha \tag{2.21}$$

$$V_{Rd,max} = \alpha_{cw} b_w z v_1 f_{cd}(\cot\theta + \cot\alpha)/(1 + \cot^2\theta) \tag{2.22}$$

The maximum effective shear reinforcement, $A_{sw,max}$, for $\cot\theta = 1$ is given by:

$$\frac{A_{sw,max}\ f_{ywd}}{b_w s} \leq \frac{\frac{1}{2}\alpha_{cw} v_1 f_{cd}}{\sin\alpha} \tag{2.23}$$

The minimum area of shear reinforcement in beams, should be calculated from

$\rho_{w,min}$ = 0.08 · ($fck^{0.5}$) / fyk (2.24)

The vertical shear reinforcement ratio is, $\frac{A_{sw}}{s b_w} \geq \rho_{w,min}$, (2.25)

Minimum links of $A_{sw}/S = 0.08\, f_{ck}{}^{0.5} b / f_{yk}$ (2.26)

The inclined shear reinforcement ratio is

$$\rho_{w,min} = \frac{A_{sw}}{s b_w \sin(\alpha)} \tag{2.27}$$

For calculate area of inclined, shear reinforcement:

$$A_{sw} = \frac{V_{Ed}\, b_w \sin(\alpha)}{f_{ywd} \cot\theta} \tag{2.28}$$

The ratio of shear reinforcement given by

$$\rho_w = A_{sw}/(s.b_w.\sin\alpha), \qquad \rho_w \ \textit{should not be less than}\ \rho_{w,min} \tag{2.29}$$

$$Where,\ \rho_{w,min} = \left(0.08\sqrt{f_{ck}}\right)/f_{yk} \tag{2.30}$$

b_w is the width of the web and s is the spacing of the shear reinforcement along the length of the member.

The angle α corresponds to the angle between shear reinforcement and the longitudinal axis. For typical shear reinforcement with perpendicular legs α = 90° and sin (α) = 1.

For members with loads applied on the upper side within a distance $0.5d \leq a_v \leq 2d$ from the contribution of this load to the shear force V_{Ed} may be multiplied by $\beta = a_v/2d$.The applied shear force V_{Ed}, calculated in this way, should satisfy the condition

$$V_{Ed} \leq A_{sw} f_{ywd} \sin\alpha \tag{2.31}$$

Where, $A_{sw} f_{ywd}$ is the resistance of the shear reinforcement crossing the inclined shear crack between the loaded areas. Only the shear reinforcement within the central $0.75a_v$ should take into account. The reduction by β should only apply for calculating the shear reinforcement. It is only valid if the longitudinal reinforcement fully anchored at the support.

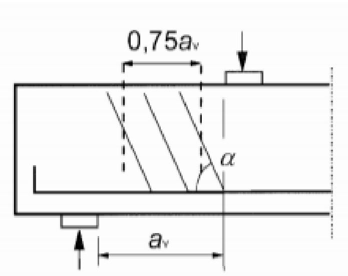


Figure 2. 12: Shear reinforcement in short shear spans with direct strut action [5]

For the shear span, $a_v < 0.5d$, the value $a_v = 0.5d$ (2.32)

The maximum longitudinal spacing between shear assemblies should not exceed $S_{l,max}$.

$$S_{l,max} = 0.75d(1 + \cot\alpha) \tag{2.33}$$

The transverse spacing of the legs in a series of shear links should not exceed$S_{l,max}$.

$$S_{l,max} = 0.75d \leq 600mm \tag{2.34}$$

2.1.4 Tekla Tedds Software for RC beams design

Tekla Tedds is powerful software developed to meet the needs of the structural engineering workflow and designed to automate the repetitive structural calculations. It combines the structural calculations with 2D frame analysis. Tedds has been written by structural engineers for structural engineers, and it should meet the requirements of everyday engineering tasks. [6] To perform calculations in Tedds, we simply need to select the calculation that we need in the Tedds calculation library. Tedds automatically runs the calculation, and prompts we for any additional information that is needed. If the calculation requires information that we would traditionally obtain from a printed source (such as a book of section properties, safe load tables, or code graphs), Tedds allows us to select the details in a data list, a data table, or a data graph. Once we have completed the input and made the appropriate selections, Tedds completes the calculations and displays the results. Once we have also performed the calculation, we can save it to disc, send it to a range of destinations, print it, or recalculate it. [6]

When we launch Tedds, the Tedds Start wizard typically displays two editions [6]:

1. Tedds
2. Tedds for Word

Tedds

Tedds is very simple and powerful and completely self-contained. In addition, Tedds requires no knowledge of any other package. This edition gives us access to all the major Tedds calculations and to most of the utilities.

Tedds for Word

Tedds for Word gives us access to all Tedds calculations (including component calculations) and all the utilities. However, Tedds for Word is even more powerful: we can include multiple Tedds calculations in the same document along with text, pictures, and output from other applications. Tedds for Word is entirely integrated with and operates within Microsoft Word. Using Tedds for word we can:

- Define our own calculations in any Microsoft word document quickly and simply,
- Access standard and component calculations by using the Library Access System,
- Include engineering data in our calculations by using data lists, tables, and graphs,
- Calculate anything from a single calculation to an entire document,
- Define multiple calculation sections in our documents, so that the same variables can have different values within our documents.

2.1.5 Nonlinear Finite Element Analysis

ABAQUS is a very powerful finite element analysis tool due to its broad selection of materials and elements and its capacity to model one, two, and three-dimensional projects. The ABAQUS program suite includes three major products: ABAQUS/CAE, ABAQUS/Standard, and ABAQUS/Explicit. The first product refers to Complete ABAQUS Environment, and used to create, analyze, and visualize model output all in one environment using GUI. ABAQUS/CAE gives the option of creating the model geometries using the software drawing tools, or importing CAD models that have been prepared by another compatible product. Users can then submit the assembled and meshed model parts for analysis. The results reviewed and graphed by the help of the available comprehensive visualization tools. ABAQUS/Standard generally used for finite element simulations of structures that subjected to static and low-speed dynamic effects. ABAQUS/Explicit on the other hand is more suitable for transient dynamic and highly nonlinear simulations. However, ABAQUS/CAE supports both Standard and Explicit version for pre-processing and post-processing simulations [7].

2.1.6 Concrete Damaged Plasticity Model

Concrete Compression Model

The stress-strain relation for a given concrete can accurately described based on uniaxial compression tests carried out on it. Having obtained a graph from laboratory tests one should transform the variables. Inelastic strains $\tilde{\varepsilon}_c{}^{in}$ used in the CDP model. In order to determine them one should deduct the elastic part (corresponding to the undamaged material) from the total strains registered in the uniaxial compression test:

$$\tilde{\varepsilon}_c{}^{in} = \varepsilon_c - \tilde{\varepsilon}_{oc}{}^{el} = \varepsilon_c - \frac{\sigma_{co}}{E_{cm}} \qquad (2.35)$$

Where $\tilde{\varepsilon}_c{}^{in}$ the inelastic strain, ε_c is the total compressive strain, $\tilde{\varepsilon}_{oc}{}^{el}$ is the elastic compressive strain corresponding to the undamaged material, σ_{co} is the compressive stress, and E_{cm} is the initial undamaged modulus of elasticity.

It is observed that concrete behaves linearly within the elastic region until the initial yield, σ_{co}.After reaching the initial yield point, concrete starts behaving in a plastic fashion and exhibits some work-hardening up to the ultimate stress, σ_{cu} followed by strain-softening.

The compressive damage parameter dc needs to define at each inelastic strain level. It ranges from zero, for an un-damaged material, to one, when the material has totally lost its load-bearing capacity. The value dc obtained only for the descending branch of the stress-strain curve of concrete in compression using the expression

$$d_c = 0 \quad \varepsilon_c < \varepsilon_{c1} \tag{2.36}$$

$$d_c = \frac{f_{ctm} - \sigma_c}{f_{ctm}} \; \varepsilon_c \geq \varepsilon_{c1} \tag{2.37}$$

Euro code 2 specifies the modulus of elasticity for concrete to be secant in a range of 0–0.4fcm. Having defined the yield stress-inelastic strain pair of variables, one needs to define now degradation variable dc. It ranges from zero for an undamaged material to one for the total loss of load-bearing capacity. These values can also have obtained from uniaxial compression tests, by calculating the ratio of the stress for the declining part of the curve to the compressive strength of the concrete. The CDP model allows one to calculate plastic strain from the formula:

$$\tilde{\varepsilon}_c{}^{pl} = \tilde{\varepsilon}_c{}^{in} - \frac{d_c}{(1-d_c)} \frac{\sigma_c}{E_{cm}} \tag{2.38}$$

Where, E_{cm} indicate the initial modulus of elasticity for the undamaged material. Knowing the plastic strain and having determined the flow and failure surface area one can calculate stress for uniaxial compression and its effective stress $\bar{\sigma_c}$.

$$\sigma_c = (1 - d_c) E_{cm} \left(\varepsilon_c - \tilde{\varepsilon}_c{}^{pl} \right), \tag{2.39}$$

$$\bar{\sigma_c} = \frac{\sigma_c}{(1-d_c)} = E_{cm} \left(\varepsilon_c - \tilde{\varepsilon}_c{}^{pl} \right) \tag{2.40}$$

Using the above equations were applied to generate the compressive behavior of concrete damage plasticity data shown in the figure

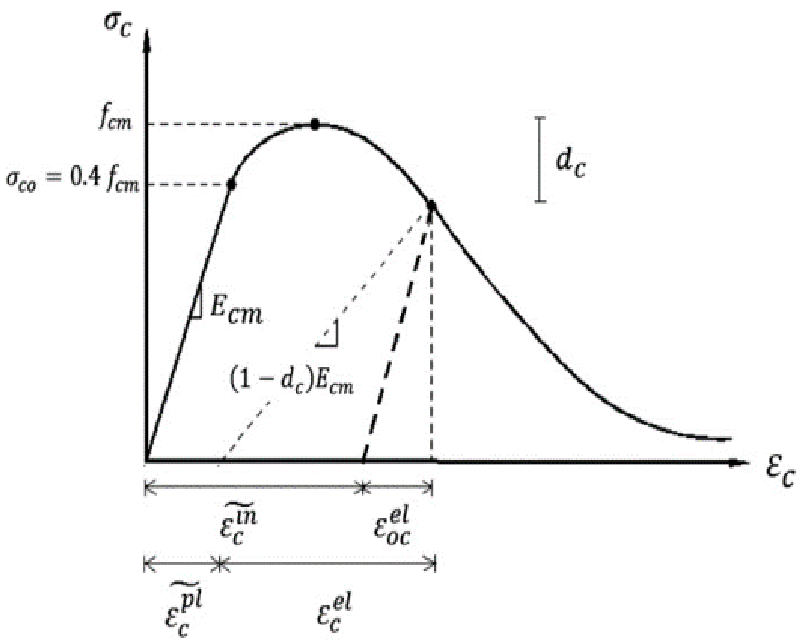


Figure 2. 13: CDP Model in compression [7]

Concrete Tension Model

Concrete under tension not regarded as a brittle-elastic body and such phenomena as aggregate interlocking in a crack and concrete-to-steel adhesion between cracks taken into account. This assumption is valid when the pattern of cracks is fuzzy. Then stress in the tensioned zone does not decrease sharply but gradually. The strain after cracking defined as the difference between the total strain and the elastic strain for the undamaged material:

$$\tilde{\varepsilon}_t{}^{ck} = \varepsilon_t - \tilde{\varepsilon}_{ot}{}^{el} = \varepsilon_t - \frac{\sigma_t}{E_{cm}} \quad (2.41)$$

Where $\tilde{\varepsilon}_t{}^{ck}$ is the cracking strain, ε_t is the total tensile strain, $\tilde{\varepsilon}_{ot}{}^{el}$ is the elastic tensile strain corresponding to the undamaged material, σ_t is the tensile stress, and E_{cm} is the initial undamaged modulus of elasticity.

The term cracking strain $\tilde{\varepsilon}_t{}^{ck}$ used in CDP model numerical analyses. The aim is to take into account the phenomenon called tension stiffening. In order to plot curve σt – εt one should define the form of the weakening function. The proper relation proposed by, among others, Wang and Hsu [8]

$$\sigma_t = E_c\varepsilon_t \text{ if } \varepsilon_t \leq \varepsilon_{cr} \quad (2.42)$$

$$\sigma_t = f_{cm}\left(\frac{\varepsilon_{cr}}{\varepsilon_t}\right)^{0.4} if\ \varepsilon_t > \varepsilon_{cr} \quad (2.43)$$

Where, ε_{cr} stands for strain at concrete cracking. Since tension stiffening may considerably affect the results of the analysis and the relation needs calibrating for a given simulation, it proposed to use the modified Wang & Hsu formula for the weakening function: [8]

$$\sigma_t = f_{cm}\left(\frac{\varepsilon_{cr}}{\varepsilon_t}\right)^{n} if\ \varepsilon_t > \varepsilon_{cr} \quad (2.44)$$

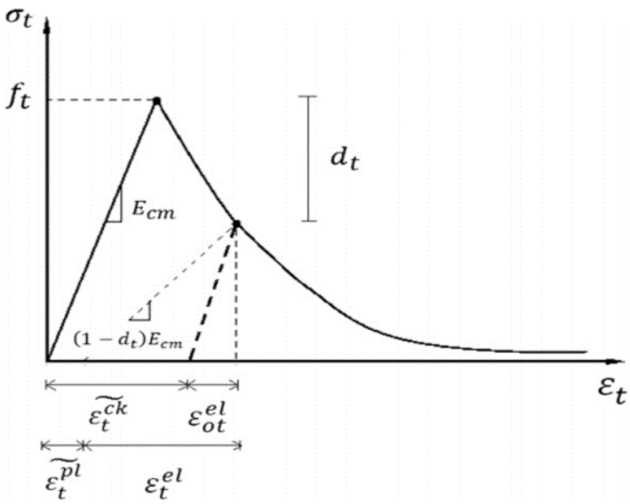


Figure 2. 14: CDP model in tension [7]

In a nonlinear analysis, ABAQUS software requires the input of the steel stress-strain curves in the form of true stress versus true plastic strain. Most materials that exhibit ductile behavior (large inelastic strains) yield at stress levels that are orders of magnitude less than the elastic modulus of the material, which implies that the relevant stress and strain measures are "true" stress (Cauchy stress) and logarithmic strain. Material data for all of models given in these measures. For nominal stress-strain data for a uniaxial test and isotropic material, a simple conversion to true stress and logarithmic plastic strain is [7]

$$\sigma_{true} = \sigma_{nom}(1 + \varepsilon_{nom}) \tag{2.45}$$

$$\varepsilon_{In}{}^{pl} = In(1 + \varepsilon_{nom}) - \frac{\sigma_{true}}{E} \tag{2.46}$$

Where E is the Young's modulus

2.2 Previous Studies on Shapes of Shear Reinforcement

Piyamahant [9] showed that the existing reinforced concrete structures should have stirrup reinforcement equal to the minimum requirement specified in the code. The theoretical analysis shows that the amount of stirrup of 0.2% is appropriate. The paper concluded that a small amount of web reinforcement is sufficient to improve the shear carrying capacity. The study focused on the applicability of the superposition method used in predicting the shear capacity of the RC beam with a small amount of web reinforcement at the shear span ratio of three.

2.2.1 Inclined Shear Reinforcements and Bent- up Bars

The inclined shear reinforcement has not been popular in the construction industry. Studies have been carried out to increases the shear capacity of the links. Colajanni et al. [10] suggested that combined vertical and inclined links increased the shear capacity of the beams. Because the vertical links only limit the shear capacity due to the high compression in the concrete stress field. Saravanakumar and Govindaraj [11] also reported improved shear resistance and stiffness as well as reducing the deflection of the beam with combined inclined and vertical links.

The use of independent bent-up bars with sufficient anchorage of shear reinforcement shown in Figure 2.13, and 2.14, have been studied at the Faculty of Civil Engineering, Universiti Teknologi Malaysia [12], which proved to have some shear capacity. As an extension of this work, independent bent-up bars with an anchorage of 50 mm and inclined links studied, which produced almost the same findings as the previous study.

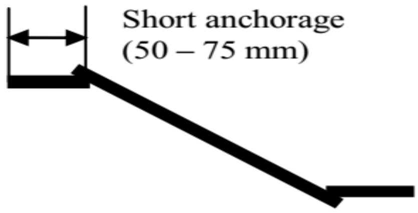


Figure 2. 15: Independent bent-up bars for Short anchorage beyond the bend

The experimental results showed that the bent-up bars have the potential to improve the shear performance of RC beams. The bent-up bars used to be common practice in the past, but its popularity decreased lately for practical reasons. The test has also shown that this type of shear reinforcement was capable of carrying shear even without any vertical links.

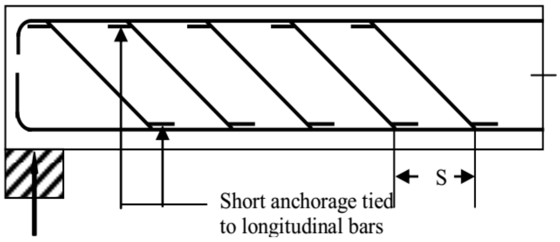


Figure 2. 16: Independent bent-up bars for multiple system of Independent Bent-Up Bars

Further studies on independent bent-up bars have brought to the experimental investigation of the modified shear reinforcement system termed as-welded inclined bars (WIB) shown in Figure 2.15. In this system, inclined bars welded to the top and bottom holding bars and thus eliminating the need to provide anchorage in the case with independent bent-up bars. The studies found that the capacity of the beams in shear was quite large, and many of the specimens did not fail in shear, but instead, they failed in flexure.

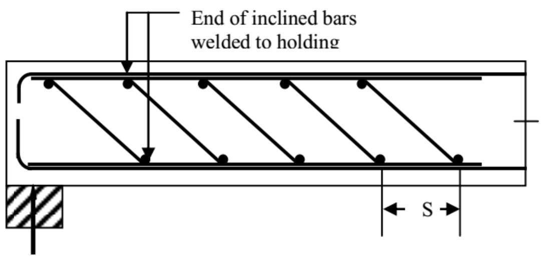


Figure 2. 17: Multiple System of welded inclined bars

2.2.2 Swimmer Bars

Moayyad M. Al-Nasra et al. [3], this study focuses on the use of different types of shear reinforcement in the RC beams. The four different types of shear reinforcement are the traditional stirrups, welded swimmer bars, bolted swimmer bars, and u-link bolted swimmer bars. There is an improvement in the shear strength of reinforced concrete beams using swimmer bars in general. The bolted swimmers bars and U-links showed similar results with the welded. The width and length of the cracks observed less using swimmer bars compared to the traditional stirrups system. The bolted and u-link bolted swimmer bars exhibit similar behaviors with the traditional stirrups system.

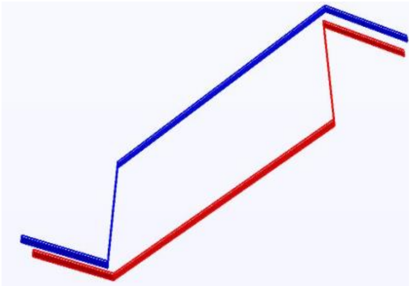


Figure 2. 18: Coupled space swimmer bars

Similarly, Muneeb Ullah Khan et al. [13] have studied the effect of using swimmer bars as the shear reinforcement at 45 degrees in reinforced concrete beams. He observed the ultimate load-carrying capacity of reinforced concrete beams with swimmer bars found to be more, and improved the shear capacity of reinforced concrete beams compared to reinforced concrete beam, which has a traditional stirrup

2.2.3 Spiral or Helical Reinforcements

N Karthik Krishna et al. [14] In this paper, six reinforced concrete beams experimentally investigated to show the improvement in load-carrying and ductility of reinforced concrete beams using the normal with helical shear reinforcements. The effectiveness of helical reinforcements in reinforced concrete beams evaluated by considering the shear cracks and the load defection reinforced concrete beams. The results obtained and compared using ANSYS software, and all the six beams tested under three-point loading, the results plotted. From the results, reinforced concrete beam with helical reinforcement has a higher ultimate load-bearing capacity than a normally reinforced concrete beam.

2.2.4 Steel Truss Reinforcements in Reinforced Concrete Beam

Sooryaprabha M Saju and Usha.S [15] they have conducted an experimental investigation on the flexural strength of steel truss reinforcement in reinforced concrete beams. From the results, they also concluded that reinforced concrete beams with an arrangement of steel truss reinforcement shown an increase in load-carrying capacity, stiffness, and strength; and less deflection.

Rudy Djamaluddin et al. [16] in their research, the effect of truss reinforced concrete beams developed the same load-carrying capacity compared to commonly use reinforced concrete beams; this study also showed that reinforced concrete beams increased in stiffness, and slower development of shear cracks.

2.2.5 Experimental Study Used for Model Validation

A reinforced concrete beam was model based on experimentally tested and reported by Roslli Noor Mohamed at Universiti Teknologi Malaysia, Malaysia [19]. The type of cement used in this study is Ordinary Portland Cement (OPC). 10mm diameter size aggregates and river sand used as coarse and fine aggregates in this mixture.

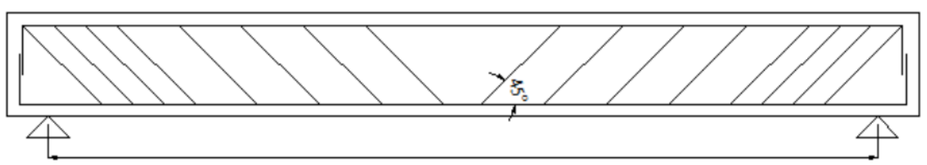
The slump test gave a result of 60 mm. Trial mix carried-out prior to the actual testing including the preparation of six concrete cubes with dimension of 100mm x 100mm x 100mm. The concrete specimens left in water tank for curing process for a period of 28 days. The concrete mix proportion given in Table 1.

Table 2. 1: Concrete mix proportions used in the experimental validation models

Concrete materials	**Mix proportions (kg/m3)**
Cement	379.6
Fine aggregate	849.9
Coarse aggregate	920.6
Water	205

The RC beam with inclined links details (RCB-B)

Reinforced concrete beam designed to achieve 30N/mm^2 at 28 days. The beam tested under a four-point loading arrangement. The loading positioned in length of 309mm from support, with the ratio of shear span to the effective depth (av/d) is 1.5. The beam dimension is 150mm in width, 250mm in height, and 2200mm long. The steel used in this experiment was high tensile strength steel with a grade of 460 N/mm^2 (denoted by T) for reinforcement steel and low tensile strength steel with a grade of 250 N/mm^2(denoted by R) for shear links. In detail, the bar size for links, tension, and compression reinforcement used were 6mm, 16mm, and 10 mm respectively. RC beam with inclined links of 45^o as shown in Figure below



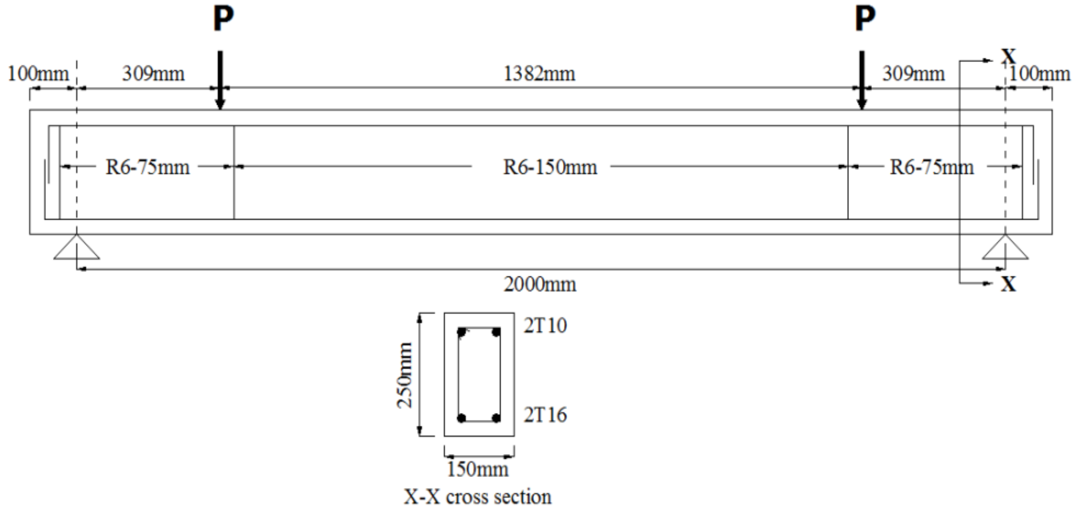


Figure 2. 19: Four-point loaded RC beam with inclined links of 45^0 (RCB-B)

Test set up and loading

The RC beam (RCB-B) was tested using a universal testing machine when the concrete reached 28 days. The main testing apparatus used were Magnus Frame, Data Logger, and hydraulic jack. Load applied using a 1000kN capacity hydraulic jack in compression. The jack was equipped with a calibrated 200kN load cell to measure the load applied during the increment process. The arrangement of the testing apparatus shown in the Figure below.

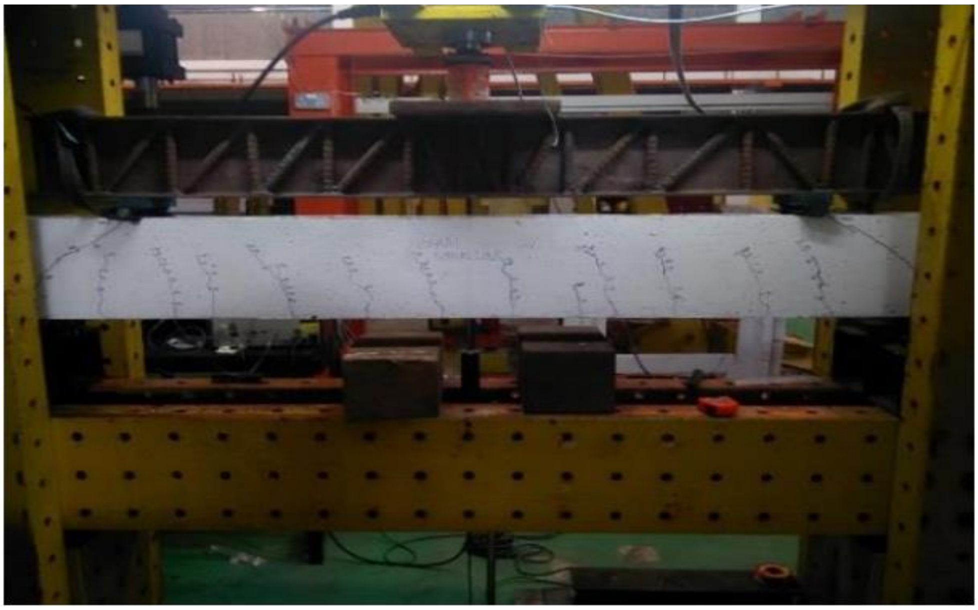


Figure 2. 20: Load test arrangement used in validation model

A linear variable displacement transducer (LVDT) was vertically attach at the concrete soffit at the center of beam's span in order to measure the deflection of tested beam.

The load was applied manually using hydraulic jack, at interval of 10 kN. During the load interval, strain at reinforcement and concrete surface recorded and checked for any crack propagation.

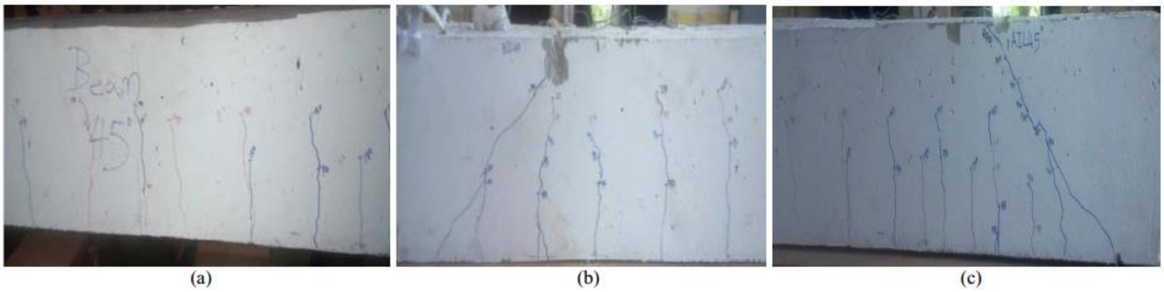


Figure 2. 21: Mode failure for RCB-B at the mid span (a), left side (b) and right side(c)

2.3 Summary and Research Gaps of the Literature

Generally, the analyses that previously done on the shapes of shear reinforcement have gaps in clearly determining the effects in reinforced concrete beams. Since the problem dominated by material properties, most studies related to shear are experimental researches as described in the above literature reviews. However, the experimental work is expensive and usually limited by the size of the facilities, the type of member or design parameters investigated in a particular set of experiment. In addition, the researchers have not identified the most efficient shape using different angles of shear reinforcements to resist shear cracks under static loads in the different span length of reinforced concrete beams based on Euro code 2 by comparing different shapes of shear reinforcement with the vertical stirrups using the Finite Element Method (FEM).

CHAPTER THREE

RESEARCH METHODOLOGY

3.1 Research Design

Reinforced concrete beams design that follows standard code was model using the Finite Element Method. All the models with different shear reinforcement shapes in reinforced concrete beams are identical in the cross-section along different span length based on the design, material properties, and amount of longitudinal steel.

In this study, a nonlinear Finite Element Method implemented to evaluate the shear resistance of reinforced concrete beams under monotonic loading. Design as a general-purpose simulation tool, the commercial software ABAQUS implemented to create quickly and easily the models by allowing the geometry of Physical and material properties together with the loads and boundary conditions to analyze and decompose the geometry into mesh regions. The assemblage of these elements idealizes the geometry of a structure by specifying the select points in space called nodes (grid points).

In this research, four phases developed to find the respective solution for the objective. In the first phase, a review of essential literature on shear reinforcement shapes and their corresponding response towards shear response in the concrete beam was covered. This review helps in getting the information about the different shear reinforcements shapes in reinforced concrete beams.

The second phase includes designing of reinforced concrete beams based on the variable strut inclination method as adopt in EC2 before proceeding to the analysis of shear reinforcement shapes. In the third phase, reinforced concrete beams were model using the software. In the fourth phase of the study, the finite element analysis of the shear reinforcement shapes carried out. After the analysis, the most efficient inclined shear reinforcement shapes in shear resisting compared to an identical reinforced concrete beam, which has vertical stirrups subject to monotonic loading.

3.2 Study Variables

3.2.1 Dependent Variables

The dependent variables considered in this study are

- Shear strength of reinforced concrete beams,
- Load- Deflection of reinforced concrete beams.

3.2.2 Independent Variables

The independent variables considered in this study are

- Cross-section of reinforced concrete beams,
- Length of reinforced concrete beams,
- Shear reinforcement shapes or configurations and
- Shear reinforcement angles or spacing

3.3 Sample Size and Sampling Method

The sample modeling method of reinforced concrete beams consists of the following steps, these are designing reinforced concrete beams using Tekla Tedds software, and then modeling reinforced concrete beams using ABAQUS software by defining the geometry.

Based on the central objectives of this research, three dimensional reinforced concrete beams models were developed, and the various items concerned with modeling is addressed such as elements type, material property, assigning sections, defining step, interaction between elements, specify boundary conditions and load, meshing, assigning job and evaluating the results.

All the analysis of reinforced concrete beams designed with the provision of 10mm diameter reinforcement at the top and 16mm diameter reinforcement at the bottom cross-section of the beams. The center-to-center length of reinforced concrete beams kept constant considering three-span length having 2m differences (2m,4m, and 6m) and 8 mm diameter shear reinforcement used in this study.

Twenty-eight reinforced concrete beams subjected to monotonic loading were compared based on shear strength using different shear reinforcement shapes and angles. The shear carrying capacity of sixteen reinforced concrete beams using different inclined shear reinforcement shapes was analyzed under four-point loading conditions and compared with reinforced concrete beams having vertical stirrups considering the same shear reinforcement spacing using ABAQUS V6.14-5 Software. Twelve reinforced concrete beams were compared based on Tekla Tedds software designed to obtain the shear strength of the reinforced concrete beams using different shear reinforcement shapes with four-angles having 15^0 inclinations differences (45^0,60^0, $75^0$and 90^o). Reinforced concrete beams with a vertical shear reinforcement used as a control sample, and the remaining reinforced concrete beams with different shear reinforcement shapes are shown in the figures below.

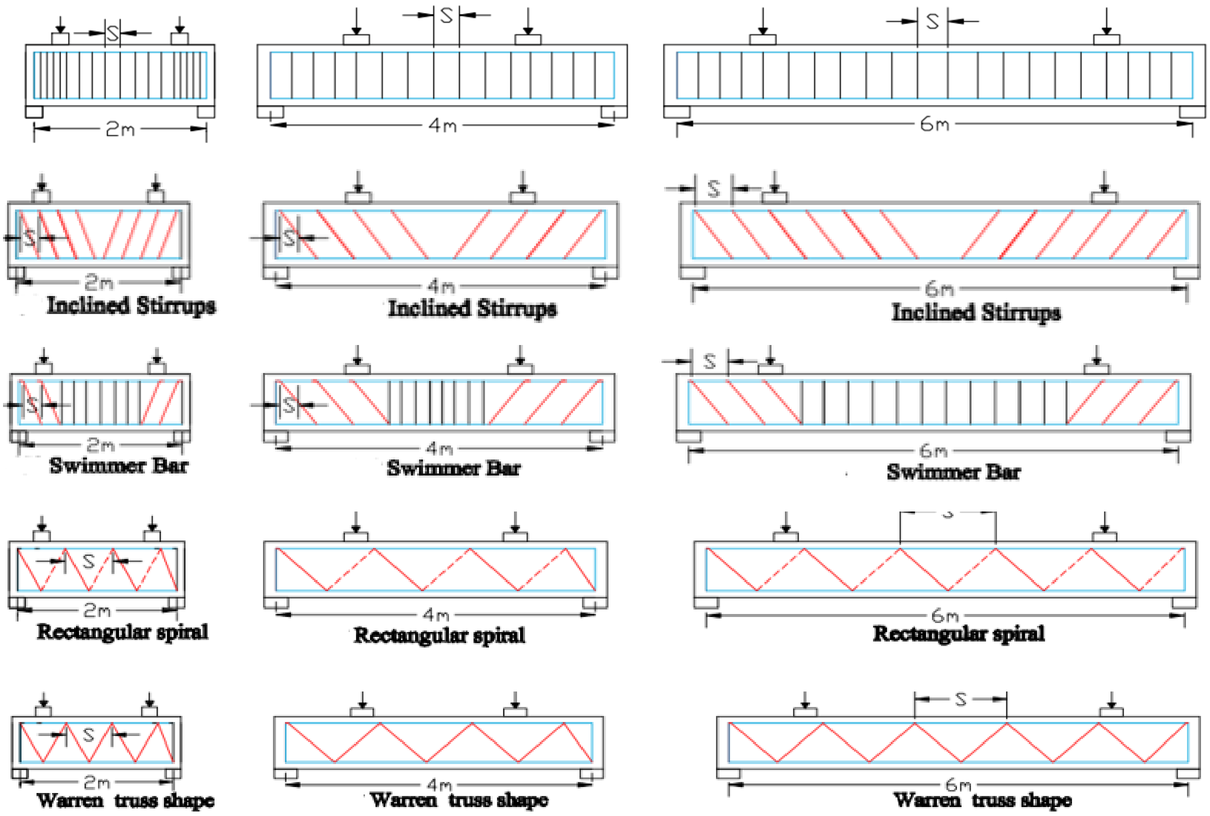


Figure 3. 1: RC beams with different shear reinforcement shapes

To identify easily reinforced concrete beams with different stirrups shapes and angles, the following notations used in the analysis as shown in the table below

Table 3. 1: Shear reinforcement notations that used in the analysis

Shear reinforcement shapes	Shear reinforcement Angles	RC beam with different shear reinforcement angles. notations	RC beam with different shear reinforcement shapes. notations
Vertical Stirrup	90^o	RCB-V	RCB-V
Inclined Stirrup	45^o	RCB-1	RCB-IS
Inclined Stirrup	60^o	RCB-2	
Inclined Stirrup	75^o	RCB-3	
Swimmer Bar	45^o	RCB-1	RCB-SB
Swimmer Bar	60^o	RCB-2	
Swimmer Bar	75^o	RCB-3	
Rectangular Spiral	45^o	RCB-1	RCB-RS
Rectangular Spiral	60^o	RCB-2	
Rectangular Spiral	75^o	RCB-3	
Warren Truss	45^o	RCB-1	RCB-WT
Warren Truss	60^o	RCB-2	
Warren Truss	75^o	RCB-3	

3.4 RC Beams Design used for finite element modeling

Instead of using assumed cross-section of beams in the finite element analysis, reinforced concrete beams designed with maximum spacing of shear reinforcements in order to compare the shear performance of the beams with different shear reinforcement shapes and angels. The reinforced concrete beams with different shear reinforcement angels (45^o, 60^o, 75^o and 90^o) designed along 2m, 4m and 6m span length that to be compared shear strength of the beams. The materials determination and designs of reinforced concrete beams done in accordance with EN1992-1-1:2004.

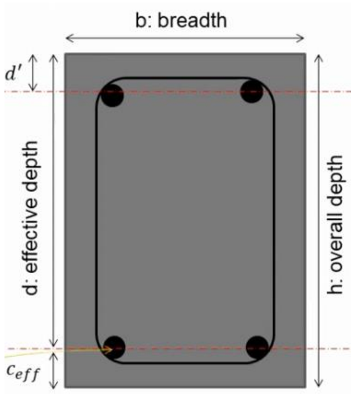


Figure 3. 2: Rectangular cross-section of RC beam

3.4.1 Procedures used for determining flexural and shear reinforcement

Procedures used for flexural reinforcement

1. Carry out analysis of beam to determine design moments (M)
2. Determine *K* from *t*he expression ,$K = \frac{M}{bd^2 f_{ck}}$ (3.1)
3. Determine *K*’ from Table 4 (EC2) or

   $K' = 0.60\delta - 0.18\delta^2 - 0.21, where\ \delta \leq 1.0$ (3.2)
4. Checking whether K ≤ *K’*?

If K ≤ K’ *No compression reinforcement required, then*

- Obtain lever arm *z* from Table 5 or using $Z = \frac{d}{2}\left[1 + \sqrt{1 - 3.53k}\right] \leq 0.95d$ (3.3)
- Calculate tension reinforcement required from $A_s = \frac{M}{{f_{yd}}^2}$ (3.4)
- Check minimum reinforcement requirements (see Table 6)

$$A_{s,min} = \frac{0.26 f_{ctm}}{f_{yk}}, where\ f_{ck} \geq 25 \quad (3.5)$$

If K>K *Compression reinforcement required, then*

- Calculate lever arm z from $Z = \frac{d}{2}\left[1 + \sqrt{1 - 3.53k'}\right]$ (3.6)
- Calculate compression reinforcement required from $A_{s2} = \frac{(K-K')f_{ck}\mathrm{bd}^2}{f_{sc}(d-d2)}$, (3.7)

  $where\ f_{sc} = 700 \leq f_{yd}$ (3.8)
- Calculate tension reinforcement required from

$$A_{s2} = \frac{K' f_{ck}\mathrm{bd}^2}{f_{yd}{}^2} + A_{s2}\frac{f_{sc}}{f_{yd}} \quad (3.9)$$

5. Check maximum reinforcement requirements A_{s_max} = 0.04 Ac for tension or compression reinforcement outside lap locations

Procedures used for deflection check

The span-to-depth ratio should ensure that deflection is limited to span/250 (3.10)

Checking the Actual L/d must be ≤ Limiting L/d x β_s (3.11)

The limiting basic span/ effective depth ratio given by;

$$L/d = K\left[11 + 1.5\sqrt{(f_{ck})}\,\rho_o/\rho + 3.2\sqrt{(f_{ck})}[(\rho_o/\rho) - 1]^{(3/2)}\right] if\ \rho \leq \rho_o \quad (3.12)$$

$$L/d = K\left[11 + 1.5\sqrt{(f_{ck})}\,\rho_o/(\rho - \rho') + 1/12\sqrt{(f_{ck})}[(p_o/p)]^{(1/2)}\right] if\ \rho > \rho_o \quad (3.13)$$

$$\rho = \frac{As_{req}}{bd} \quad (3.14)$$

Where; L/d is the limiting span/depth ratio

K is Factor to take into account different structural systems

ρ_o *is* Reference reinforcement ratio = $10^{-3} x\sqrt{(f_{ck})}$ (3.15)

ρ is Tension reinforcement ratio to resist moment due to design load

ρ' is Compression reinforcement ratio

The value of K depends on the structural configuration of the member, and relates the basic span/depth ratio of reinforced concrete members. This given in the table below;

Table 3. 2: Basic span/Effective depth ratio of structural systems [5]

Structural System	K	Highly Stressed $\rho = 1.5\%$	Lightly stressed $\rho = 0.5\%$
Simply supported beams and slabs	1.0	14	20

$$\beta_s = \frac{(500As_{prov})}{F_{yk}\,As_{req}} \tag{3.16}$$

Flange width factor; F1 = 1 = 1.000 (3.17)

Determine Factor 2 (F2)

Where the slab span exceeds 7 m and it supports

Brittle partitions, F2 = 7/l_{eff} ≤ 1.0, Otherwise F2 = 1.0 (3.18)

Determine Factor 3 (F3)

F3 = 310/σ_s (3.19)

Where σ_s = Stress in reinforcement at serviceability

limit state (see Figure 8 in EC2)

σ_s may assumed to be 310 MPa (i.e. F3 = 1.0) (3.20)

$A_{s,prov} \leq 1.5A_{s,req'd}(UK\ National\ Annex)$ (3.21)

Checking is basic *l*/*d* x *K* x F1 x F2 x F3 ≥ Actual *l*/*d* (3.22)

If it is not greater than Actual *l*/*d*, then we increase $A_{s,prov}$

Minimum area of longitudinal reinforcement

The minimum area of reinforcement is $A_{s,min} = 0.26f_{ctm}b_td/f_{yk} > 0.0013b_td$, (3.23)

where b_t is the mean width of the tension zone (see table 6 in EC2)

Maximum area of longitudinal reinforcement

The maximum area of tension or compression reinforcement, outside lap locations should not exceed $A_{s,max} = 0.004A_c$ (3.24)

Minimum spacing of reinforcement

The minimum clear distance between bars should be the greater of

$$= \begin{cases} \phi_L \\ \text{Aggregate size plus 5mm and 20mm} \end{cases} \tag{3.25}$$

Procedures used for determining shear reinforcement

1. Determine V_{Ed}

   where $V_{Ed} = design\ shear\ stress$

   $[V_{Ed} = V_{Ed}/b_wz] = V_{Ed}/(0.9b_wd)$ (3.26)

2. Determine the concrete strut capacity $V_{Rd,\max\cot\theta} = 2.5$ from Table 7

The values of $\cot\theta$ for use in a country has its limits and it may found in its National Annex.

The value recommended $1 \leq \cot\theta \leq 2.5$ (3.27)

The crushing strength $V_{Rd,\max}$ of the concrete diagonal strut of the beam given by

$$V_{Rd,max} = \frac{\alpha_{cw} b_w z v_1 f_{cwd}}{(\cot\theta + \tan\theta)} \quad (3.28)$$

The value of α_{cw} = 1 for non-prestressed structures (3.29)

Where, $v1$is the value of the strength reduction factor for concrete cracked in shear

For the design stress of the shear, reinforcement is below 80 % of the characteristic yield stress f_{yk}, $v1$ taken as

$v1 = 0.6$ for $f_{ck} \leq 60$MPa (3.30)

Checking $V_{Ed} < V_{Rd,\max\cot\theta} = 2.5$

For $V_{Ed} < V_{Rd,\max\cot\theta} = 2.5$, then we Calculate area of shear reinforcement:

$$\frac{A_{sw}}{s} = \frac{V_{Ed}\, b_w}{f_{ywd}\cot\theta} \quad (3.31)$$

With minimum links of $Asw/S = 0.08\, f_{ck}{}^{0.5} b\, /\, fyk$ (3.32)

Checking the maximum spacing for vertical shear reinforcement: $s_{l,max}$= 0.75d (3.33)

If $V_{Ed} > V_{Rd,\max\cot\theta} = 2.5$, the we check $V_{Ed} > V_{Rd,\max\cot\theta} = 1.0$(see Table 7)

If it is greater than 1.0 or 22º < θ < 45º, then we determine θ using:

$$\theta = 0.5 \sin^{-1}\left[\frac{V_{Ed}}{0.20 f_{ck}(1-(1-\frac{f_{ck}}{250})}\right] \quad (3.34)$$

The shear resistance for the members with inclined shear reinforcement is using

$$V_{Rd,max} = \alpha_{cw} b_w z v_1 f_{cd}(\cot\theta + \cot\alpha)/(1 + \cot^2\theta) \quad (3.35)$$

Minimum area of shear reinforcement

The minimum area of shear reinforcement in beams, should be calculated from

$\rho_{w,min} = 0.08 \cdot (fck^{0.5}) / fyk$, (3.36)

Where $\rho_{w,min}$ can obtained from Table 9

The vertical shear reinforcement ratio is $\frac{A_{sw}}{sb_w} \geq \rho_{w,min}$ (3.37)

The inclined shear reinforcement ratio is $\rho_{w,min} = \frac{A_{sw}}{sb_w \sin(\alpha)}$ (3.38)

For calculate area of inclined, shear reinforcement: $A_{sw} = \frac{V_{Ed}\, b_w \sin(\alpha)}{f_{ywd}\cot\theta}$ (3.39)

Where, b_w is the width of the web and s is the spacing of the shear reinforcement along the length of the member.

The angle α corresponds to the angle between shear reinforcement and the longitudinal axis. For typical shear reinforcement with perpendicular legs α = 90° and sin (α) = 1.and Checking the maximum distance between links limited to

S = 0.75 d (1 + cot α), where α is the inclination of the links to the horizontal. (3.40)

The transverse spacing of the legs in a series of shear links should not exceed$S_{l,max}$.

$S_{l,max} \leq 600mm$ (3.41)

The RC beams designed in accordance with EN1992-1-1:2004 incorporating Tekla Tedds 2019 calculation Software (version 3.2.02) as follows

Concrete details - Table 3.1. Strength and deformation characteristics for concrete

Concrete strength class; C30/37

Aggregate type; Quartzite

Aggregate adjustment factor - cl.3.1.3 (2); AAF = 1.0

Characteristic compressive cylinder strength; f_{ck} = 30 N/mm^2

Mean value of compressive cylinder strength; f_{cm} = f_{ck} + 8 N/mm^2 = 38 N/mm^2

Mean value of axial tensile strength; f_{ctm} = 0.3 N/mm^2 x (f_{ck}/ 1 N/mm^2)$^{2/3}$ = 2.9 N/mm^2

Elastic modulus of concrete; E_{cm} = 22 kN/mm^2 x [f_{cm}/10 N/mm^2]$^{0.3}$ x AAF = 32837 N/mm^2

Ultimate strain - Table 3.1; ε_{cu2} = 0.0035

Shortening strain - Table 3.1; ε_{cu3} = 0.0035

Effective compression zone height factor; λ = 0.80

Effective strength factor; η = 1.00

Coefficient k_1; k_1 = 0.40

Coefficient k_2; k_2 = 1.0 x (0.6 + 0.0014 / ε_{cu2}) = 1.00

Coefficient k_3; k_3 = 0.40

Coefficient k_4; k_4 = 1.0 x (0.6 + 0.0014 / ε_{cu2}) = 1.00

Partial factor for concrete -Table 2.1N; γ_c= 1.50

Compressive strength coefficient - cl.3.1.6 (1); α_{cc} = 0.85

Design compressive concrete strength - exp.3.15; f_{cd} = α_{cc} x fck / γ_c = 17.0 N/mm^2

Compressive strength coefficient - cl.3.1.6 (1); α_{ccw} = 1.00

Design compressive concrete strength - exp.3.15; f_{cwd} = α_{ccw} x fck / γ_c = 20.0 N/mm^2

Maximum aggregate size; h_{agg} = 20 mm

Monolithic simple support moment factor; β_1= 0.25

Reinforcement details

Characteristic yield strength of reinforcement; f_{yk} = 460 N/mm^2

Partial factor for reinforcing steel - Table 2.1N; γ_s = 1.15

Design yield strength of reinforcement; $f_{yd} = f_{yk} / \gamma_s$= 400 N/mm^2

3.4.2 Calculation of required nominal concrete cover for reinforcements

Defined parameters:

Concrete characteristic strength f_{ck}: 30 MPa

The maximum longitudinal reinforcement diameter: 16mm

Exposure classes related to environmental conditions: XC1

The maximal aggregate size: dg = 20 mm (< 32 mm).

The design working life of the structure: 50 years.

Design of the concrete cover of a reinforced concrete beam with exposure class XC1.

Calculation of structural class

The initial structural class is S4 (corresponding to design working life of 50 years)

The next working life class that is applicable for the structure is 50 years

The minimum structural class is S1

Therefore, the structural class is S4.

Calculation of concrete cover for durability

For reinforcement steel the minimum cover for durability $c_{min,dur}$ is calculated

For structural class S4 and exposure, class XC1,

$c_{min,dur}$= 10.0 mm.

Calculation of concrete cover for bond

The minimum cover for bond $c_{min,b}$ is calculated

$c_{min,b}$= 1.0·Φ, where Φ is the diameter of the reinforcement bar

Therefore, $c_{min,b}$ = 16.0 mm.

Calculation of minimum concrete cover

The greater value of concrete cover satisfying the requirements for both bond and durability is used:c_{min} = max ($c_{min,b}$; $c_{min,dur}$+ $\Delta c_{dur,\gamma}$- $\Delta c_{dur,st}$- $\Delta c_{dur,add}$; 10 mm)

The additive safety element is $\Delta c_{dur,\gamma}$ = 0.0 mm.

The following modification factors are not applicable:

Reduction of minimum cover for use of stainless steel $\Delta c_{dur,st}$= 0 mm

Reduction of minimum cover for use of additional protection $\Delta c_{dur,add}$= 0 mm.

Therefore, the minimum concrete cover calculated as:

c_{min} = max {16.0 mm, 10.0 mm + 0.0 mm - 0 mm - 0 mm, 10 mm} = 16.0 mm

Calculation of nominal concrete cover

The nominal concrete cover c_{nom} is calculated by adding to the minimum cover c_{min} the allowance for deviation Δc_{dev}.The allowance for deviation is Δc_{dev} = 10.0 mm.

The required nominal concrete cover is:

$c_{nom} = c_{min} + \Delta c_{dev}$ = 16.0 mm + 10.0 mm = 26.0 mm

Therefore, the required nominal concrete cover is c_{nom} = 26.0 mm.

Due to uneven surfaces other than the ones examined the required nominal concrete cover used c_{nom} = 30.0 mm. Therefore, 30 mm nominal cover to top reinforcement (c_{nom_t}), bottom reinforcement(c_{nom_b}) and side reinforcemen (c_{nom_s}) used.

Fire resistance

Standard fire resistance period; R = 60 min

Number of sides exposed to fire; 3

Minimum width of beam - EN1992-1-2 Table 5.5; b_{min} = 120 mm

Beam - Span 1

Rectangular section details

Section width; b = 150 mm

Section depth; h = 250 mm

PASS - Minimum dimensions for fire resistance met

3.4.3 Determination of the RC beams effective span

Defined parameters:

Height of the beam cross-section: 250mm

Width of the supporting element: 100mm

Longitudinal Reinforcement diameter: 10mm and 16mm

Shear Reinforcement diameter: 8mm

Clear Span of the beam: 1900mm

The effective span, l_{eff} of a member calculated as follows:

$l_{eff} = l_n + a_1 + a_2$…….EC (5.8)

Where:l_n The clear distance between the faces of the supports

t is the width of the supporting element.

Values for a_1 and a_2 , at each end of the span, may be determined from the appropriate a_i values in Figure below

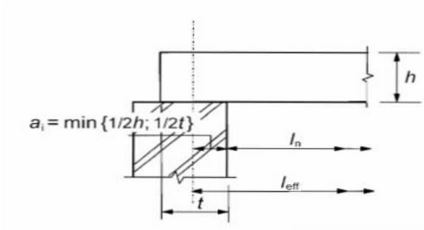


Figure 3. 3: Non-continues member [5]

$$a_1 = 0.5 * 250 = 125mm \ and \ a_2 = 0.5 * 100 = 50mm$$

Therefore, the minimum value of $a_2 = 50mm$ and l_{eff}=1900mm+50mm+50mm=2000mm

The loads applied on the upper side within a distance 0.5d and 2.0d

d=h-cover-$\frac{\emptyset}{2} - \emptyset_{link} = 250 - 30 - \frac{16}{2} - 8 = 204mm$

Minimum Shear span of the RC beams, a_v =0.5x204mm=102mm and

Maximum Shear span of the RC beams, a_v =2x204mm=408 mm

3.4.4 The design anchorage length of longitudinal reinforcement

Defined Parameters:

Steel characteristic yield strength, f_{yk} =460 Mpa

Coefficient taking account of long term effects and loading effects on the tensile strength of concrete, α_{ct} =1

Concrete partial material safety factor=1.5

Reinforcement steel partial material safety factor=1.15

Ultimate bond stress f_{bd}

The design value of ultimate bond stress for ribbed bars is

$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd}$

Where, $f_{ctd} = \alpha ct \cdot fctk, 0.05 / \gamma c$ is the design tensile strength of concrete

Formula for 5% fractile tensile strength $f_{ctk,0.05}$

$f_{ctk,0.05} = 0.7 \cdot f_{ctm}$

Characteristic tensile strength

Formula for mean tensile strength fctm

f_{ctm} [MPa] = $0.30 \cdot fck^{2/3}$ for concrete class ≤ C50/60

$f_{ctm} = 0.3 * (30)^{2/3} = 0.3 * 9.655 = 2.9\ Mpa$

$f_{ctk,0.05}$ =0.7*2.9=2.03 Mpa

$f_{ctd} = {1 * 2.03}/{1.5} = 1.35\ Mpa$

The coefficient η_2 takes into account the effect of large bar diameters Φ > 32 mm as follows:

η_2 = min [1.0, (132 - Φ) / 100], where Φ in mm

The coefficient η_1 takes a value of 1.0 when 'good' bond conditions are obtained and a value of 0.7 otherwise i.e. when 'poor' bond conditions exist.

Good' bond conditions obtained when any of the following conditions fulfilled:

Vertical bars or almost vertical bars inclined at an angle $45° \leq \alpha \leq 90°$ from the horizontal

Bars that are located up to 250 mm from the bottom of the formwork for elements with height $h \leq 600$ mm

Bars that are located at least 300 mm from the free surface during concreting for elements with height $h > 600$ mm

'Poor' bond conditions are applicable for all other cases and for bars in structural elements built with slip-forms, unless it shown that 'good' bond conditions exist.

f_{bd} =2.25x1x1x1.35=3.04 Mpa

Basic anchorage length $l_{b,rqd}$

The basic required anchorage length $l_{b,rqd}$ for anchoring a straight steel bar with diameter Φ under design stress σsd is

$l_{b,rqd} = (\Phi / 4) \cdot (\sigma sd / fbd)$

The maximum value of the design steel stress σsd under ULS loads is equal to the design yield strength of the bar fyd = fyk / γs.

fyd $=\frac{460}{1.15} = 400Mpa$

When the actual design strength of the bar is smaller than fyd then the basic required anchorage length reduced proportionally.

$l_{b,rqd} = (16/4) * (400/3.04) = 526.32mm$

Minimum anchorage length $l_{b,min}$

When no other limitation is applicable, the provided anchorage length should be at least equal to the minimum value $l_{b,min}$

-For anchorages in tension: $l_{b,min} \geq \max[0.3\cdot l_{b,req}, 10\cdot\Phi, 100 \text{ mm}]$

$l_{b,min} \geq max[0.3 * 526.32, 10 * 16, 100]$

$$\geq max[158, 160, 100]$$

$$= 160mm$$

- For anchorages in compression: $l_{b,min} \geq \max[0.6\cdot l_{b,req}, 10\cdot\Phi, 100 \text{ mm}]$

$l_{b,min} \geq max[0.6 * 526.32, 10 * 16, 100]$

$$\geq max[316, 160, 100] = 316mm$$

Design anchorage length l_{bd}

Design anchorage length l_{bd} is

$l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd} \geq l_{b,min}$

Coefficient α_1 accounts for the effect of the shape of the bar ($\alpha_1 = 1.0$ for straight bars)

Coefficient α_2 accounts for the effect of minimum concrete cover:

For any bar shape in compression $\alpha_2 = 1.0$

Coefficient α_3 accounts for the effect of confinement by transverse reinforcement not welded to main reinforcement.

For bars in compression $\alpha_3 = 1.0$.

Coefficient α_4 accounts for the effect of confinement by welded transverse reinforcement. If the requirements of *EN1992-1-1 Table 8.2* are fulfilled, then it can take the value $\alpha_4 = 0.7$.

Coefficient α_5 accounts for the effect of confinement by transverse pressure. For bars in tension, it takes values $0.7 \leq \alpha_5 \leq 1.0$ depending on the value of transverse pressure. For bars in compression, α_5 is not applicable. In any case, the lower limit of the product $(\alpha_2 \cdot \alpha_3 \cdot \alpha_5) \geq 0.7$ must observed.

As a simplified and conservative alternative the equivalent anchorage length $l_{b,eq}$ may be provided that is $l_{b,eq} = \alpha 1 \cdot l_{b,rqd}$ for straight, bend, hook, and loop bar shapes, or $l_{b,eq} = \alpha 4 \cdot$ lb,rqd for bars with welded transverse bars.

$l_{bd} = 1x526.32 \geq l_{b,min} = 160mm$

$l_{bd} = 526mm$

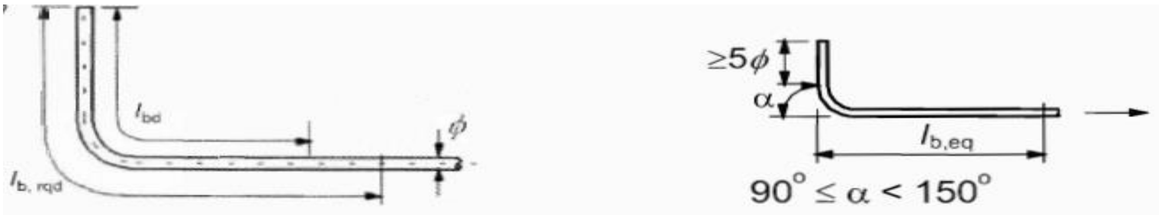


Figure 3. 4: Basic tension and equivalent anchorage length for standard bend [5]

Figure 3. 5: Anchorage of links and shear reinforcement for bend [5]

3.4.5 Moment and Shear design of the beams

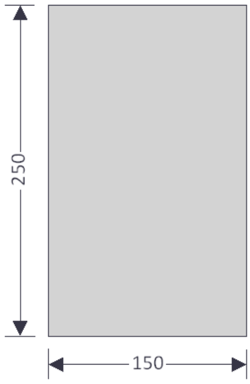


Figure 3. 6: RC beam cross-section for 2m span length

Table 3. 3: The beam cross-section and material properties used in the design

Span	Length (m)	Section(mm)	Start Support	End Support
1	2	R 150x250	Pinned	Pinned
R 150x250: Area 375 cm^2, Inertia Major 19531 cm^4, Inertia Minor 7031 cm^4, Shear area parallel to Minor 313 cm^2, Shear area parallel to Major = 313 cm^2				
Concrete (C30): Density 2500 kg/m^3, Young's 32.836568 kN/mm^2, Shear 13.6819033 kN/mm^2 ,Thermal 0.00001 ^{0}C^{-1}				

Geometry and Loading

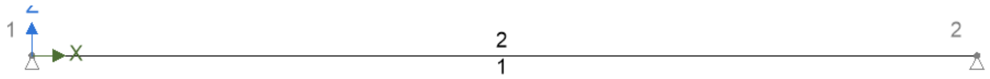


Figure 3. 7: Span-1with the length of 2m

The Two concentrated Ultimate design load applied on the beams: 100KN

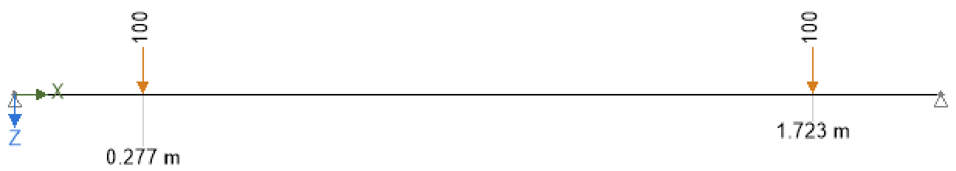


Figure 3. 8: Permanent - Loading (kN)

Moment design

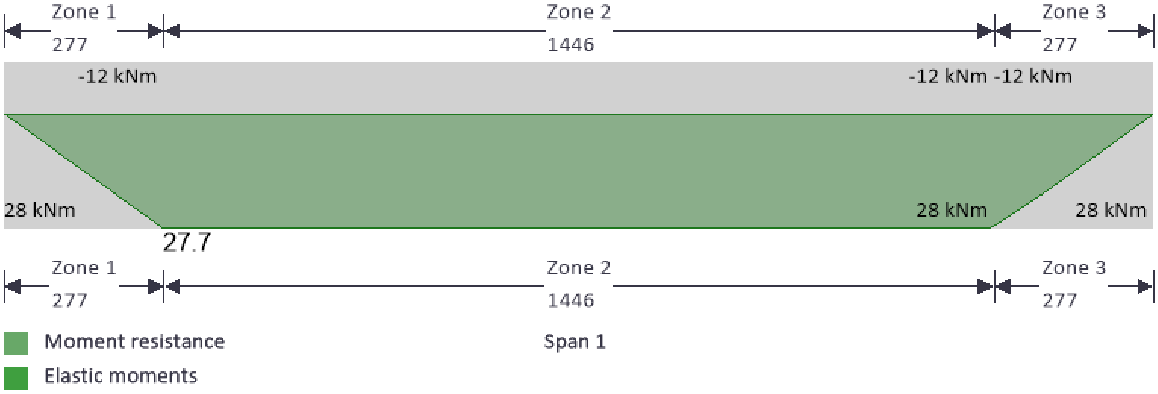


Figure 3. 9: Moment resistance and elastic moment result for 2m span length

Zone 1 (0 mm - 277 mm) Positive moment - section 6.1

Design bending moment; $M = abs\ (M\ m1\_s1\_z1\_max\_red)$ = 27.7 kNm

Effective depth of tension reinforcement; d = 204 mm

Redistribution ratio; δ = min ($M_{pos\_red\_z1\ /\ Mpos\_z1,\ 1}$) = 1.000

K = M / (b x d^2 x f_{ck}) = 0.148

K' = (2 x η x α_{cc} / γ_c) x (1 - λ x (δ - k_1) / (2 x k_2)) x (λ x (δ - k_1) / (2 x k_2)) = 0.207

K' > K - No compression reinforcement is required

Lever arm; z = min (0.5 x d x [1 + (1 - 2 x K / ($\eta\ x\ \alpha_{cc}$ / γ_c))$^{0.5}$], 0.95 x d) = 173 mm

Depth of neutral axis; x = 2 x (d - z) / λ = 79 mm

Area of tension reinforcement required; $As_{,req}$ = M / (fyd x z) = 401 mm^2

Tension reinforcement provided; 2 x 16ϕ

Area of tension reinforcement provided; $As_{,prov}$ = 402 mm^2

Min. Reinforcement area - exp.9.1N; $As_{,min}$ = max(0.26 x fctm / fyk, 0.0013) x b x d = 50 mm^2

Max. Reinforcement area - cl.9.2.1.1 (3); $As_{,max}$ = 0.04 x b x h = 1500 mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Maximum crack width; wk = 0.4 mm

Design value modulus of elasticity reinf - 3.2.7(4); Es = 200000 N/mm^2

Mean value of concrete tensile strength; $f_{ct,eff}$ = f_{ctm} = 2.9 N/mm^2

Stress distribution coefficient; kc = 0.4

Non-uniform self-equilibrating stress coefficient; k = min (max (1 + (300 mm - min (h, b)) x 0.35 / 500 mm, 0.65), 1) = 1.00

Actual tension bar spacing; s_{bar} = (b - (2 x (c_{nom_s} + $\phi_{m1\_s1\_z1\_v}$) + $\phi_{m1\_s1\_z1\_b\_L1}$ x $N_{m1\_s1\_z1\_b\_L1}$)) / ($N_{m1\_s1\_z1\_b\_L1}$ - 1) + $\phi_{m1\_s1\_z1\_b\_L1}$ = 58 mm

Maximum stress permitted (Table 7.3N); σ_s = 360 N/mm^2

Steel to concrete modulus of elast. Ratio; α_{cr} = Es / Ecm = 6.09

Distance of the Elastic NA from bottom of beam; y = (b x h^2 / 2 + As,prov x (α_{cr} - 1) x (h - d)) / (b x h + As,prov x (α_{cr} - 1)) = 121 mm

Area of concrete in the tensile zone; Act = b x y = 18137 mm^2

Min. Reinforcement area required - exp.7.1; Asc,min = kc x k x $f_{ct,eff}$ x Act / σ_s = 58 mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Permanent load ratio; R_{PL} = 0.65

Service stress in reinforcement; σ_{sr} = fyd x As,req / As,prov x R_{PL} = 260 N/mm^2

Maximum bar spacing - Tables 7.3N; $s_{bar,max}$ = 225.6 mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Zone 1 (0 mm - 277 mm) Negative moment - section 6.1

Design moment; M = max (β_1 x abs (M $_{m1\_s1\_max\_red}$), abs (M $_{m1\_s1\_z1\_min\_red}$)) = 6.9 kNm

Effective depth of tension reinforcement; d = 207 mm

Redistribution ratio; δ = 1 = 1.000

K = M / (b x d^2 x f_{ck}) = 0.036

K' = (2 x η x α_{cc} / γ_c) x (1 - λ x (δ - k_1) / (2 x k_2)) x (λ x (δ - k_1) / (2 x k_2)) =0.207

K' > K - No compression reinforcement is required

Lever arm; z = min (0.5 x d x [1 + (1 - 2 x K / η x α_{cc} / γ_c))$^{0.5}$], 0.95 x d) =197 mm

Depth of neutral axis; x = 2 x (d - z) / λ = 26 mm

Area of tension reinforcement required; As,req = M / (fyd x z) = 88 mm^2

Tension reinforcement provided; 2 x 10ϕ

Area of tension reinforcement provided; As,prov = 157 mm^2

Min. Reinforcement area- exp.9.1N; As,min = max(0.26 x fctm / fyk, 0.0013) x b x d = 51 mm^2

Maximum area of reinforcement - cl.9.2.1.1 (3); As,max = 0.04 x b x h = 1500 mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Maximum crack width; wk = 0.4 mm

Design value modulus of elasticity reinf - 3.2.7(4); Es = 200000 N/mm^2

Mean value of concrete tensile strength; fct,eff = fctm = 2.9 N/mm^2

Stress distribution coefficient; kc = 0.4

Non-uniform self-equilibrating stress coefficient; k = min (max (1 + (300 mm - min (h, b)) x 0.35 / 500 mm, 0.65), 1) = 1.00

Actual tension bar spacing; s_{bar} = (b - (2 x (c $_{nom\_s}$ + $\phi_{m1\_s1\_z1\_v}$) + ϕ $_{m1\_s1\_z1\_t\_L1}$ x N $_{m1\_s1\_z1\_t\_L1}$)) / (N $_{m1\_s1\_z1\_t\_L1}$ - 1) + ϕ $_{m1\_s1\_z1\_t\_L1}$ = 64 mm

Maximum stress permitted (Table 7.3N); σ_s = 360 N/mm^2

Steel to concrete modulus of elast. Ratio; α_{cr} = Es / Ecm = 6.09

Distance of the Elastic NA from bottom of beam; y = (b x h^2 / 2 + As,prov x (α_{cr} - 1) x (h - d)) / (b x h + As,prov x (α_{cr} - 1)) =123 mm

Area of concrete in the tensile zone; Act = b x y = 18493 mm^2

Min. Reinforcement area required - exp.7.1; Asc,min = kc x k x fct,eff x Act / σ_s = 60 mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Permanent load ratio; R_{PL} = 0.65

Service stress in reinforcement; σ_{sr} = fyd x As,req / As,prov x R_{PL} = 146 N/mm^2

Maximum bar spacing - Tables 7.3N; $s_{bar,max}$ = 300 mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing (Section 8.2)

Top bar spacing; s_{bar} = (b - (2 x (c $_{nom\_s}$ + $\phi_{m1\_s1\_z1\_v}$) + ϕ $_{m1\_s1\_z1\_t\_L1}$ x N $_{m1\_s1\_z1\_t\_L1}$)) / (N $_{m1\_s1\_z1\_t\_L1}$ - 1) = 54.0 mm

Minimum allowable top bar spacing; stop,min = max(ϕ $_{m1\_s1\_z1\_t\_L1}$ x k_{s1}, h_{agg} + k_{s2}, 20mm)

= 25.0 mm

PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing; s_{bot} = (b - (2 x (c $_{nom\_s}$ + $\phi_{m1\_s1\_z1\_v}$) + ϕ $_{m1\_s1\_z1\_b\_L1}$ x N $_{m1\_s1\_z1\_b\_L1}$)) / (N $_{m1\_s1\_z1\_b\_L1}$ - 1) = 42.0 mm

Minimum allowable bottom bar spacing; sbot,min = max($\phi_{m1\_s1\_z1\_b\_L1}$ x k_{s1}, h_{agg} + k_{s2}, 20mm)

= 25.0 mm

PASS - Actual bar spacing exceeds minimum allowable

Zone 2 (277 mm - 1723 mm) Positive moment - section 6.1

Design bending moment; $M = abs\ (M\ m1\_s1\_z2\_max\_red)$ = 27.7 kNm

Effective depth of tension reinforcement; d = 204 mm

Redistribution ratio; δ = min (M $_{pos\_red\_z2\ /\ Mpos\_z2,\ 1}$) = 1.000

K = M / (b x d^2 x f_{ck}) = 0.148

K' = (2 x η x α_{cc} / γ_c) x (1 - λ x (δ - k_1) / (2 x k_2)) x (λ x (δ - k_1) / (2 x k_2)) = 0.207

K' > K - No compression reinforcement is required

Lever arm; z = min (0.5 x d x [1 + (1 - 2 x K / $\eta\ x\ \alpha_{cc}$ / γ_c))$^{0.5}$], 0.95 x d) = 173 mm

Depth of neutral axis; x = 2 x (d - z) / λ = 79 mm

Area of tension reinforcement required; As,req = M / (fyd x z) = 401 mm^2

Tension reinforcement provided; 2 x 16ϕ

Area of tension reinforcement provided; As,prov = 402 mm^2

Min. Reinforcement area- exp.9.1N; As,min = max(0.26 x fctm / fyk, 0.0013) x b x d = 50 mm^2

Maximum area of reinforcement - cl.9.2.1.1 (3); As,max = 0.04 x b x h = 1500 mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Maximum crack width; wk = 0.4 mm

Design value modulus of elasticity reinf - 3.2.7(4); Es = 200000 N/mm^2

Mean value of concrete tensile strength; fct,eff = fctm = 2.9 N/mm^2

Stress distribution coefficient; kc = 0.4

Non-uniform self-equilibrating stress coefficient; k = min (max (1 + (300 mm - min (h, b)) x 0.35 / 500 mm, 0.65), 1) = 1.00

Actual tension bar spacing; s_{bar} = (b - (2 x (c_{nom_s} + $\phi_{m1\_s1\_z2\_v}$) + $\phi_{m1\_s1\_z2\_b\_L1}$ x $N_{m1\_s1\_z2\_b\_L1}$)) / ($N_{m1\_s1\_z2\_b\_L1}$ - 1) + $\phi_{m1\_s1\_z2\_b\_L1}$ = 58 mm

Maximum stress permitted (Table 7.3N); σ_s = 360 N/mm^2

Steel to concrete modulus of elast. ratio; α_{cr}= Es / Ecm = 6.09

Distance of the Elastic NA from bottom of beam; y = (b x h^2 / 2 + As,prov x (α_{cr} - 1) x (h - d)) / (b x h + As,prov x (α_{cr} - 1)) =121 mm

Area of concrete in the tensile zone; Act = b x y = 18137 mm^2

Min. Reinforcement area required - exp.7.1; Asc,min = kc x k x fct,eff x Act / σ_s = 58 mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Permanent load ratio; R_{PL} = 0.65

Service stress in reinforcement; σ_{sr} = fyd x As,req / As,prov x R_{PL} =260 N/mm^2

Maximum bar spacing - Tables 7.3N; $S_{bar,max}$ = 225.6 mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Deflection control – Section 7.4

Reference reinforcement ratio; ρ_{m0} = (fck / 1 $N/mm^2)^{0.5}$ / 1000 = 0.00548

Required tension reinforcement ratio; ρ_m = $A_{s,req}$ / (b x d) = 0.01312

Required compression reinforcement ratio; ρ'_m= $A_{s2,req}$ / (b x d) = 0.00000

Structural system factor - Table 7.4N; K_b = 1.0

Basic allowable span to depth ratio ; $span\_to\_depth_{basic}$ = Kb x [11 + 1.5 x (fck / 1 $N/mm^2)^{0.5}$ x ρ_{m0} / (ρ_m - ρ'_m) + (f_{ck} / 1 $N/mm^2)^{0.5}$ x (ρ'_m / $\rho_{m0})^{0.5}$ / 12] = 14.430

Reinforcement factor - exp.7.17; Ks = min ($A_{s,prov}$ / $A_{s,req}$ x 500 N/mm^2 / fyk, 1.5) = 1.089

Flange width factor; F1 = 1 = 1.000

Long span supporting brittle partition factor; F2 = 1 = 1.000

Allowable span to depth ratio; $span\_to\_depth_{allow}$ = min ($span\_to\_depth_{basic}$ x Ks x F1 x F2, 40 x K_b) = 15.713

Actual span to depth ratio; $span\_to\_depth_{actual}$ = L $_{m1\_s1}$ / d = 9.804

PASS - Actual span to depth ratio is within the allowable limit

Minimum bar spacing (Section 8.2)

Top bar spacing; s_{top} = (b - (2 x (c $_{nom\_s}$ + $\phi_{m1\_s1\_z2\_v}$) + ϕ $_{m1\_s1\_z2\_t\_L1}$ x N $_{m1\_s1\_z2\_t\_L1}$)) / (N $_{m1\_s1\_z2\_t\_L1}$ - 1) = 54.0 mm

Minimum allowable top bar spacing; stop,min = max(ϕ $_{m1\_s1\_z2\_t\_L1}$ x k_{s1}, h_{agg} + k_{s2}, 20mm) = 25.0 mm

PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing;sbot = (b - (2 x (cnom_s + ϕ $_{m1\_s1\_z2\_v}$) + ϕ $_{m1\_s1\_z2\_b\_L1}$ x N $_{m1\_s1\_z2\_b\_L1}$ + ϕ $_{m1\_s1\_z1\_b\_L1}$ x N $_{m1\_s1\_z1\_b\_L1}$)) / ((N $_{m1\_s1\_z2\_b\_L1}$ + N $_{m1\_s1\_z1\_b\_L1}$) - 1) = 42.0 mm

Minimum allowable bottom bar spacing; sbot,min = max(ϕ $_{m1\_s1\_z2\_b\_L1}$ x k_{s1}, h_{agg} + k_{s2}, 20mm) = 25.0 mm

PASS - Actual bar spacing exceeds minimum allowable

Zone 3 (1723 mm - 2000 mm) Positive moment - section 6.1

Design bending moment; $M = abs\ (M\ m1\_s1\_z3\_max\_red)$ = 27.7 kNm

Effective depth of tension reinforcement; d = 204 mm

Redistribution ratio; δ = min (M $_{pos\_red\_z3\ /\ Mpos\_z3,\ 1}$) = 1.000

K = M / (b x d^2 x f_{ck}) = 0.148

K' = (2 x η x α_{cc} / γ_c) x (1 - λ x (δ - k_1) / (2 x k_2)) x (λ x (δ - k_1) / (2 x k_2)) = 0.207

K' > K - No compression reinforcement is required

Lever arm; z = min (0.5 x d x [1 + (1 - 2 x K / $\eta\ x\ \alpha_{cc}$ / γ_c))$^{0.5}$], 0.95 x d) = 173 mm

Depth of neutral axis; x = 2 x (d - z) / λ = 79 mm

Area of tension reinforcement required; As,req = M / (fyd x z) = 401 mm^2

Tension reinforcement provided; 2 x 16 ϕ

Area of tension reinforcement provided; As,prov = 402 mm^2

Min. Reinforcement area- exp.9.1N; As,min = max(0.26 x fctm / fyk, 0.0013) x b x d = 50 mm^2

Maximum area of reinforcement - cl.9.2.1.1 (3); As,max = 0.04 x b x h = 1500 mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Maximum crack width; wk = 0.4 mm

Design value modulus of elasticity reinf - 3.2.7(4); Es = 200000 N/mm^2

Mean value of concrete tensile strength; fct,eff = fctm = 2.9 N/mm^2

Stress distribution coefficient; kc = 0.4

Non-uniform self-equilibrating stress coefficient; k = min (max (1 + (300 mm – min (h, b)) x 0.35 / 500 mm, 0.65), 1) = 1.00

Actual tension bar spacing; s_{bar} = (b - (2 x (c $_{nom\_s}$ + $\phi_{m1\_s1\_z3\_v}$) + $\phi_{m1\_s1\_z3\_b\_L1}$ x N $_{m1\_s1\_z3\_b\_L1}$)) / (N $_{m1\_s1\_z3\_b\_L1}$ - 1) + $\phi_{m1\_s1\_z3\_b\_L1}$ =58 mm

Maximum stress permitted (Table 7.3N); σ = 360 N/mm^2

Steel to concrete modulus of elast. ratio; α_{cr}= Es / Ecm = 6.09

Distance of the Elastic NA from bottom of beam; y = (b x h^2 / 2 + As,prov x (α_{cr}= - 1) x (h - d)) / (b x h + As,prov x (α_{cr} - 1)) = 121 mm

Area of concrete in the tensile zone; Act = b x y = 18137 mm^2

Min. Reinforcement area required - exp.7.1; Asc,min = kc x k x $f_{ct,eff}$ x Act / σ = 58 mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Permanent load ratio; R_{PL} = 0.65

Service stress in reinforcement; σ_{sr} = fyd x As,req / As,prov x R_{PL} = 260 N/mm^2

Maximum bar spacing - Tables 7.3N; $s_{bar,max}$ = 225.6 mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Zone 3 (1723 mm - 2000 mm) Negative moment - section 6.1

Design bending moment; M = max (β_1 x abs(M $_{m1\_s1\_max\_red}$), abs(M $_{m1\_s1\_z3\_min\_red}$)) = 6.9 kNm

Effective depth of tension reinforcement; d = 207 mm

Redistribution ratio; δ = 1 = 1.000

K = M / (b x d^2 x f_{ck}) =0.036

K' = (2 x η x α_{cc} / γ_c) x (1 - λ x (δ - k_1) / (2 x k_2)) x (λ x (δ - k_1) / (2 x k_2)) == 0.207

K' > K - No compression reinforcement is required

Lever arm; z = min (0.5 x d x [1 + (1 - 2 x K / η x α_{cc} / γ_c))$^{0.5}$], 0.95 x d) =197 mm

Depth of neutral axis; x = 2 x (d - z) / λ = 26 mm

Area of tension reinforcement required; As,req = M / (fyd x z) = 88 mm^2

Tension reinforcement provided; 2 x 10ϕ

Area of tension reinforcement provided; As,prov = 157 mm^2

Minimum area of reinforcement - exp.9.1N; As,min = max(0.26 x fctm / fyk, 0.0013) x b x d = 51 mm^2

Maximum area of reinforcement - cl.9.2.1.1 (3); As,max = 0.04 x b x h = 1500 mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Maximum crack width; wk = 0.4 mm

Design value modulus of elasticity reinf - 3.2.7(4); Es = 200000 N/mm^2

Mean value of concrete tensile strength; fct,eff = fctm = 2.9 N/mm^2

Stress distribution coefficient; kc = 0.4

Non-uniform self-equilibrating stress coefficient; k = min (max (1 + (300 mm – min (h, b)) x 0.35 / 500 mm, 0.65), 1) = 1.00

Actual tension bar spacing; s_{bar} = (b - (2 x (c_{nom_s} + $\phi_{m1\_s1\_z3\_v}$) + $\phi_{m1\_s1\_z3\_t\_L1}$ x $N_{m1\_s1\_z3\_t\_L1}$)) / ($N_{m1\_s1\_z3\_t\_L1}$ - 1) + $\phi_{m1\_s1\_z3\_t\_L1}$ =64 mm

Maximum stress permitted (Table 7.3N); σ_s = 360 N/mm^2

Steel to concrete modulus of elast. ratio; α_{cr}= Es / Ecm = 6.09

Distance of the Elastic NA from bottom of beam; y = (b x h^2 / 2 + As,prov x (α_{cr} - 1) x (h - d)) / (b x h + As,prov x (α_{cr} - 1))= 123 mm

Area of concrete in the tensile zone; Act = b x y = 18493 mm^2

Min. Reinforcement area required - exp.7.1; Asc,min = kc x k x fct,eff x Act / σ_s = 60 mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Permanent load ratio; R_{PL} = 0.65

Service stress in reinforcement; σ_{sr} = fyd x As,req / As,prov x R_{PL} = 146 N/mm^2

Maximum bar spacing - Tables 7.3N; $s_{bar,max}$ = 300 mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing (Section 8.2)

Top bar spacing; s_{top} = (b - (2 x (c_{nom_s} + $\phi_{m1\_s1\_z3\_v}$) + $\phi_{m1\_s1\_z3\_t\_L1}$ x $N_{m1\_s1\_z3\_t\_L1}$)) / ($N_{m1\_s1\_z3\_t\_L1}$ - 1) = 54.0 mm

Minimum allowable top bar spacing; stop,min = max($\phi_{m1\_s1\_z3\_t\_L1}$ x k_{s1}, h_{agg} + k_{s2}, 20mm) = 25.0 mm

PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing; s_{bot} = = (b - (2 x (c_{nom_s} + $\phi_{m1\_s1\_z3\_v}$) + $\phi_{m1\_s1\_z3\_b\_L1}$ x $N_{m1\_s1\_z3\_b\_L1}$)) / ($N_{m1\_s1\_z3\_b\_L1}$ - 1) = 42.0 mm

Minimum allowable bottom bar spacing; sbot,min = max($\phi_{m1\_s1\_z3\_b\_L1}$ x k_{s1}, h_{agg} + k_{s2}, 20mm)

= 25.0 mm

PASS - Actual bar spacing exceeds minimum allowable

Table 3. 4: Moment design reinforcement areas results for 2m span length

	Unit	Provided	Required	Utilisation	Result
Zone 1 top (-6.9 kNm)	mm^2	157	88	0.560	PASS
Zone 1 bot (27.7 kNm)	mm^2	402	401	0.998	PASS
Zone 2 top (0.0 kNm)	mm^2	157	0	0.000	PASS
Zone 2 bot (27.7 kNm)	mm^2	402	401	0.998	PASS
Zone 3 top (-6.9 kNm)	mm^2	157	88	0.560	PASS
Zone 3 bot (27.7 kNm)	mm^2	402	401	0.998	PASS
Span to depth ratio		9.80	15.71	0.624	PASS

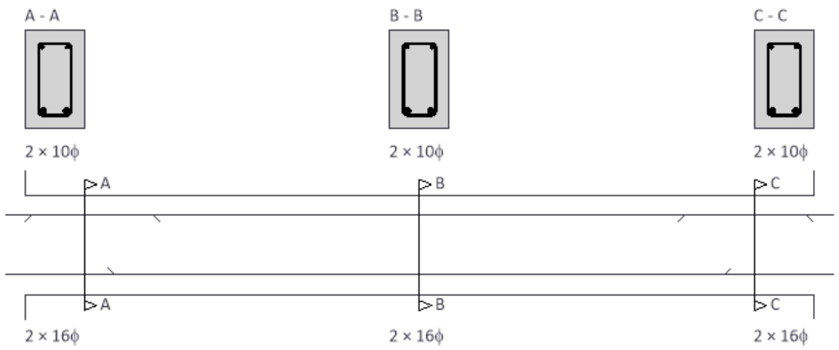


Figure 3. 10: Top and bottom longitudinal reinforcement provision for 2m span length

Shear design

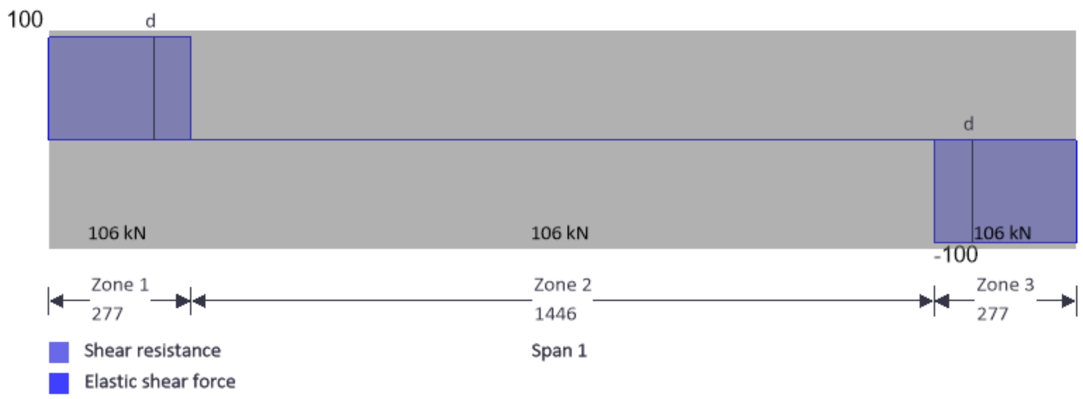


Figure 3. 11: Shear resistance and elastic shear force result for 2m span length

Angle of comp. shear strut for maximum shear; θ_{max}= 45 deg

Strength reduction factor - cl.6.2.3 (3); $v_1 = 0.6 \times (1 - fck / 250\ N/mm^2) = 0.528$

Compression chord coefficient - cl.6.2.3 (3); $\alpha_{cw} = 1.00$

Minimum area of shear reinforcement - exp.9.5N; Asv,min = 0.08 N/mm^2 x b x (fck / 1 N/mm^2)$^{0.5}$ / fyk = 143 mm^2/m

For inclined shear reinforcement, the minimum area of shear reinforcement

Shear reinforcement with 45^0 inclination, $\text{Asv, min} = 0.08\ \text{N}/mm^2 \text{ x b x } \sin(45^o)$ (fck / $1\ \text{N}/mm^2)^{0.5}$/ fyk = 102 mm^2/m

Shear reinforcement with 60^0 inclination, $\text{Asv, min} = 0.08\ \text{N}/mm^2 \text{ x b x } \sin(60^o)$ (fck / $1\ \text{N}/mm^2)^{0.5}$/ fyk = 89 mm^2/m

Shear reinforcement with 75^0 inclination, $\text{Asv, min} = 0.08\ \text{N}/mm^2 \text{ x b x } \sin(75^o)$ (fck / $1\ \text{N}/mm^2)^{0.5}$/ fyk = 86 mm^2/m

Zone 1 (0 mm - 277mm) shear - section 6.2

Design shear force at support; $V_{Ed,max} = \max(\text{abs}(V_{z1\_max}), \text{abs}(V_{z1\_red\_max})) = 100$ kN

Min lever arm in shear zone; z = 173 mm

Maximum design shear resistance - exp.6.9; $V_{Rd,max} = \alpha_{cw}$ x b x z x v_1 x fcwd / (cot(θmax) + tan(θmax)) = 137 kN

PASS - Design shear force at support is less than maximum design shear resistance

The shear resistance for the members with inclined shear reinforcement

$$V_{Rd,max} = \alpha_{cw} b_w z v_1 f_{cwd} (\cot\theta + \cot\alpha)/(1 + \cot^2\theta)$$

Shear reinforcement with 45^0 inclination, $V_{Rd,max} = 280$ KN

Shear reinforcement with 60^0 inclination, $V_{Rd,max} = 221$ KN

Shear reinforcement with 75^0 inclination, $V_{Rd,max} = 178$ KN

Design shear force at support is less than maximum design shear resistance in the all cases of inclined shear reinforcements.

Design shear force at 204mm from support; $V_{Ed} = 100$ kN

Design shear stress; $v_{Ed} = V_{Ed} / (b \times z) = 3.854\ N/mm^2$

Angle of concrete compression strut - cl.6.2.3; θ = min (max (0.5 x Asin (min (2 x v_{Ed} / (α_{cw} x f_{cwd} x v_1), 1)), 21.8 deg), 45deg) = 21.8 deg

Area of shear reinforcement required - exp.6.8; Asv,des = v_{Ed} x b / (fyd x cot(θ)) = 578 mm^2/m

Area of shear reinforcement required; Asv,req = max(Asv,min, Asv,des) = 578 mm^2/m

Shear reinforcement provided; 2 x 8 legs @ 150 c/c

Area of shear reinforcement provided; Asv,prov = 670 mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N; svl,max = 0.75 x d = 153 mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Shear reinforcements with different angle of inclination

Area of inclined shear reinforcement: $\mathrm{Asv, des} \ = \frac{v_{Ed}\, b_w \sin(\alpha)}{f_{yd} \cot\theta}$

Shear reinforcement with 45^0 inclination, $\mathrm{Asv, des} \ = 321$ mm^2/m

Area of shear reinforcement required; Asv,req = max(Asv,min , Asv,des) = 321 mm^2/m

Shear reinforcement provided; 2 x 8 legs @ 300 c/c

Area of shear reinforcement provided; Asv,prov = 335 mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N; svl,max = 0.75 x d (1 + cot α) =306 mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Shear reinforcement with 60^0 inclination, $\mathrm{Asv, des} \ = 394$ mm^2/m

Area of shear reinforcement required; Asv,req = max(Asv,min , Asv,des) = 394 mm^2/m

Shear reinforcement provided; 2 x 8 legs @ 225 c/c

Area of shear reinforcement provided; Asv,prov = 447 mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N; svl,max = 0.75 x d (1 + cot α) =242 mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Shear reinforcement with 75^0 inclination, A_{sw} =439 mm^2/m

Area of shear reinforcement required; Asv,req = max(Asv,min , Asv,des) = 439 mm^2/m

Shear reinforcement provided; 2 x 8 legs @ 175 c/c

Area of shear reinforcement provided; Asv,prov = 574 mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N; svl,max = 0.75 x d (1 + cot α) = 194 mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Zone 2 (277 mm - 1723 mm) shear - section 6.2

Design shear force at support; $V_{Ed,max}$ = max(abs(V_{z2_max}), abs($V_{z2\_red\_max}$)) = 100 kN

Min lever arm in shear zone; z = 173 mm

Maximum design shear resistance - exp.6.9; $V_{Rd,max}$ =α_{cw} x b x z x v_1 x fcwd / (cot(θmax) + tan(θmax)) =137 kN

PASS - Design shear force at support is less than maximum design shear resistance

The shear resistance for the members with inclined shear reinforcement

$$V_{Rd,max} = \alpha_{cw} b_w z v_1 f_{cwd} (\cot\theta + \cot\alpha)/(1 + \cot^2\theta)$$

Shear reinforcement with 45^0 inclination, $V_{Rd,max} = 280$ KN

Shear reinforcement with 60^0 inclination, $V_{Rd,max} = 221$ KN

Shear reinforcement with 75^0 inclination, $V_{Rd,max} = 178$ KN

Design shear force at support is less than maximum design shear resistance in the all cases of inclined shear reinforcements.

Design shear force within zone; V_{Ed} = 100 kN

Design shear stress; $v_{Ed} = V_{Ed}$ / (b x z) = 3.854 N/mm^2

Angle of concrete compression strut - cl.6.2.3; θ = min (max (0.5 x Asin(min(2 x v_{Ed} / (α_{cw} x b x z x v_1),1)), 21.8 deg), 45deg) = 21.8 deg

Area of shear reinforcement required - exp.6.8; Asv,des = v_{Ed} x b / (fyd x cot(θ)) = 578 mm^2/m

Area of shear reinforcement required;Asv,req = max(Asv,min, Asv,des) = 578 mm^2/m

Shear reinforcement provided; 2 x 8 legs @ 150 c/c

Area of shear reinforcement provided; Asv,prov = 670 mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N; $s_{vl,max}$ = 0.75 x d = 153 mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Shear reinforcements with different angle of inclination

Area of inclined shear reinforcement: $\mathrm{Asv,des} = \frac{v_{Ed}\, b_w \sin(\alpha)}{f_{yd} \cot\theta}$

Shear reinforcement with 45^0 inclination, $\mathrm{Asv,des} = 321$ mm^2/m

Area of shear reinforcement required;Asv,req = max(Asv,min , Asv,des) = 321 mm^2/m

Shear reinforcement provided; 2 x 8 legs @ 300 c/c

Area of shear reinforcement provided; Asv,prov = 335 mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N; svl,max = 0.75 x d (1 + cot α) =306 mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Shear reinforcement with 60^0 inclination, $\mathrm{Asv,des} = 394$ mm^2/m

Area of shear reinforcement required;Asv,req = max(Asv,min , Asv,des) = 394 mm^2/m

Shear reinforcement provided; 2 x 8 legs @ 225 c/c

Area of shear reinforcement provided; Asv,prov = 447 mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N; svl,max = 0.75 x d (1 + cot α) =242 mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Shear reinforcement with 75^0 inclination, A_{sw} =439 mm^2/m

Area of shear reinforcement required; Asv,req = max(Asv,min , Asv,des) = 439 mm^2/m

Shear reinforcement provided; 2 x 8 legs @ 175 c/c

Area of shear reinforcement provided; Asv,prov = 574 mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N; svl,max = 0.75 x d (1 + cot α) = 194 mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Zone 3 (1723 mm - 2000 mm) shear - section 6.2

Design shear force at support; $V_{Ed,max}$ = max(abs(V_{z3_max}), abs($V_{z3\_red\_max}$)) = 100 kN

Min lever arm in shear zone; z = 173 mm

Maximum design shear resistance - exp.6.9; $V_{Rd,max}$ =α_{cw} x b x z x v_1 x fcwd / (cot(θmax) + tan(θmax)) = 137 kN

PASS - Design shear force at support is less than maximum design shear resistance

The shear resistance for the members with inclined shear reinforcement

$$V_{Rd,max} = \alpha_{cw} b_w z v_1 f_{cwd} (\cot\theta + \cot\alpha)/(1 + \cot^2\theta)$$

Shear reinforcement with 45^0 inclination, $V_{Rd,max}$ = 274 KN

Shear reinforcement with 60^0 inclination, $V_{Rd,max}$ = 216 KN

Shear reinforcement with 75^0 inclination, $V_{Rd,max}$ = 174 KN

Design shear force at support is less than maximum design shear resistance in the all cases of inclined shear reinforcements.

Design shear force at 204mm from support; V_{Ed} = 100 kN

Design shear stress; v_{Ed} = V_{Ed} / (b x z) = 3.854 N/mm^2

Angle of concrete compression strut - cl.6.2.3; θ = min (max (0.5 x Asin(min(2 x v_{Ed} / (α_{cw} x b x z x v_1),1)), 21.8 deg), 45deg) =21.8 deg

Area of shear reinforcement required - exp.6.8; Asv,des = v_{Ed} x b / (fyd x cot(θ)) = 578 mm^2/m

Area of shear reinforcement required; Asv,req = max(Asv,min, Asv,des) = 578 mm^2/m

Shear reinforcement provided; 2 x 8 legs @ 150 c/c

Area of shear reinforcement provided; Asv,prov = 670 mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N; $s_{vl,max}$ = 0.75 x d = 153 mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximu

Shear reinforcements with different angle of inclination

For calculate area of inclined shear reinforcement: $\mathrm{Asv,des} = \frac{v_{Ed}\, b_w \sin(\alpha)}{f_{yd} \cot\theta}$

Shear reinforcement with 45^0 inclination, $\mathrm{Asv,des} = 410$ mm^2/m

Area of shear reinforcement required; Asv,req = max(Asv,min , Asv,des) = 410 mm^2/m

Shear reinforcement provided; 2 x 8 legs @ 300 c/c

Area of shear reinforcement provided; Asv,prov = 335 mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N; svl,max = 0.75 x d (1 + cot α) =306 mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Shear reinforcement with 60^0 inclination, $\mathrm{Asv,des} = 503$ mm^2/m

Area of shear reinforcement required;Asv,req = max(Asv,min , Asv,des) = 503 mm^2/m

Shear reinforcement provided; 2 x 8 legs @ 225 c/c

Area of shear reinforcement provided; Asv,prov = 447 mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N; svl,max = 0.75 x d (1 + cot α) =242 mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Shear reinforcement with 75^0 inclination, A_{sw} =561 mm^2/m

Area of shear reinforcement required; Asv,req = max(Asv,min , Asv,des) = 561 mm^2/m

Shear reinforcement provided; 2 x 8 legs @ 175 c/c

Area of shear reinforcement provided; Asv,prov = 574 mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N; svl,max = 0.75 x d (1 + cot α) = 194 mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Table 3. 5: Shear design reinforcement areas results for 2m span length

	Unit	Provided	Required	Utilisation	Result
Zone 1 (100.0 kN)	mm^2/m	670	631	0.941	PASS
Zone 2 (100.0 kN)	mm^2/m	670	631	0.941	PASS
Zone 3 (100.0 kN)	mm^2/m	670	631	0.941	PASS

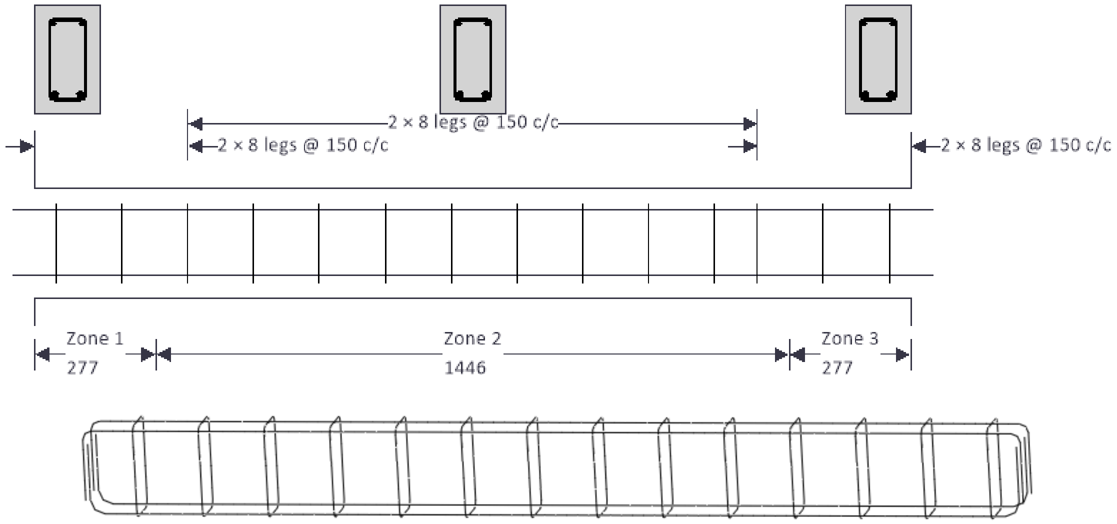


Figure 3. 12: Vertical stirrups provision in 2m span length of RC beam

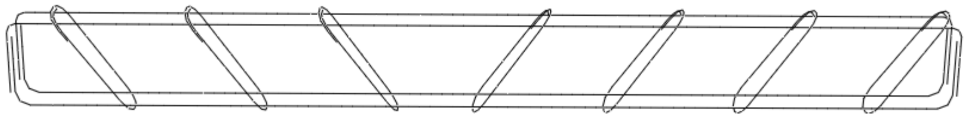


Figure 3. 13: 45^0 inclined Stirrups provision in 2m span length of RC beam

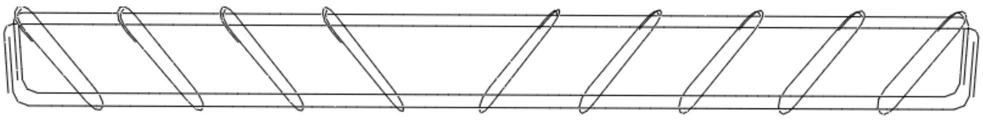


Figure 3. 14: 60^0 inclined Stirrups provision in 2m span length of RC beam

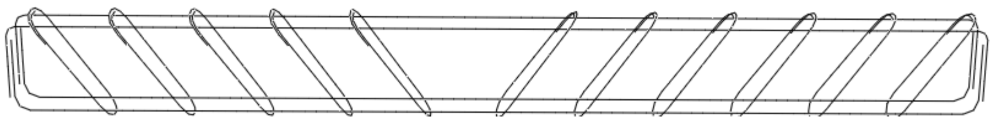


Figure 3. 15: 75^0 inclined Stirrups provision in 2m span length of RC beam

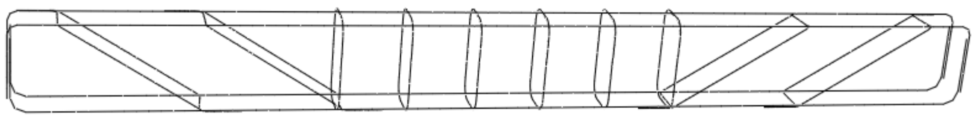


Figure 3. 16: 45^0 Swimmer bars provision in 2m span length of RC beam

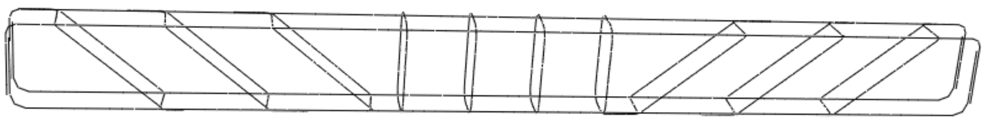


Figure 3. 17: 60^0 Swimmer bars provision in 2m span length of RC beam

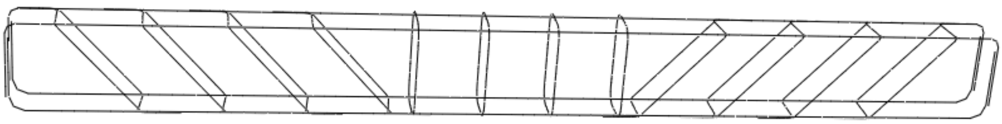


Figure 3. 18: 75^0 Swimmer bars provision in 2m span length of RC beam

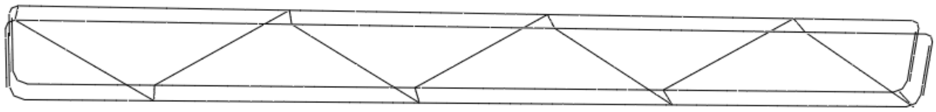


Figure 3. 19: 45^0 Rectangular spiral bars provision in 2m span length of RC beam

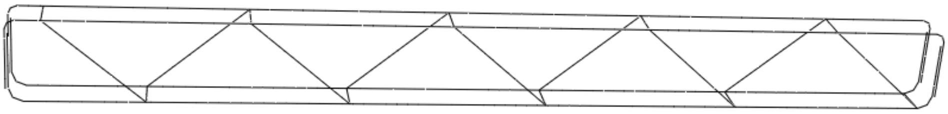


Figure 3. 20: 60^0 Rectangular spiral bars provision in 2m span length of RC beam

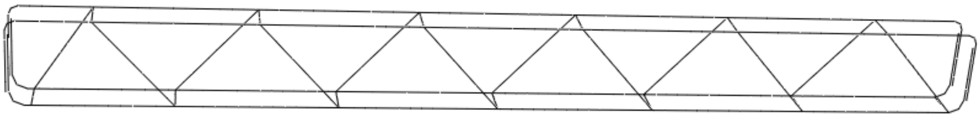


Figure 3. 21: 75^0 Rectangular spiral bars provision in 2m span length of RC beam

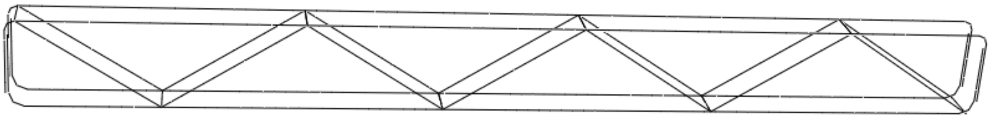


Figure 3. 22: 45^0 Warren truss reinforcement's provision in 2m span length of RC beam

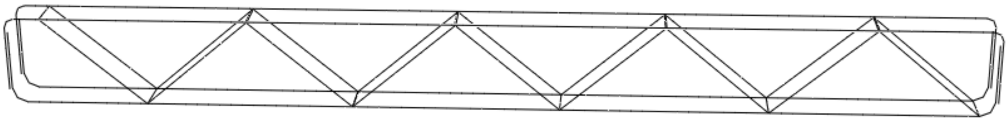


Figure 3. 23: 60^0 Warren truss reinforcement's provision in 2m span length of RC beam

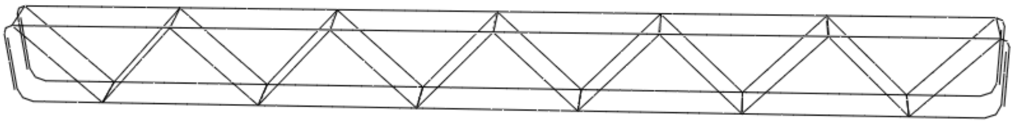


Figure 3. 24: 75^0 Warren truss reinforcement's provision in 2m span length of RC beam

Reinforced concrete beams design to be use in finite element analysis with the span length of 4m and 6m summarized in appendix A.

3.5 Finite Element Model

This study presents a modeling of reinforced concrete beams with different shapes of shear reinforcement to show their effects in reinforced concrete beams considering the shear strength, load-deflection of reinforced concrete beam under monotonic loadings. The finite element analysis of the concrete as a non-linear material can be a useful way of modeling the concrete. Many Finite element packages are commercially available. However, to obtain an efficient and accurate finite element method, the analysis conducted using nonlinear Finite element analysis software, which called ABAQUS 6.14-5.

A non-linear analysis of reinforced concrete beams in ABAQUS software operated using two options a smeared crack concrete model and a concrete damaged plasticity model. Both models are providing a general capability for modeling concrete in all types of structures, even though it is planning primarily for the analysis of reinforced concrete structures, they can use it with rebar to model concrete reinforcement; and both use for plain concrete.

For this particular design of analysis modeling using ABAQUS software, a concrete damaged CDP) model used for the determination of the effectiveness of shear reinforcements shapes in reinforced concrete beams. Concrete damaged plasticity model in ABAQUS/Explicit provides the ability to model the behavior of plain or reinforced concrete elements subjected to both static and dynamic loads.

Since FE, commercial software can be highly demanding in computational time, two symmetry planes take into consideration. The first plane place in the center of the beam along its width. For this symmetry plane, a constraint along the x-axis is considering to obey the same principles as the actual beam. Moreover, a second plane considering along the length and the translation along the z-axis is constraining. Incorporate reduced integration with the first-order 3D, 8-node solids element used for steel and concrete to overcome the possible errors and to consider the cracks in tension.

Table 3. 6: Reinforced concrete beams designed summary

NO.	Span length (m)	W x D (mm)	RC beams with shear reinforcement angles	Max. Stirrups c/c Spacing (mm)	Max. No. of Top 10mm dia Long. Bars	Max. No. of Bottom 16mm dia Long. Bars
1	**2**	**150x250**	**RCB-IS (Validation)**	**75 and 150**	**2**	**2**
2	**2**	**150x250**	**RCB-VS**	**150**	**2**	**2**
3	2	150x250	RCB-1	300	2	2
4	2	150x250	RCB-2	225	2	2
5	2	150x250	RCB-3	175	2	2
6	**4**	**200x350**	**RCB-VS**	**225**	**2**	**3**
7	4	200x350	RCB-1	450	2	3
8	4	200x350	RCB-2	350	2	3
9	4	200x350	RCB-3	275	2	3
10	**6**	**250x500**	**RCB-VS**	**325**	**3**	**3**
11	6	250x500	RCB-1	575	3	3
12	6	250x500	RCB-2	525	3	3
13	6	250x500	RCB-3	425	3	3

3.5.1 Geometry

Analysis performed to model the non-linear behavior of the beams with geometry and reinforcement. In this study rectangular cross-section of RC beams was modeled using 3D, deformable and extrusion method. For all concrete beams, the hanging region width provide to gives sufficient bond length to prevent deboning according to the design codes. Partition was creating on the surface of the RC beams in order to keep the compatibility between steel plate and concrete. This enables to refine the mesh and get better result. The two-point loads applied symmetrically in the RC beams to ensure the shear cracks occur near the support. A sketch of the concrete and steel section created separately with ABAQUS software, which extrude in any direction.

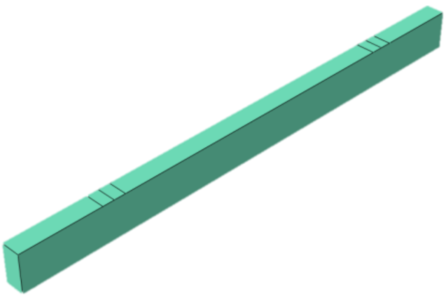


Figure 3. 25: 3D Plain concrete used in 6m Span length

3.5.2 Longitudinal and Shear Reinforcements

The longitudinal and shear reinforcements modeled using 3D, deformable and wire methods.

Figure 3. 26: Top and bottom longitudinal reinforcements used in ABAQUS Analysis

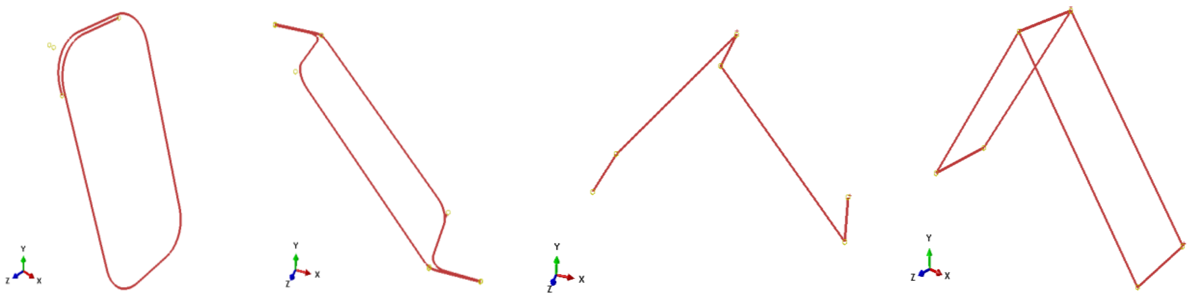


Figure 3. 27: Shear reinforcement shapes used in ABAQUS Analysis

3.5.3 Steel Plate

A steel loading plate and a support plate was tie-up with the concrete beam to remove the stress concentrations around the points of loading and support. The RC beams analyzed under a four-point loading arrangement by providing pinned support on both ends. In the software, these support conditions were model as discrete rigid of steel plates and the actual constraints inserted at the middle of steel plates. The steel plates used as support condition at the bottom and distribute the load to the concrete surface at the top surface of the concrete beam. The dimension used in this study different along the different span lengths. They are modeled using 3D, discrete rigid and extrusion method.

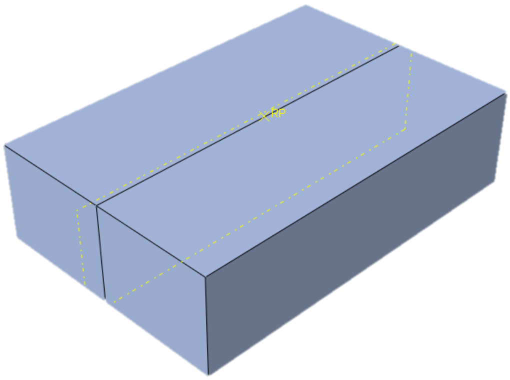


Figure 3. 28: Steel plate for load support

3.6 Material properties

3.6.1 Concrete

The details of the RC beams design to achieve a concrete compressive strength of 30/37 N/mm^2, which has taken from the experimental validation.

Table 3. 7: Concrete material properties for C30/37 [5]

f_{ck} Mpa	$f_{ck,cube}$ Mpa	f_{cm} Mpa	f_{ctm} Mpa	E_{cm} Gpa	Density (kg/m3)	Poisson's ratio
30	37	38	2.896468154	32.83656803	2400	0.2

Stress-strain curve for uniaxial compression

The relation between stress-strain for short-term uniaxial loading described by the Expression

$$\frac{\sigma_c}{f_{cm}} = \frac{k\eta-\eta}{1+(k-2)\eta} \tag{3.42}$$

Where $\eta = \varepsilon_c/\varepsilon_{c1}$ (3.43)

The constant $k = 1.05E_{cm}x|\varepsilon_{c1}|/f_{cm}(f_{cm}\ according\ to\ Table\ 3.1\ in\ EC2)$ (3.44)

The above expression is valid for $0 < |\varepsilon_{c1}| < |\varepsilon_{cu1}|$ where, ε_{cu1} is the nominal ultimate strain.

In order to begin an analysis of the stress-strain curve is the longitudinal modulus of elasticity (Ecm) of the concrete. Its value can calculate using

$$E_{cm} = 22(0.1f_{cm})^{0.3} \tag{3.45}$$

Where, Mean Concrete Compressive Strength, $f_{cm} = f_{ck} + 8(Mpa)$ (3.46)

f_{ck} Characteristic cylinder strength

The strain at peak stress according to Table 3.1 EC2

$$\varepsilon_{c1}(\%) = 0.7(f_{cm})^{0.31} \leq 2.8\% \tag{3.47}$$

The nominal ultimate strain ε_{cu1}

For $f_{ck} \geq 50Mpa$, $\varepsilon_{cu1}(\%) = 2.8 + 27[(98 - f_{cm})/100]^4 \leq 3.5\%$ (3.48)

The compressive stress strain of concrete data generated using EC2 as shown in Figure below

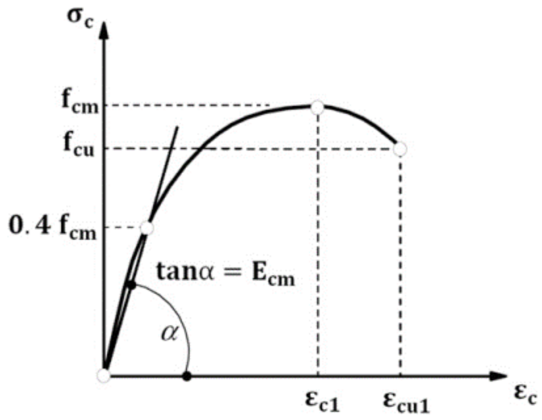


Figure 3. 29: Stress-strain diagram of concrete in compression [5]

Using the above equations were applied to generate the Stress-strain diagram of concrete in compression data shown in the figure below

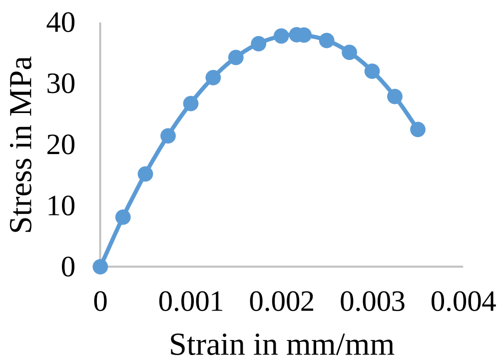


Figure 3. 30: Stress Strain Curve for concrete in compression material model

Concrete damage plasticity used in this research to model concrete plasticity parameters used for the determination of biaxial failure (yield) surface. The default parameters used when concrete modeling are

- Dilation angle- A parametric study carried out by Malm [22] suggested that there is no significant difference between 20° and 40° dilation angle if a reinforced concrete beam is subjected to bending. The best agreement with experimental data was reached between 30° and 40°,
- The flow potential eccentricity which is a small positive number, defines the rate at which the hyperbolic flow potential approaches its asymptote [7]
- The biaxial stress ratio and the tensile-to compressive meridian ratio or the ratio of second stress invariant were assumed to be equal to 1.16 and 0.667, respectively, based on recommendations of Chen and Ha [23]
- **fbo/fco** the ratio of bi-axial compressive stress to initial uniaxial compressive stress and
- viscous parameter μ=0.

Table 3. 8: Concrete damage parameters used in the models

Dilation Angle	**Eccentricity**	**fbo/fco**	**K**	**Viscosity Parameter** μ
36	0.1	1.16	0.667	0

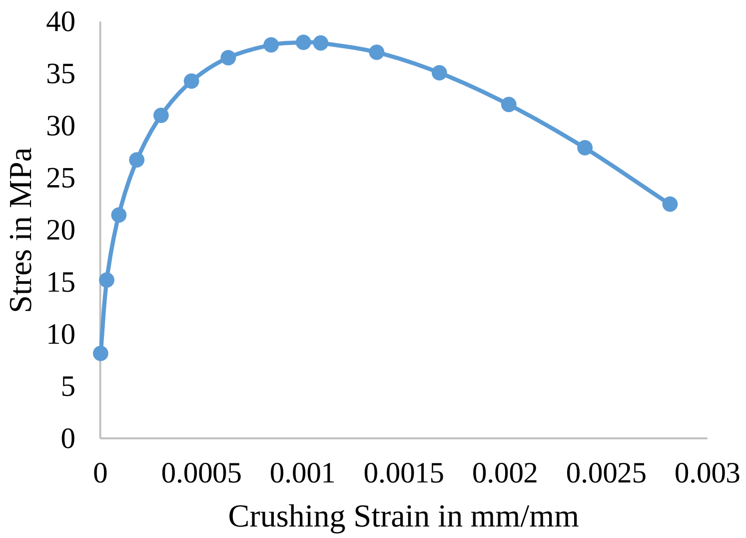


Figure 3. 31: Compressive stress-crushing strain diagram of concrete Steel

3.6.2 Compressive damage variables

The damage variables were prepared based on Alfarah B., et al. proposed methodology and equation. This equation described as follows: [20]

$$d_c = 1 - \frac{1}{2+a_c}[2(1+a_c)exp(-b_c\varepsilon_c{}^{ch}) - a_c exp(-2b_c\varepsilon_c{}^{ch})] \quad (3.49)$$

Where $\varepsilon_c{}^{ch}$ is compressive crushing strain (inelastic strain)

$$a_c = 7.873, b_c = \frac{1.97(f_{ck}+8)l_{eq}}{G_{ch}} \quad (3.50)$$

Where l_{eq} is the characteristic length of the element.

f_{ck} is cylindrical compressive strength of concrete.

G_{ch} is crushing energies, GF is fracture energies

$$G_{ch} = \left(\frac{f_{cm}}{f_{tm}}\right)^2 G_F \quad (3.51)$$

$$G_F(N/mm) = 0.073\, f_{cm}{}^{0.18}, f_{cm}(Mpa) \quad (3.52)$$

Using this equation damage variables compressive damage variable-crushing strain shown below but the table presented in appendix A.

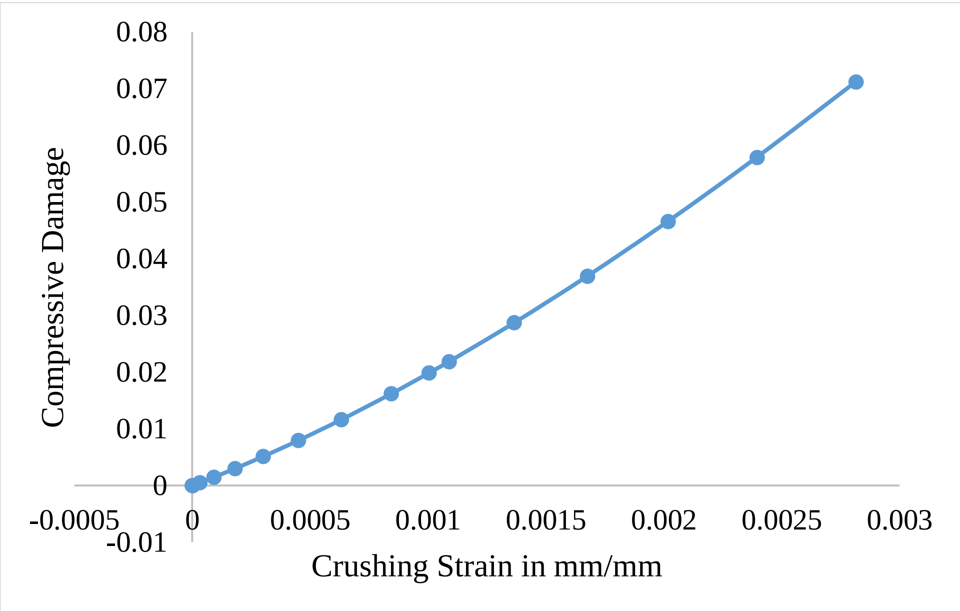


Figure 3. 32: Compressive damage-crushing strain diagram of concrete.

Stress-strain curve for uniaxial tension

The tensile strength of concrete under uniaxial stress is seldom determined through a direct tension test because of the difficulties involved in its execution and the large scatter of the results. Indirect methods, such as sample splitting or beam bending, tend to used (EC 2, 2004).

Mean Concrete tensile strength, $f_{ctm} = 0.30 x f_{ck}{}^{(2/3)} \leq C50/60$ (3.53)

3.6.3 Tensile damage variables

The tensile damage variables were also prepared based on Alfarah B. proposed methodology and equation. This equation described as follows: [20]

$$d_t = 1 - \frac{1}{2+a_t}[2(1+a_t)exp(-b_t \varepsilon_t{}^{ck}) - a_t exp(-2b_t \varepsilon_t{}^{ck})] \tag{3.54}$$

Where $\varepsilon_c{}^{ck}$ is tensile crushing strain (inelastic strain).

$$a_t = 1, b_c = \frac{0.453(f_{ck})^{\frac{2}{3}} l_{eq}}{G_F} \tag{3.55}$$

Using this equation damage variables tensile damage variable-crushing strain shown below

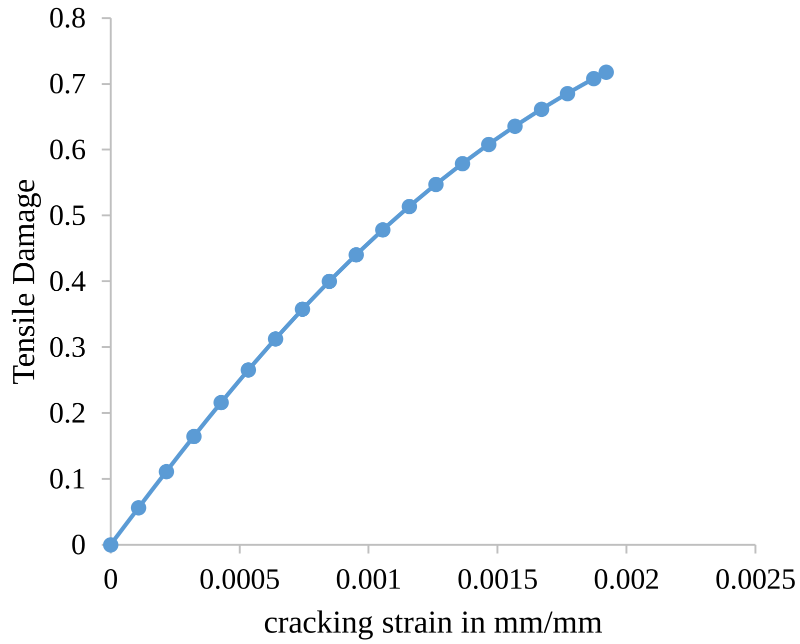


Figure 3. 33: Tensile damage variables-cracking strain diagram of concrete.

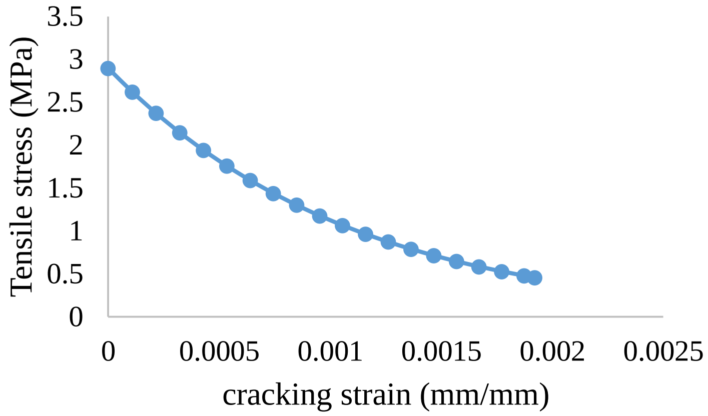


Figure 3. 34: Tensile stress-cracking strain curve for concrete in tension material model

3.6.4 Steel

The longitudinal reinforcements with 10mm and 16mm, and the shear links with 8mm used in the models. The elastic behavior of the RC beam was model by considering linear elasticity with a constant material of Young modulus of elasticity and Poisson ratio according to EC2. The steel property also taken from the previous study of claeson [21]. Deformed bars of Swedish type Ks40S (466MPa) used as the lateral and longitudinal reinforcement. Both values of parameters present in the Table 3.9.

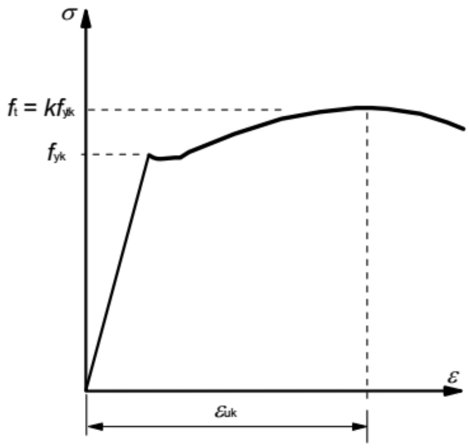


Figure 3. 35: shows stress-strain curves for typical hot rolled [5]

Table 3. 9: Steel properties used in the models

PROPERTIES	STEEL
Density(kg/m3)	7850 kg/m3
Modulus of elasticity	200 GPa
Poisson's ratio	0.3

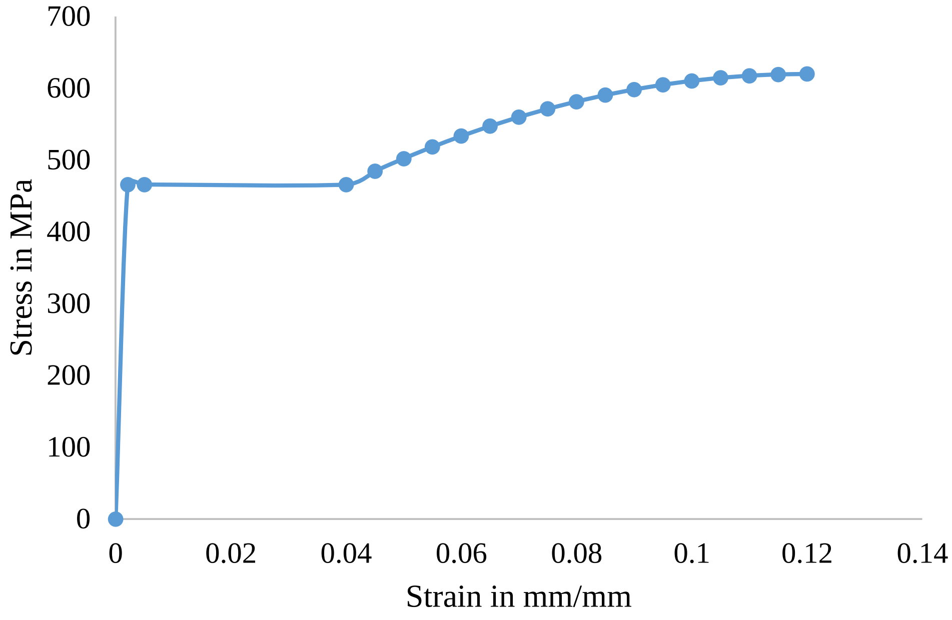


Figure 3. 36: Stress–strain curves for reinforcements in the models

Using nominal true stress of stress-strain, and logarithmic plastic strain computed as shown

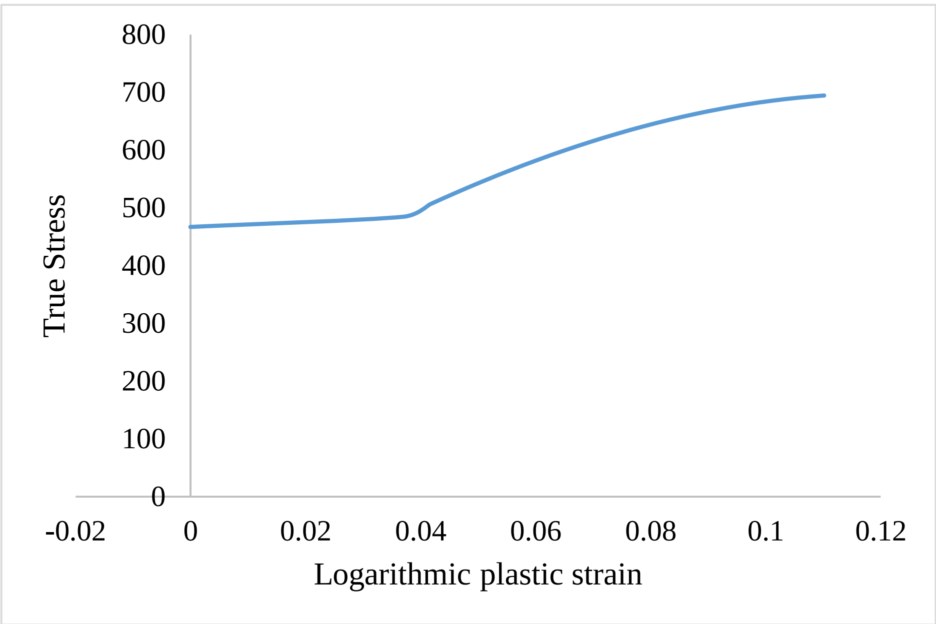


Figure 3. 37: True stress-plastic strain of steel reinforcements.

3.7 Analysis Step

In addition to initial step, only one-step created. In the created step, static standard method selected. Static standard method includes material nonlinearity and geometrical nonlinearity. It is capable of analyzing post-buckling analysis.

3.8 Meshing

3.8.1 Element type

The type of an element characterized by several parameters such as degree of freedom, number of nodes, formulation, and integration. Using ABAQUS software, the user can choose the type of integration to perform on the elements. Hence increasing the number of elements in a Finite element model will increase accuracy but at the same point it will take more time to solve the equations. A solid element such C3D8 is an 8-node continuum element with eight integration points refers to full integration option. While a C3D8R is the same element with the exception of having only one reduced integration, point (reduced integration option) [7]. For this study, C3D8R used for Plain concrete beam and steel plate part and T3D2, which is first order three dimensional truss element used for lateral reinforcement.

3.8.2 Mesh size

The finite element analysis requires meshing of the model. Meshing plays a vital role in the FEA since the properties and governing relationships are assumed over the discretized elements and expressed mathematically on the specified points called nodes. A mesh part generated by define nodes and connecting them to define the element. A convergence of results obtained when an adequate number of elements used in a model. An important step in finite element modelling is the selection of the mesh density. For this particular research, the models meshed with fine size of elements along the span length of 2m, 4m and 6m. After assigning the properties, assembling and meshing an input file was prepared. Finally, the data checked and submitted to get visualize analytically results. The meshed models with solid C3D8 element shown in figure blow.

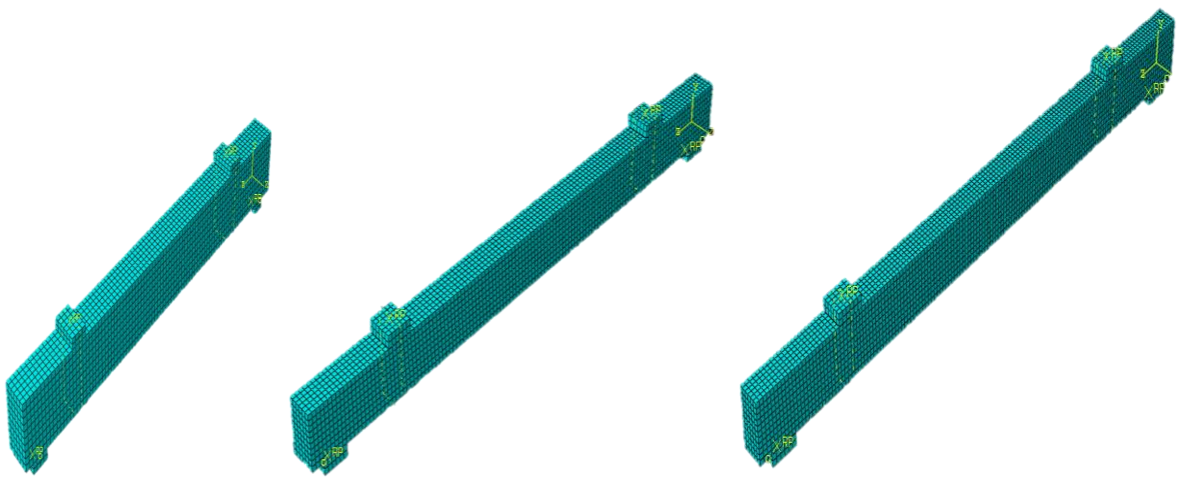


Figure 3. 38: Meshing in 2m, 4m and 6m span length of RC beam models

3.9 Finite Element Model Validation

For validation, an RC beam with 45^0 inclined shear reinforcement was model in ABAQUS 6.14-5 Software based on published experimentally tested RC beam in Malaysian Journal of Civil Engineering in 2018 at the Universiti Teknologi Malaysia by Roslli Noor Mohamed [19].

The finite element model created using ABAQUS software based on the parameters and conditions that used in the laboratory test. The RC beams are analyzing under a four-point loading arrangement by providing pinned support on both ends. Using the ABAQUS software, the support conditions model as steel plate and the material properties used same as the reinforcement considered. The actual constraints are inserting along the beam's width on a line place in the middle of steel plates. The two-point loads are applying symmetrically in the RC beams, and a mesh part is generated by define nodes and connecting them to define the element. A sketch of the concrete and steel section is creating separately with ABAQUS software.

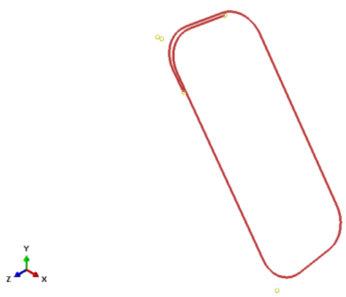


Figure 3. 39: Inclined shear reinforcement used in validation model

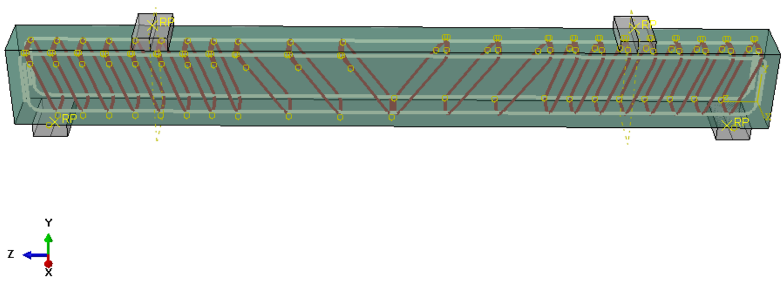


Figure 3. 40: Assembly of experimental validation model with inclined links

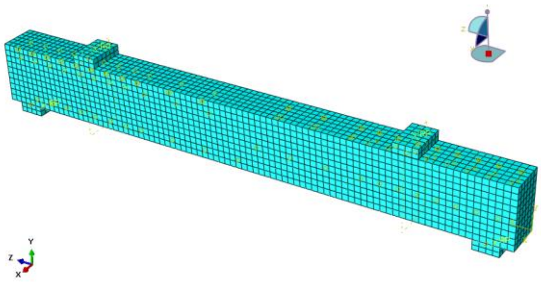


Figure 3. 41: Meshing used in experimental validation model

3.10 Sources of Data

EN 1992-1-1 code [5] used as a primary source of data for considering the material properties and design of the beams. Tekla Tedds 2019 User guide [6] used for the design of reinforced concrete beams, ABAQUS theory and documentation manual [7,17,18] used to analyze the shear behavior of reinforced concrete beams, the experimental works [19] related to this study that previously published, and other textbooks like Mechanics and Design [1] used as a reference.

3.11 Data Presentation and Analysis

Data presentation tools used in this study are attribute and variable data. The shear reinforcements that placed in reinforced concrete beams grouped into categories based on the shapes and angles as attribute data. The shear resistance and load- deflection of reinforced concrete beams data also presented based on some continuous scale measurement when subject to monotonic loading condition as a variable data.

The results obtained presented using the two different approaches, by considering the numerical calculations that followed EN1992-1-1 [5] code and involving a Finite Element Analysis created in ABAQUS software. On the basis of the central objectives of this research, three dimensional Finite Element models of reinforced concrete beam were developed in the analysis, and the various items concerned with modeling are addressed by defining the:

- Elements type
- Material property
- Assigning sections
- Step
- Interaction between elements
- Boundary conditions and load
- Meshing
- Assigning job
- Evaluating the results

The comparative analysis results of the shear reinforcements shapes with different angle of inclination presented using tables, graphs, and charts. The results from these allow us to make a comparison easily by showing which the reinforced concrete beams have the most efficient shapes of shear reinforcement regarding the shear resistance and minimum deflection. The findings of research data present for the civil and environmental engineering department in the presence of examiners.

CHAPTER FOUR

RESULT AND DISCUSSION

4.1 General

This chapter presents the results obtained from finite element analysis method using ABAQUS 6.14-5 software. The obtained results discussed using each independent variable by comparing their load-deflection values obtained from software's using diagrams, tables, and graphs with excel Spreadsheets.

4.2 Load-Deflection Behavior of RC Beam validation model

The finite element model and the experimental study compared for analysis validation is done. The model validation done by the comparison of load-deflection data obtained from the ABAQUS job visualization results. From the comparison between the model and test results, there was good agreement in load-deflection responses over their entire loading profiles until failure. The load-deflection curve for both experimental and finite element analysis is present in the figure below.

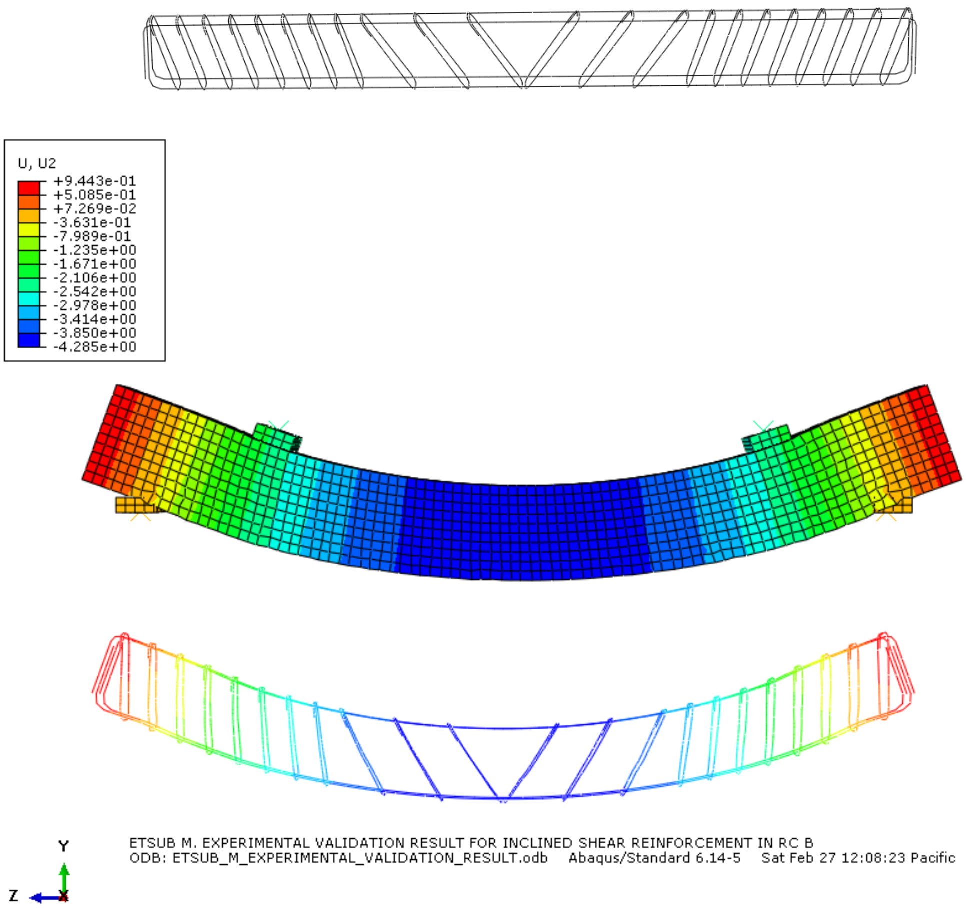


Figure 4. 1: Deflection of experimental validation model at ultimate load applied

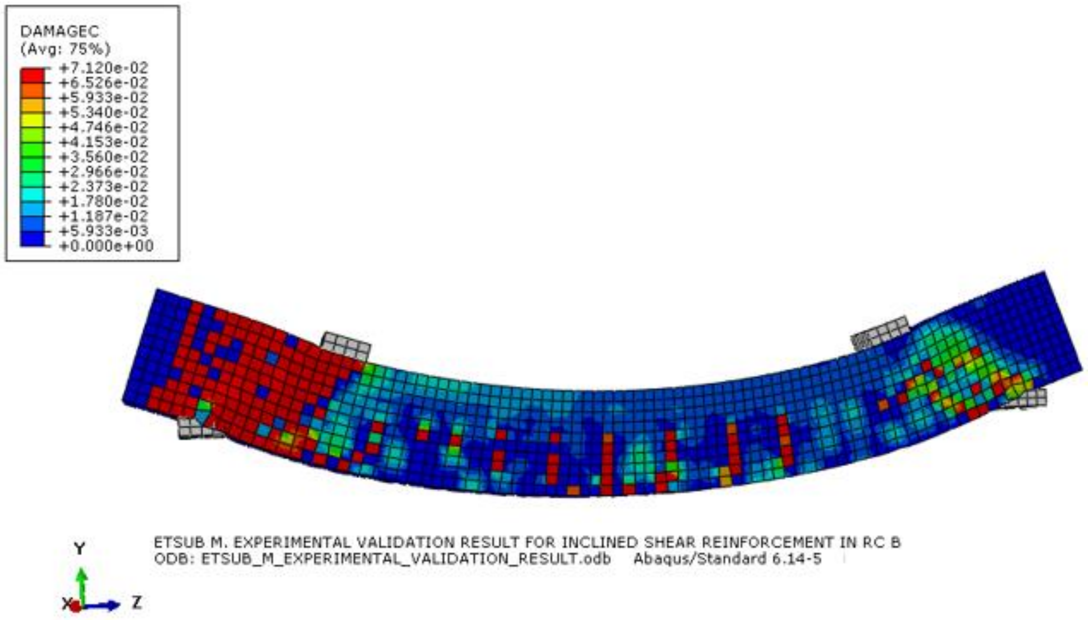


Figure 4. 2: Damage shear crack of validation model

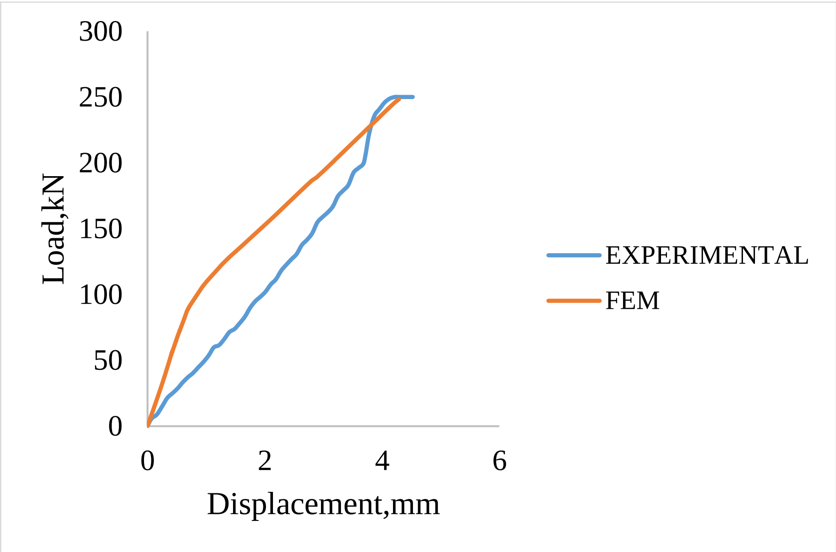


Figure 4. 3: Comparison of FE load-displacement curve with experimental

Maximum shear capacity of reinforced concrete beam from the experimental and ABAQUS result is 250kN and 248 KN respectively. Generally, the comparison between the experimental and ABAQUS results showed a good agreement in shear capacity between experimental and finite element analysis as summarized in the table below

Table 4. 1: Comparison between experimental and FEA for load-deflection value

	Experimental Result	**ABAQUS Result**
Ultimate Load (KN)	250	248
Maximum Deflection (mm)	4.52	4.28

From the table 4.1 the percentage difference in ultimate load carrying capacity between Finite element model and previous experimental study is 0.8|%, which indicates the Finite Element Method can be used as alternative method of analysis for shear reinforcement shapes in reinforced concrete beams.

4.3 RC beams shear strength Results for the Case Study

The shear strength of RC beams obtained from Tekla Tedds calculation software using 45^0,60^0, 75^0, and 90^o shear reinforcement angles based on the EC2 design as shown in the figure below

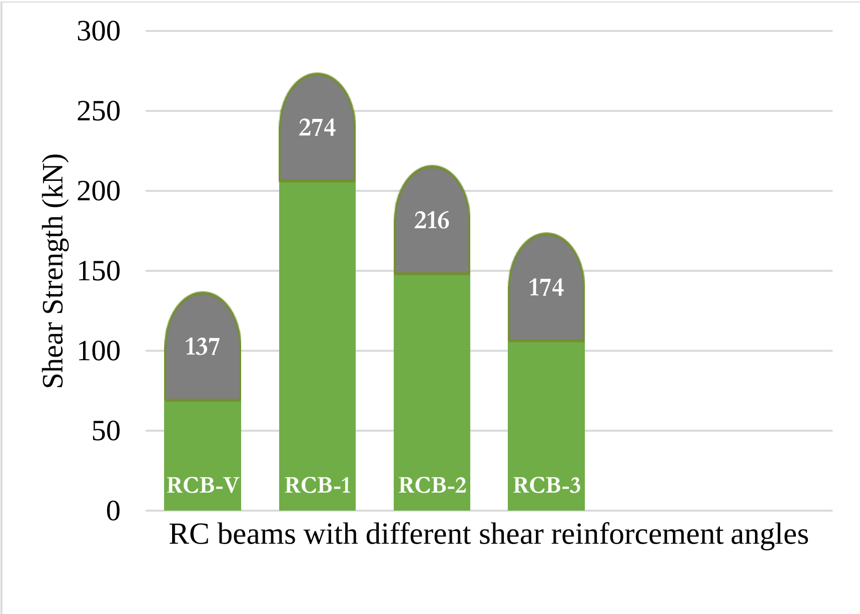


Figure 4. 4: Shear strength of RC beams in 2m span length

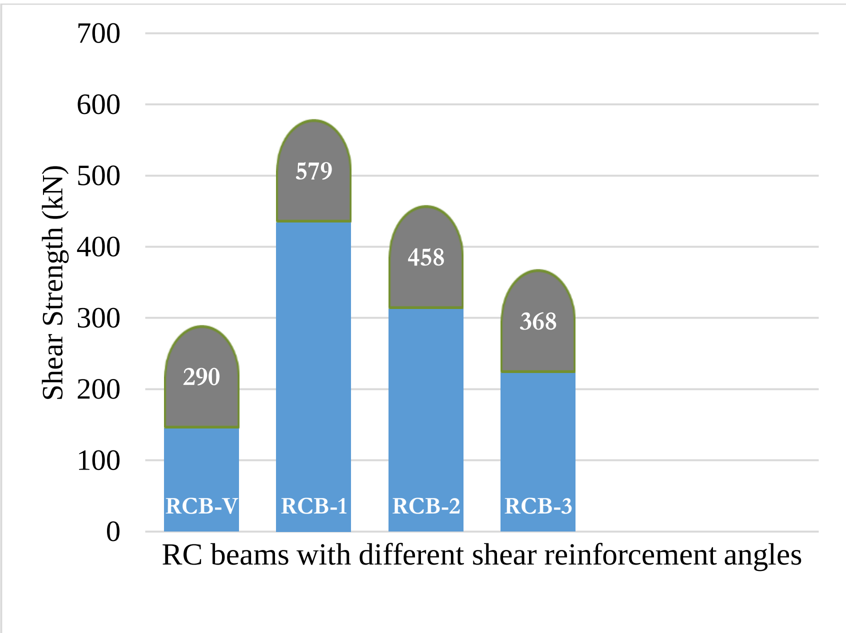


Figure 4. 5: Shear strength of RC beams in 4m span length

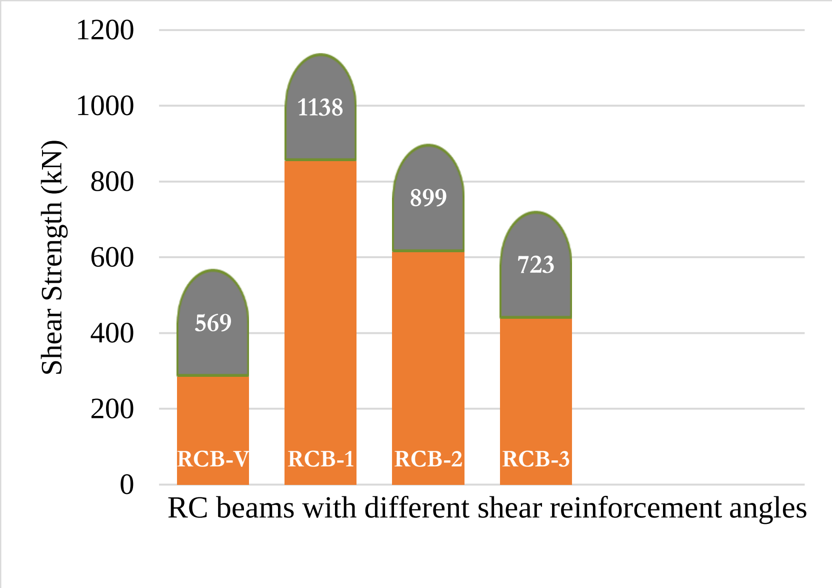


Figure 4. 6: Shear strength of RC beams in 6m span length

The above vertical bar chart Figures shows that a reinforced concrete beam with an inclination of 45^o shear reinforcement exhibit an increase in shear strength using maximum range of bar spacing. The difference in amount of shear strength between reinforced concrete beams with vertical stirrups and reinforced concrete beams with inclined shear reinforcements becomes less, then the shear reinforcement spacing provided in reinforced concrete beams gets smaller.

Reinforced concrete beams with an inclination of 45^o, 60^o and 75^o shear reinforcement system showed an improvement in beam shear performance of 50%, 37% and 21% over the beams reinforced with vertical stirrups along 2m,4m and 6m span length respectively.

4.4 Finite Element Analysis Results

4.4.1 Shear capacity of reinforced concrete beams

Reinforced concrete beams with different shear reinforcement shapes analyze for shear under a four-point loading system. Shear span about 13.85% of the span length used for all reinforced concrete beams analysis. Reinforced concrete beams were subjected to two concentrated applied loads at the top of the cross-section to study the load-deflection of reinforced concrete beams. The same spacing is used for the comparison of all RC beams shear capacity along different span lengths. The maximum shear reinforcement used along 2m,4m, and 6m span lengths are 150mm, 225mm, and 325mm, respectively. These maximum shear reinforcement spacing are used based on the design of reinforced concrete beams with vertical stirrups.

Reinforced concrete beams using the same spacing and different shear reinforcement shapes were studied based on determining the load-deflection of beams subjected to a static load. The load-deflection results of reinforced concrete beams having the same spacing stirrups and different configurations of shear reinforcement shapes along span length of 2m, 4m, and 6m shown in Figures below.

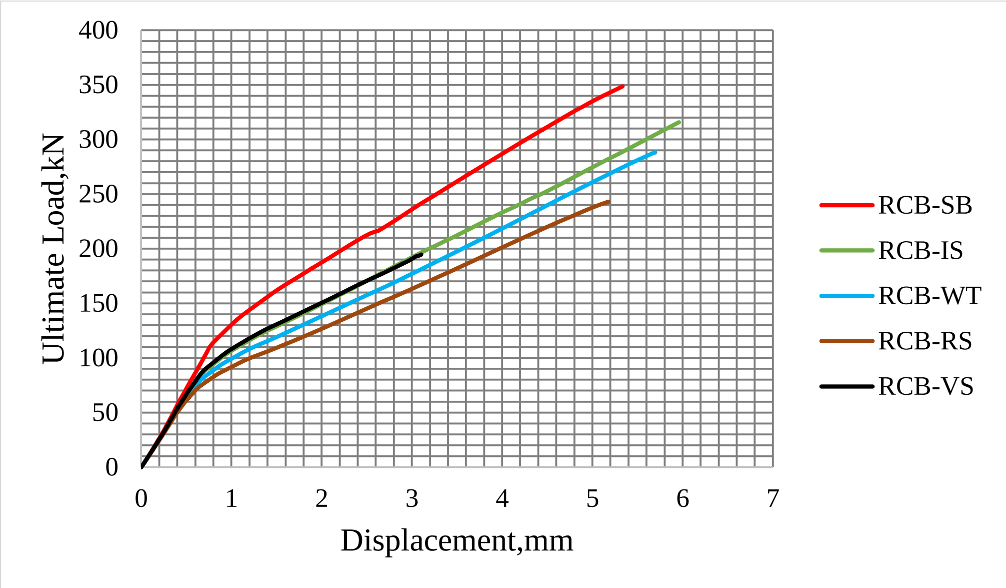


Figure 4. 7: Ultimate load-displacement curve for 2m span length of RC beams

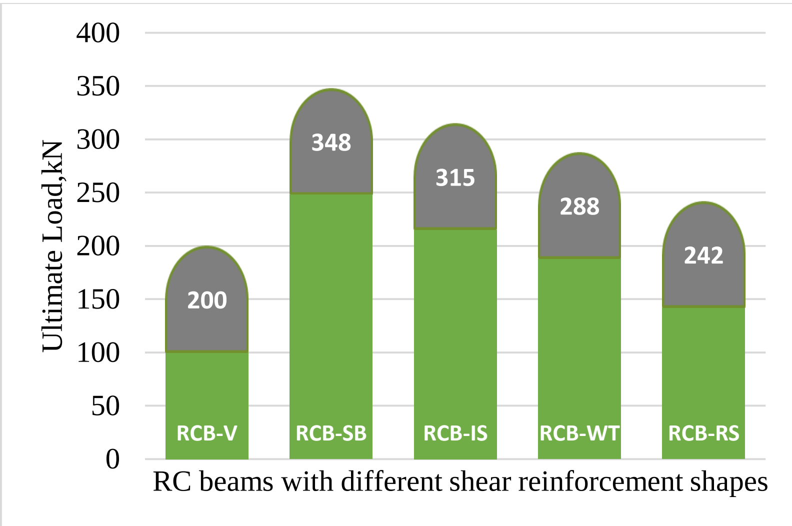


Figure 4. 8: RC beams shear capacity along 2m span length

From the Figure 4.7 and 4.8, Finite element model of the beams with swimmer bars, inclined stirrups, warren truss and rectangular spiral shapes as shear reinforcement showed 43%, 37%, 31% and 17% respectively, increased shear carrying capacity compared to reinforced concrete beam model with vertical shear reinforcements.

The deflection of reinforced concrete beams along 2m span length having different shapes and the same spacing of shear reinforcements shown in Figures below.

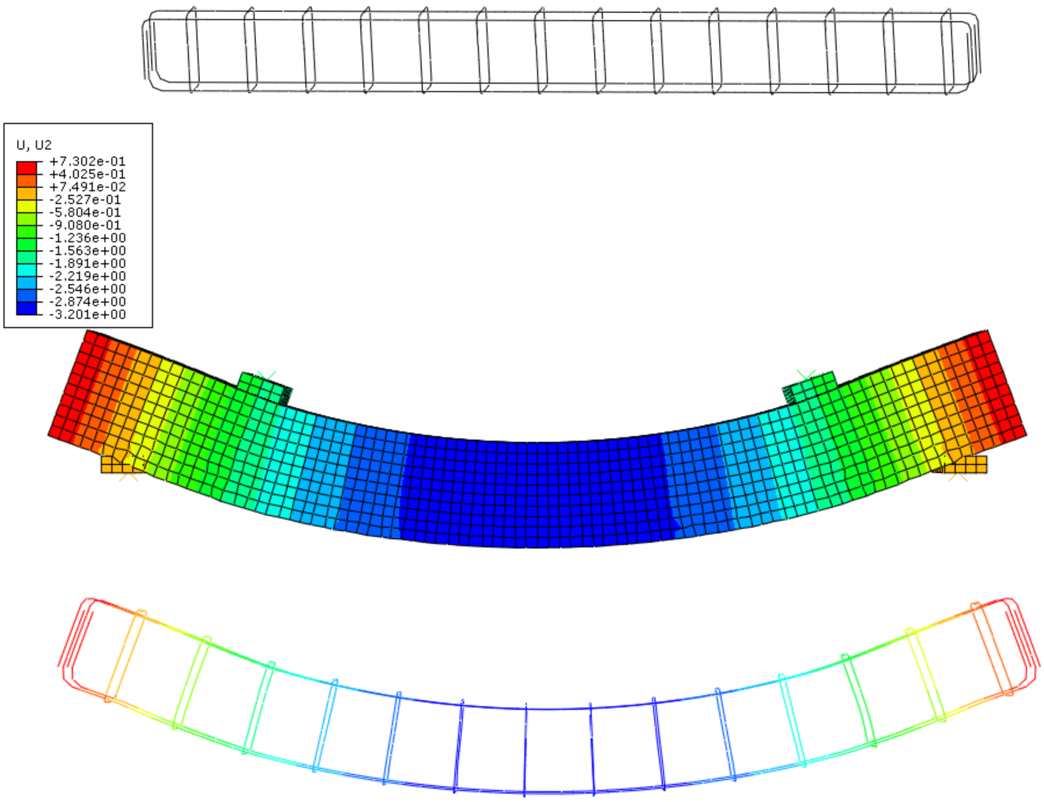


Figure 4. 9: Deflection of RCB-V along 2m span length

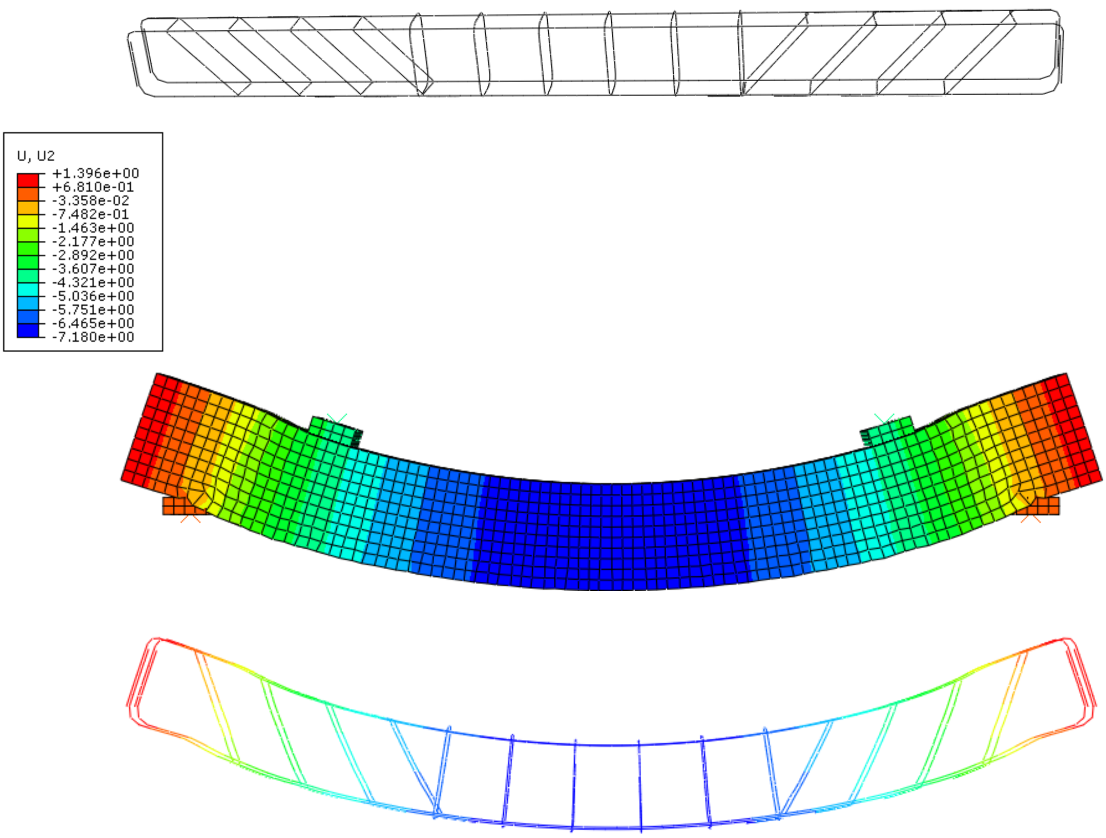


Figure 4. 10: Deflection of RCB-SB along 2m span length

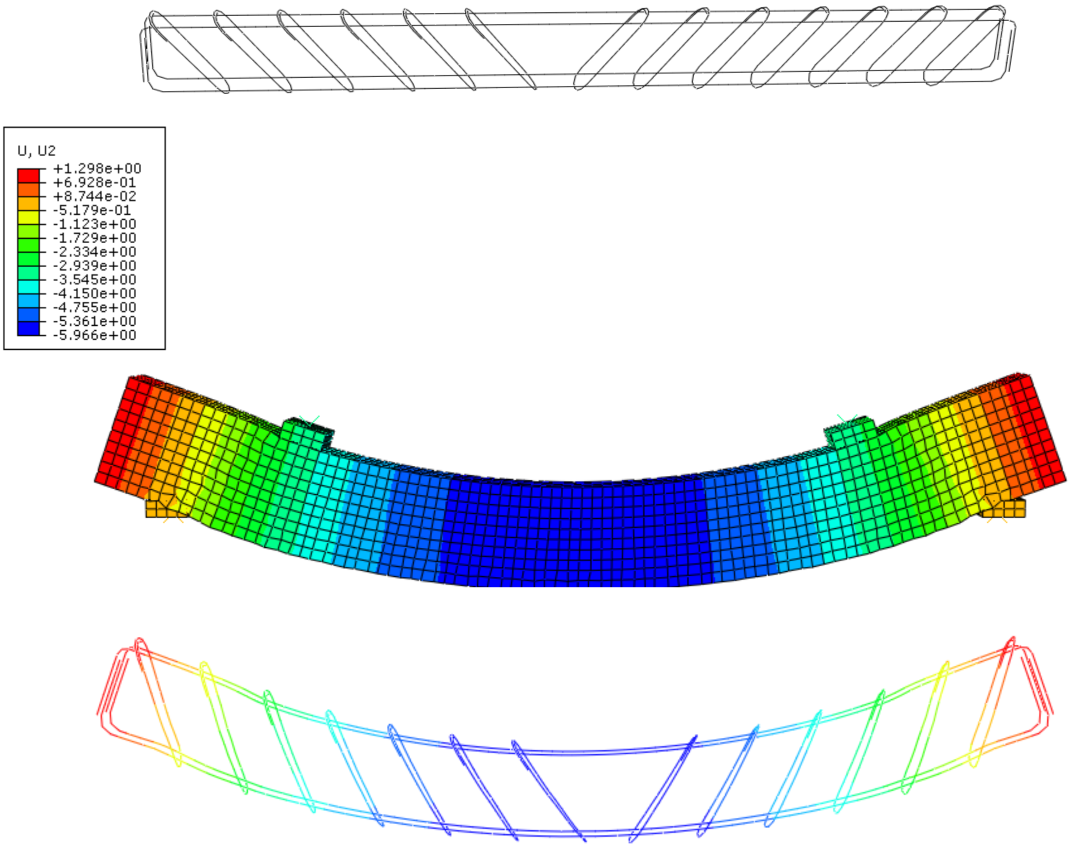


Figure 4. 11: Deflection of RCB-IS along 2m span length

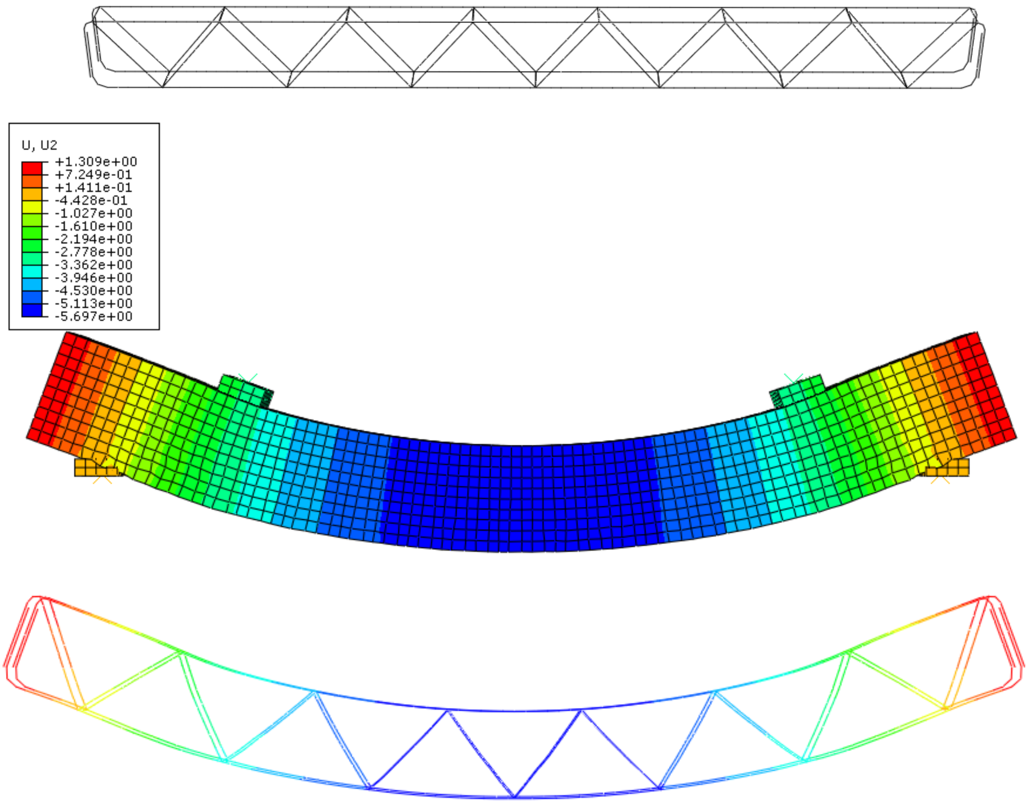


Figure 4. 12: Deflection of RCB-WT along 2m span length

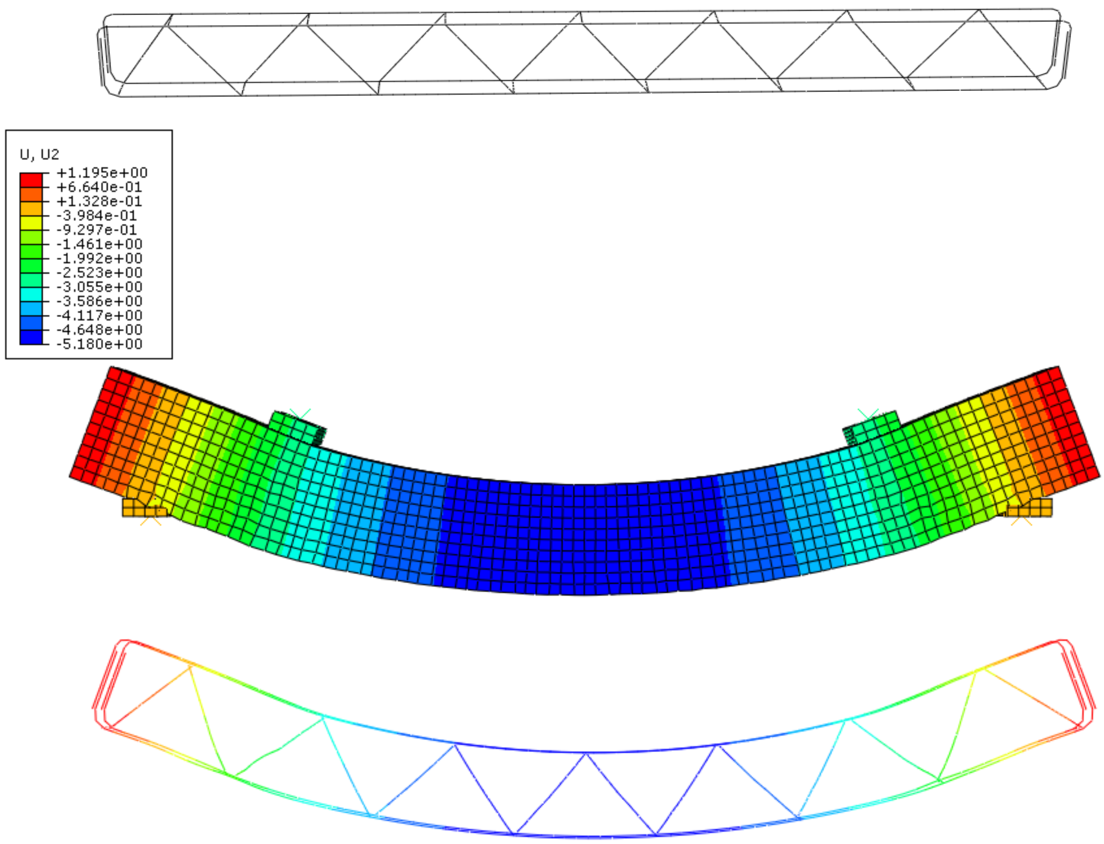


Figure 4. 13: Deflection of RCB-RS along 2m span length

The load-deflection results of reinforced concrete beams having the same spacing and different shear reinforcement shapes along 4m span length shown in Figure below.

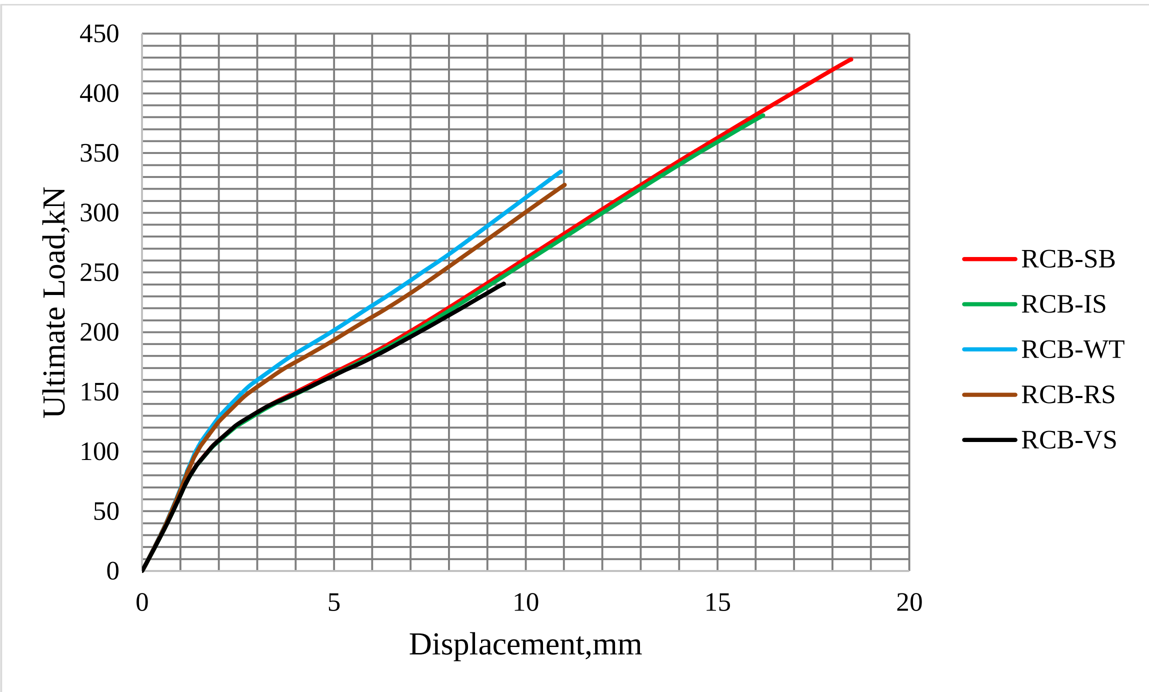


Figure 4. 14: Ultimate load-displacement curve for 4m span length of RC beams

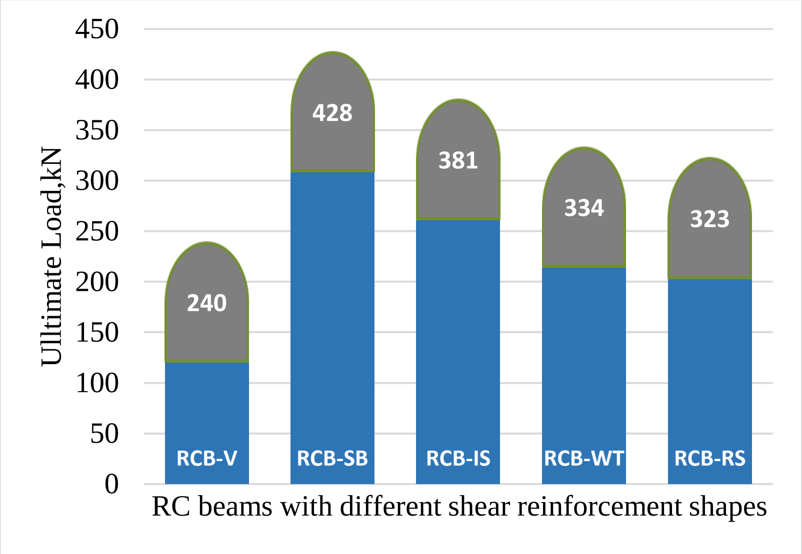


Figure 4. 15: RC beams shear capacity along 4m span length

From the Figure 4.14 and 4.15, Finite element model of the beams with swimmer bars, inclined stirrups, warren truss and rectangular spiral shapes as shear reinforcement showed 44%, 37%, 28% and 26% respectively, increased shear carrying capacity compared to reinforced concrete beam model with vertical shear reinforcements.

The deflection of reinforced concrete beams along 4m span length having different shapes and the same spacing of shear reinforcements shown in Figures below.

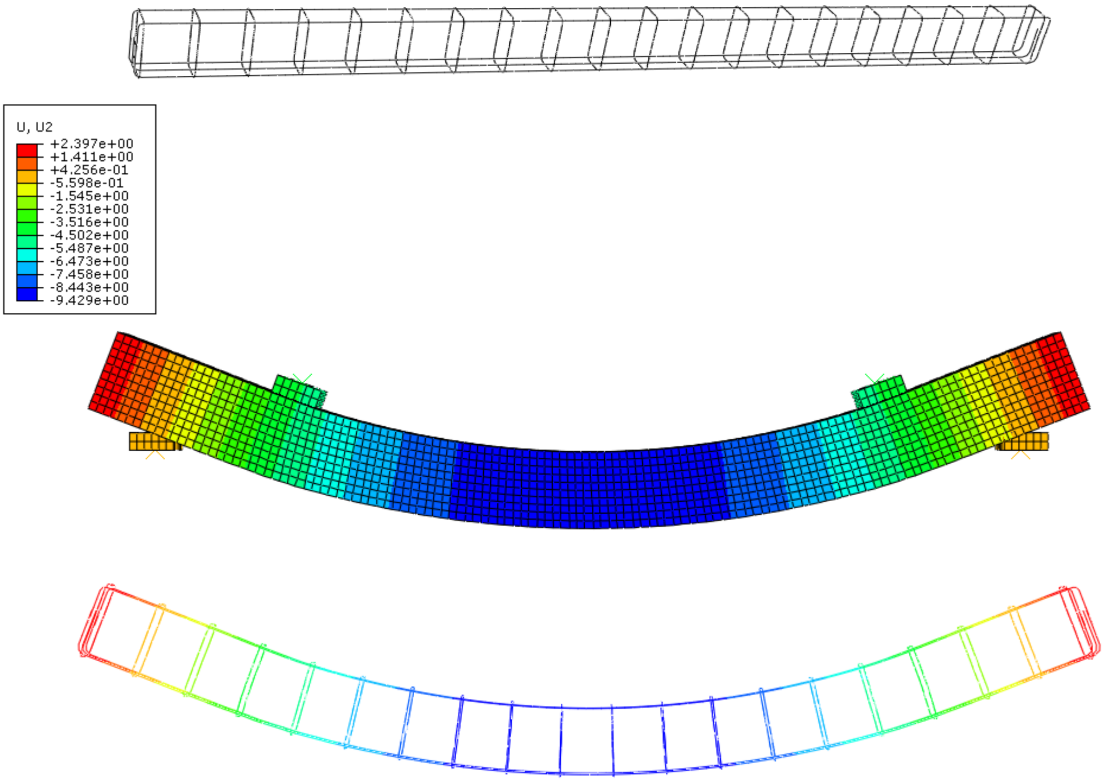


Figure 4. 16: Deflection of RCB-V along 4m span length

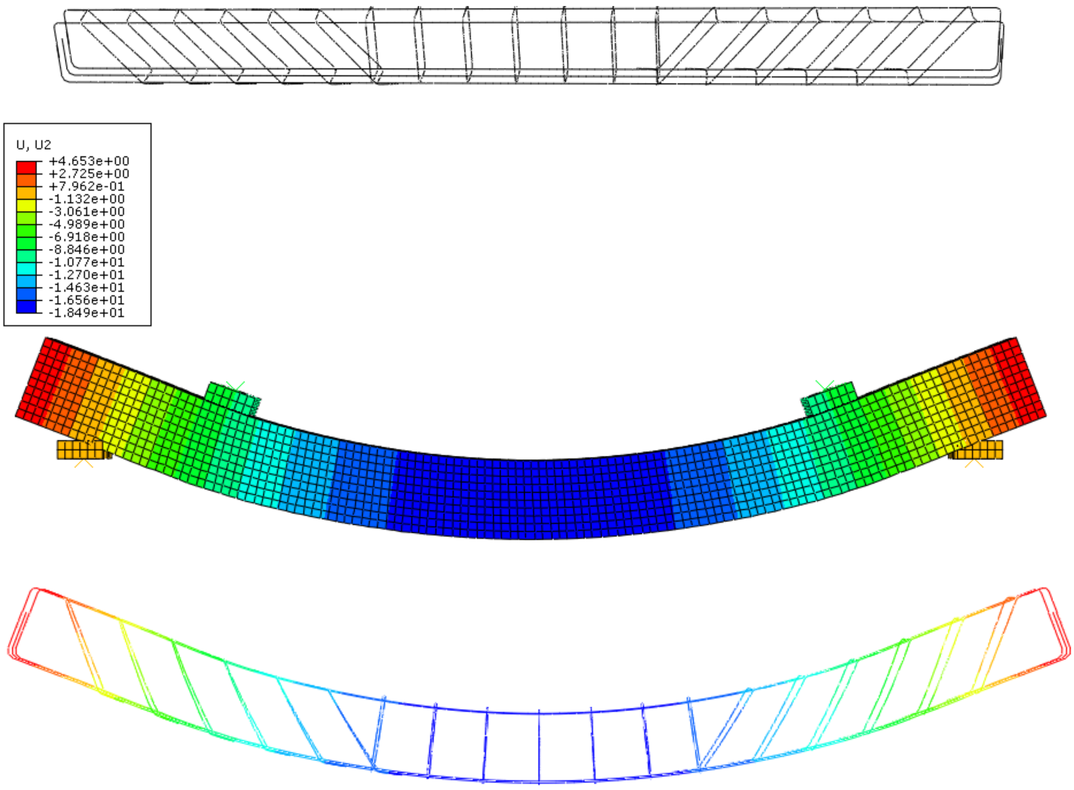


Figure 4. 17: Deflection of RCB-SB along 4m span length

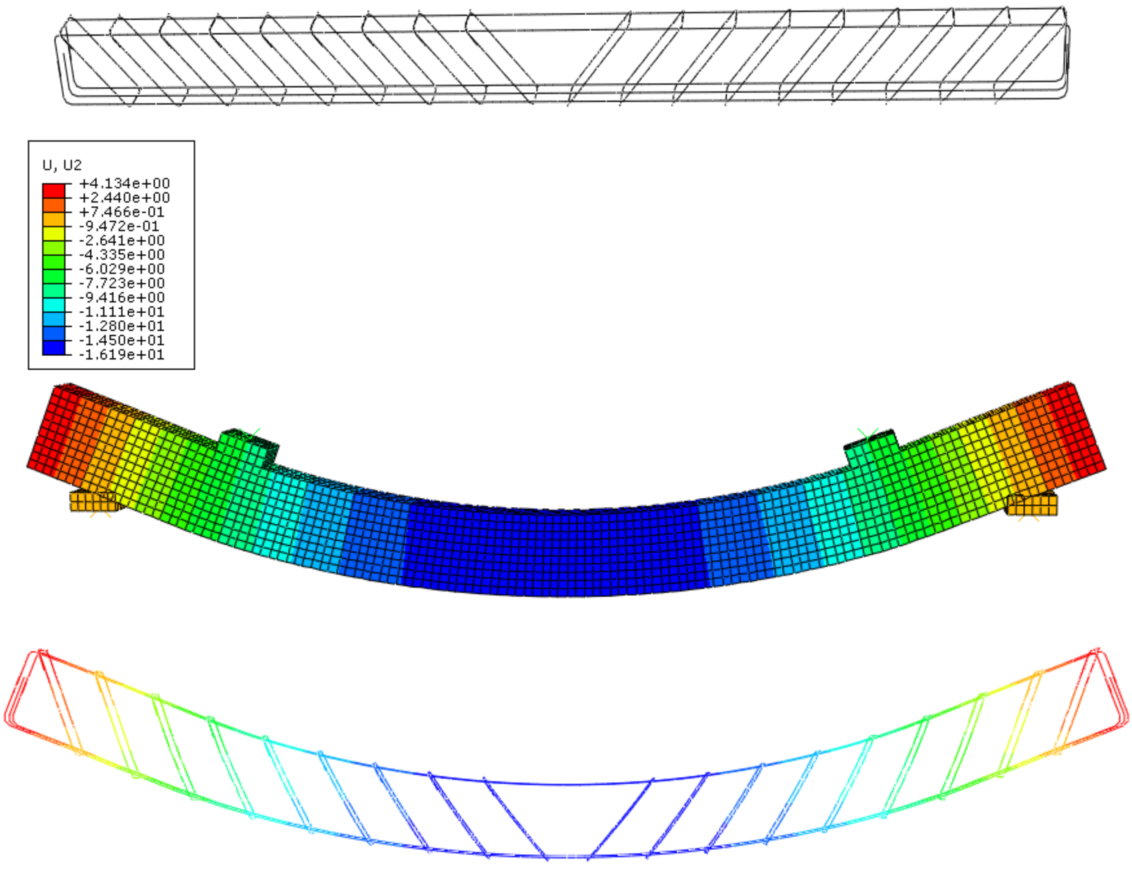


Figure 4. 18: Deflection of RCB-IS along 4m span length

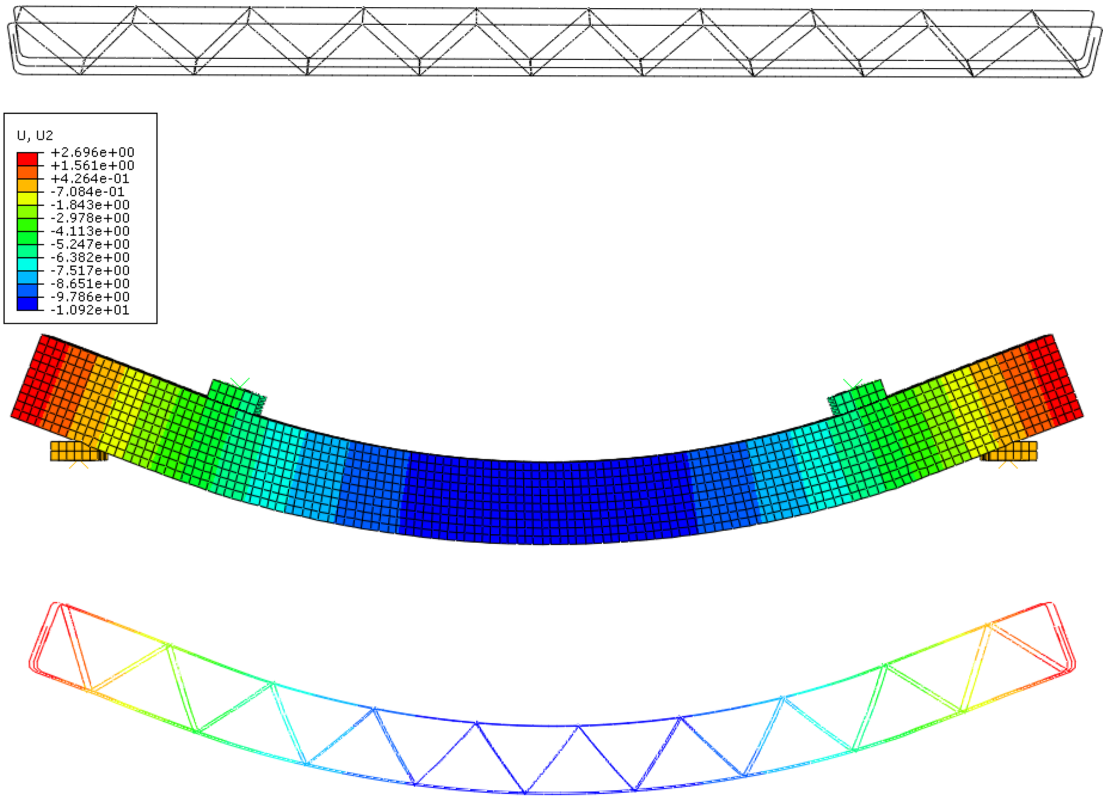


Figure 4. 19: Deflection of RCB-WT along 4m span length

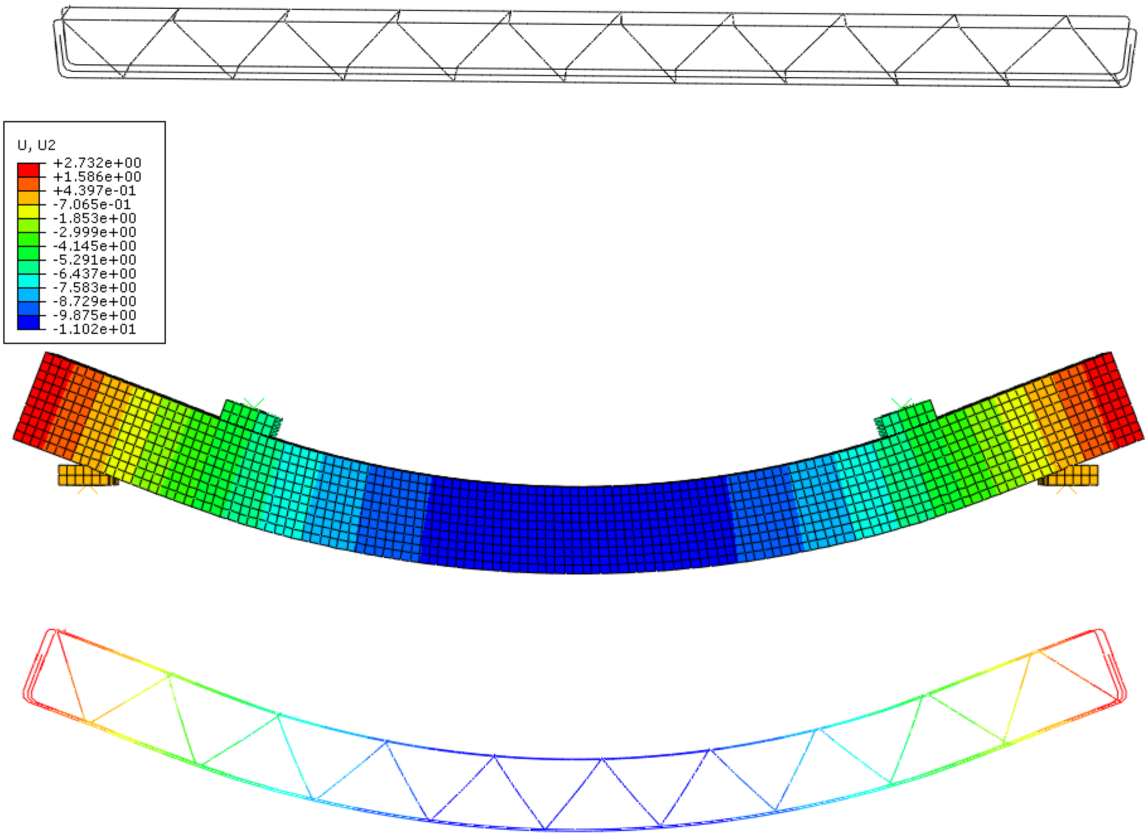


Figure 4. 20: Deflection of RCB-RS along 4m span length

The load-deflection results of reinforced concrete beams having the same spacing and different shear reinforcement shapes along 6m span length shown in Figure below.

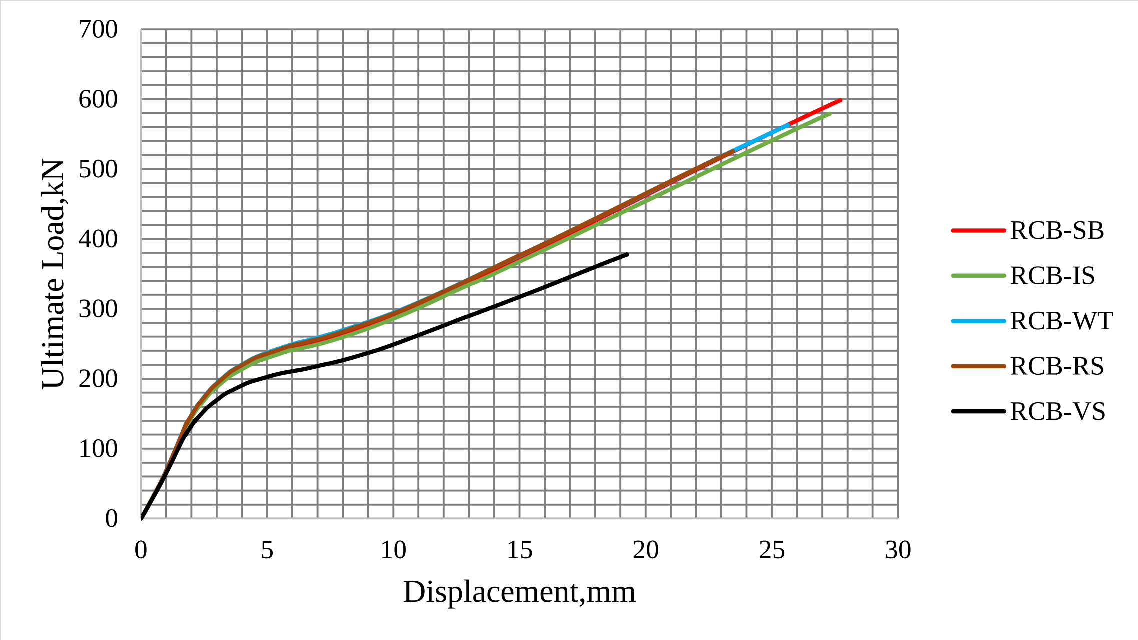


Figure 4. 21: Ultimate load-displacement curve for 6m span length of RC beams

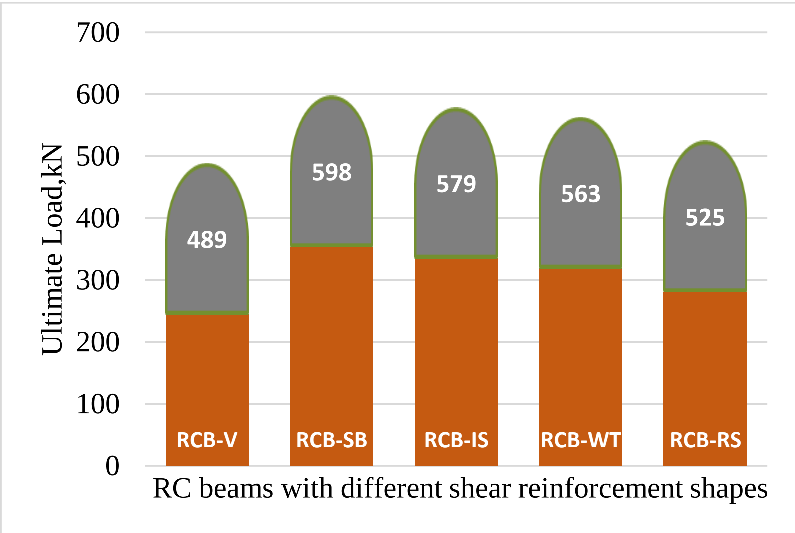


Figure 4. 22: RC beams shear capacity along 6m span length

From the Figure 4.21 and 4.22, Finite element model of the beams with swimmer bars, inclined stirrups, warren truss and rectangular spiral shapes as shear reinforcement showed 18%, 16%, 13% and 7% respectively increased shear carrying capacity compared to reinforced concrete beam model with vertical shear reinforcements. The deflection of reinforced concrete beams along 6m span length having different shapes and the same spacing of shear reinforcements shown in Figures below.

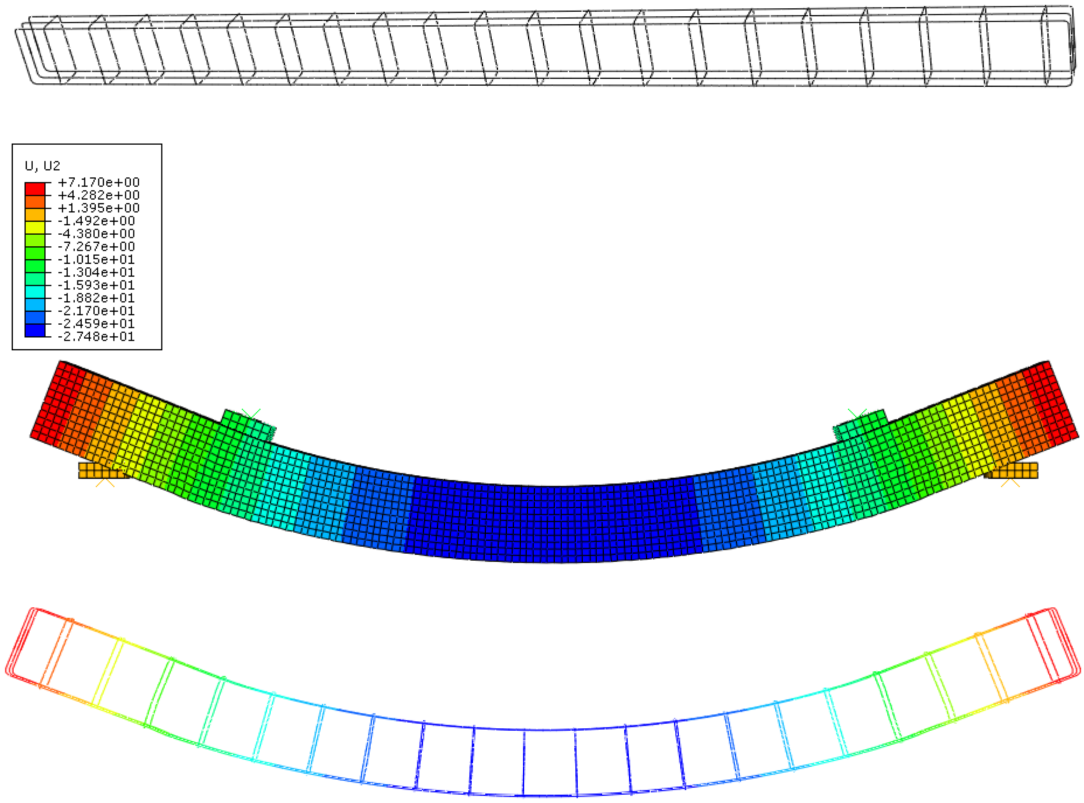


Figure 4. 23: Deflection of RCB-V along 6m span length

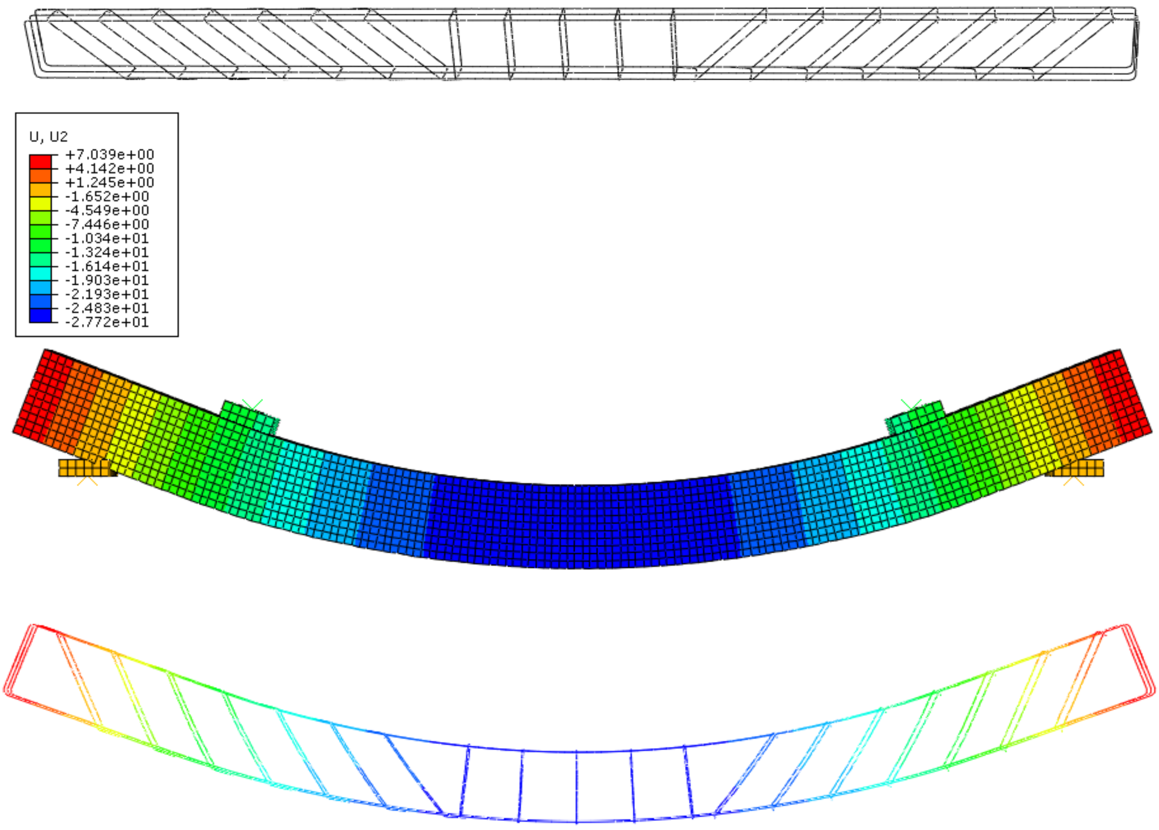


Figure 4. 24: Deflection of RCB-SB along 6m span length

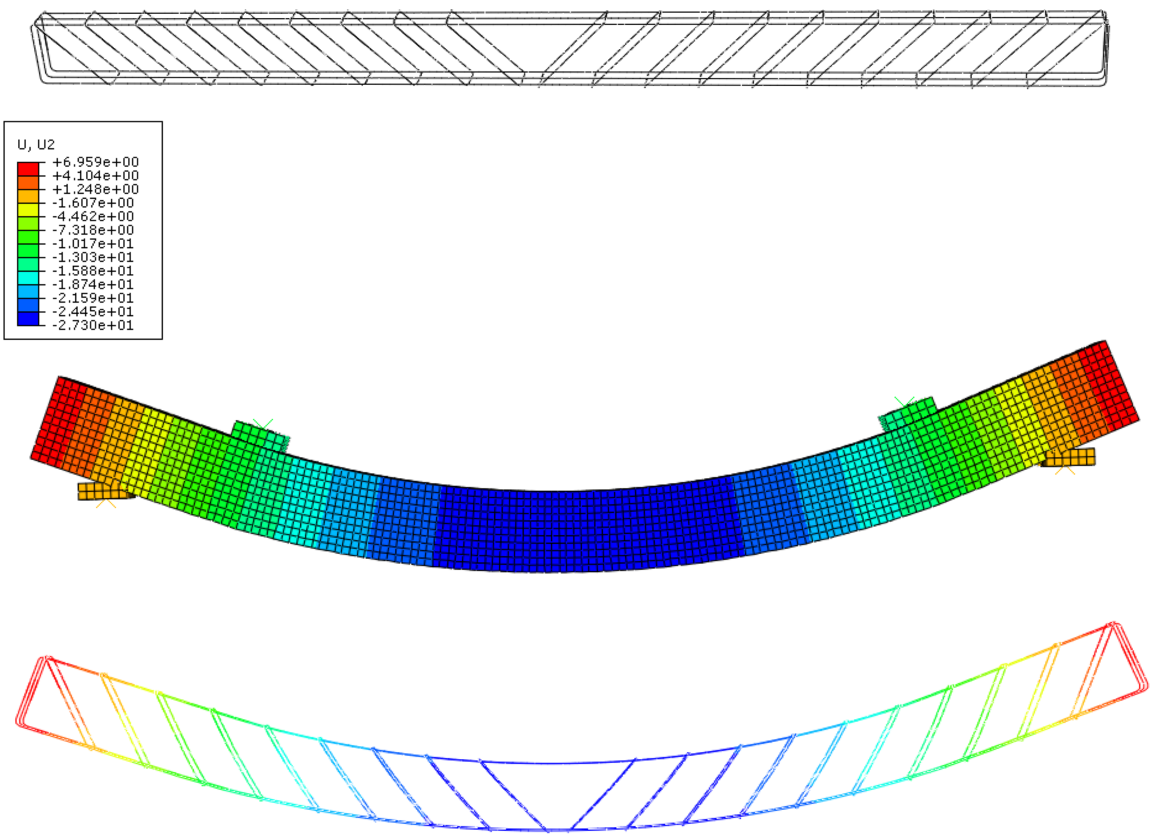


Figure 4. 25: Deflection of RCB-IS along 6m span length

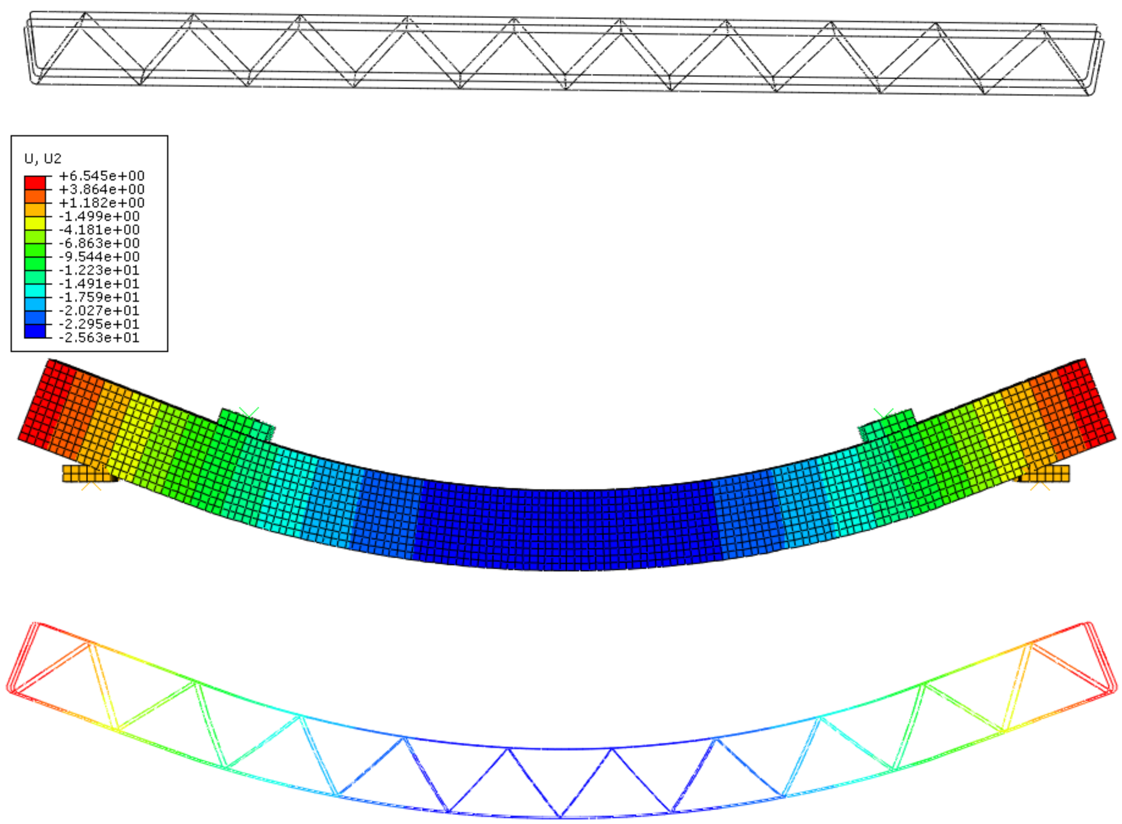


Figure 4. 26: Deflection of RCB-WT along 6m span length

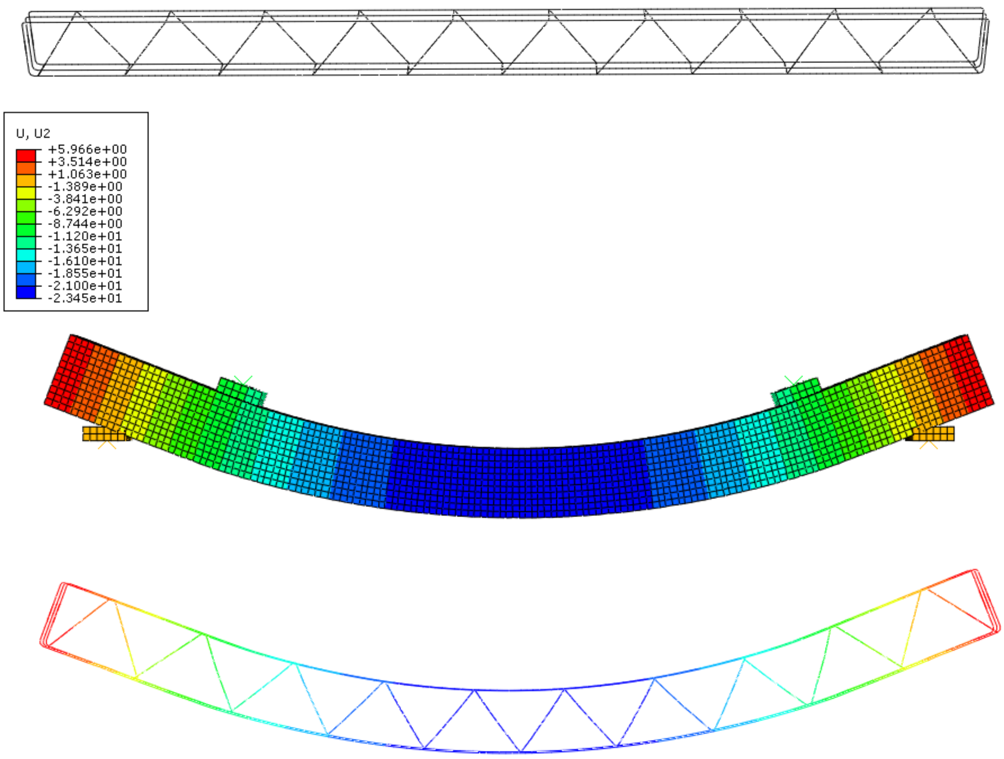


Figure 4. 27: Deflection of RCB-RS along 6m span length

Generally, from the Figures above, Swimmer bars as shear reinforcement shapes gives the maximum shear capacity for a beam as compared to the other shear reinforcement shapes because of inclined reinforcement with both ends bent horizontally for a short distance along with compressive and tensile reinforcements provision to resist principal tensile stress in inclined and horizontal direction. The RC beams shear cracks having different shear reinforcement shapes developed in a diagonal at two-point loading shown in the Figure below.

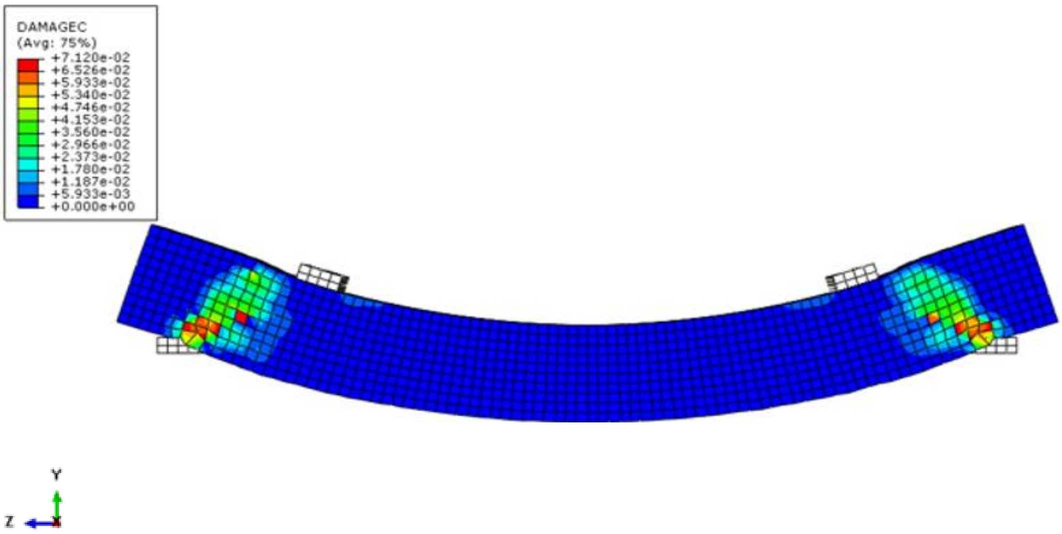


Figure 4. 28: Shear cracks developed in RC beams model

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

This paper presents a finite element model use to analyze the non-linear behavior of reinforced concrete elements. This finite element model is validated using previous experimental results available in the literature. A total of fifteen reinforced concrete beams were analyzed and compared in ABAQUS software, using a four-point bending set-up to study reinforced concrete beams with vertical, inclined, swimmer bars, rectangular spirals, and warren truss as shear reinforcement shapes. From theoretical and analytical results obtained, it observed that:

- An inclination of 45^o, 60^o, and 75^o shear reinforcement system showed higher shear strength of 50%, 37%, and 21% respectively over the beams reinforced with vertical stirrups along 2m, 4m, and 6m span length when the beams subjected to the same load.
- Reinforced concrete beams with 45^o inclined shear reinforcements showed higher shear strength compared to the beams having other angles of inclination based on EC2 design.
- The increased shear capacity of reinforced concrete beams using swimmer bars, inclined stirrups, warren truss, and rectangular spirals as shear reinforcement shapes compared to reinforced concrete beams with vertical stirrups along 2m, 4m, and 6m span length showed averagely 35%, 30%, 24%, and 17% respectively considering the same shear reinforcement spacing.
- Reinforced concrete beams with swimmer bar shear reinforcements enhanced higher shear capacity when compared to reinforced concrete beams with other shear reinforcement shapes used in this study.
- Continuous rectangular spiral shear reinforcement shapes showed lower ultimate load-bearing capacity compared to other inclined shear reinforcement shapes in reinforced concrete beams.

5.2 Recommendations

The following recommendations are suggested for future research studies:

- This study showed swimmer bars as shear reinforcement enhanced higher shear capacity in reinforced concrete beams along different span length. Similarly, further studies should also conduct to determine the effectiveness of Swimmer bars in reinforced concrete columns or frames.
- The shear performance of reinforced concrete beams having 45^{o} inclination of swimmer bar and inclined stirrup shapes should also study along different span lengths of reinforced concrete beams considering different Span to depth ratios, material property, cyclic loading, and three-point loading conditions.

REFERENCES

[1] Wight, J.K. and J.G. MacGregor, Reinforced Concrete: Mechanics and Design.6 ed. 2012.

[2] Ziara, M. M. The influence of confining the compression zone in the design of structural concrete beams. Heriot-Watt University.1993

[3] Moayyad M. Al-Nasra., Naiem M. Asha. Shear reinforcements in the reinforced concrete beams. American Journal of Engineering Research (AJER).2013; 2(10): 191-199.

[4] Nawy, E. (2003). Reinforced concrete. Upper Saddle River, N.J.: Prentice Hall

[5] EN 1992-1-1: Euro code 2: Design of concrete structures - Part 1-1: General rules and rules for buildings. European Committee for Standardization.2004

[6] Tekla Tedds 2019 User guide

[7] ABAQUS (2014). ABAQUS 6.14: Documentation, Dassault Systemes Simulia Corp., Providence, RI, USA.

[8] Wang T., Hsu T.T.C. Nonlinear finite element analysis of concrete structures using new constitutive models, Computers and Structures, Iss.2001;79(32): 2781–2791.

[9] Piyamahant, “Shear behavior of reinforced concrete beams with a small amount of web reinforcement”, M. Eng. Dissertion, Kochi University of Technology, Japan,.2002.

[10] Colajanni, P., Mendola, L.L., Mancini, G., Recupero, A., Spinella, N. Shear Capacity in Concrete Beams Reinforced by Stirrup with Two Different Inclinations. Journal of Engineering.2014; 81: 444-453.

[11] Saravanakumar, P., Govindaraj, A. Influence of Vertical and Inclined Shear Reinforcement on Shear Cracking Behavior in Reinforced Concrete Beams. International Journal of Civil Engineering and Technology.2016; 7(6): 602-610.

[12] Saleh Baras, S.A., Influence of Independent Bent-Up Bars on the Shear Capacity of Reinforced Concrete Beams, Universiti Teknologi Malaysia.2013

[13] Muneeb Ullah Khan, Muhammad Fahad, Khan Shahzada. Shear Capacity Assessment of Reinforced Concrete Beams Using Swimmer Bars as Shear Reinforcement. SSRG International Journal of Civil Engineering.2015; 2(6)

[14] Kaarthik Krishna, S. Sandeep, and K. M. Mini, “Study on reinforced concrete beams with helical transverse reinforcement,” IOP Conf. Ser. Mater. Sci. Eng.2018; 310 (1)

[15] Sooryaprabha M Saju and Usha. S. Study on Flexural Strength of Truss Reinforced Concrete Beams.International Research Journal of Engineering and Technology.2016; 3(7)

[16] Rudy Djamaluddin et al. Effect of the Truss System to the Flexural Behaviour of the External Reinforced Concrete.2014

[17] Hibbitt, Karlsson. “Sorensen, Inc. ABAQUS Theory Manual.” 2000

[18] ABAQUS 6.12 Documentation Manual, 2016.

[19] Nor Fazlin Zamri, Roslli Noor Mohamed, NurHafizah A. Khalid & Kang Yong Chiat. The Effects of Inclined Shear Reinforcement in Reinforced Concrete Beam, Universiti Teknologi Malaysia, Malaysia. Malaysian Journal of Civil Engineering. 2018; 30(1):85-96

[20] B. Alfarah, F. López-Almansa, S. Oller. New methodology for calculating damage variables evolution in Plastic Damage Model for RC structures. Engineering Structures 132 (2017) 70–86.

[21] Christina Claeson and Kent Gylltoff. Slender high-strength concrete columns subjected to eccentric loading. Journal of Structural Engineering.1998;124(3):233-240

[22] Malm, R., James, G., Sundquist, H. Monitoring and evaluation of shear crack initiation and propagation in webs of concrete box-girder sections. In International Conference on Bridge Engineering-Challenges in the 21st Century, Hong Kong. Civil Division, The Hong Kong Institution of Engineers.2006

[23] Chen, W. F., Han, D. J.Plasticity for structural engineers. Springer Science & Business Media.2012

APPENDIX A

MATERIAL PROPERTY

A.1 Concrete properties

Table A. 1: Compressive Stress-total strain of concrete

σ_c	εc
0	0
8.147686415	0.00025
15.2	0.000494681
21.45087569	0.000744681
26.72373628	0.000994681
31.00523263	0.001244681
34.28177155	0.001494681
36.53951019	0.001744681
37.7643503	0.001994681
38	0.002161877
37.94193229	0.002244681
37.05762916	0.002494681
35.09654026	0.002744681
32.04348481	0.002994681
27.88299534	0.003244681
22.47458591	0.0035

Table A. 2: Stress-crushing strain

σ_c	${\varepsilon_c}^{ch}$
15.2	0
21.45087569	9.14196E-05
26.72373628	0.000180841
31.00523263	0.000300453
34.28177155	0.000450669
36.53951019	0.000631913
37.7643503	0.000844611
38	0.00100463
37.94193229	0.001089203
37.05762916	0.001366134
35.09654026	0.001675857
32.04348481	0.002018834
27.88299534	0.002395537
22.47458591	0.002815562

Table A. 3: Concrete damage variable

dc	$\varepsilon_c{}^{ch}$
0	0
2.96688E-05	1.87156E-06
0.000507873	3.17829E-05
0.00148381	9.14196E-05
0.003002457	0.000180841
0.005135187	0.000300453
0.007972838	0.000450669
0.011623379	0.000631913
0.016208805	0.000844611
0.019862497	0.00100463
0.021861358	0.001089203
0.028719182	0.001366134
0.036921546	0.001675857
0.046603813	0.002018834
0.057892312	0.002395537
0.071195419	0.002815562

Table A. 4: Tensile Stress-Total strain of concrete

σ_c	εt
0	0
2.896468154	8.82086E-05
2.620832766	0.000188209
2.371427553	0.000288209
2.145756384	0.000388209
1.941560666	0.000488209
1.75679674	0.000588209
1.589615426	0.000688209
1.438343518	0.000788209
1.301467035	0.000888209
1.177616072	0.000988209
1.065551086	0.001088209
0.964150493	0.001188209
0.872399443	0.001288209
0.789379659	0.001388209
0.714260253	0.001488209
0.646289403	0.001588209
0.584786835	0.001688209
0.52913701	0.001788209
0.478782966	0.001888209
0.456896099	0.001935

Table A. 5: Tensile stress-cracking strain of concrete

σ_t	ε_{cr}
2.896468154	0
2.620832766	0.000108394
2.371427553	0.00021599
2.145756384	0.000322862
1.941560666	0.000429081
1.75679674	0.000534707
1.589615426	0.000639799
1.438343518	0.000744406
1.301467035	0.000848574
1.177616072	0.000952346
1.065551086	0.001055758
0.964150493	0.001158847
0.872399443	0.001261641
0.789379659	0.001364169
0.714260253	0.001466457
0.646289403	0.001568527
0.584786835	0.0016704
0.52913701	0.001772094
0.478782966	0.001873628
0.456896099	0.001921086

Table A. 6: Tensile damage variables-cracking strain of concrete

d_t	ε_{cr}
0	0
0.05611099	0.000103649
0.111125953	0.000206961
0.164572899	0.000309989
0.216122779	0.000412778
0.265556052	0.000515363
0.31273694	0.00061777
0.357593517	0.000720023
0.40010228	0.00082214
0.440276166	0.000924136
0.478155215	0.001026025
0.513799306	0.001127818
0.547282494	0.001229523
0.578688596	0.00133115
0.608107764	0.001432704
0.635633829	0.001534192
0.661362248	0.001635618
0.685388538	0.001736989
0.707807082	0.001838307
0.71777107	0.001885699

A.2 Steel properties

Table A. 7: Tensile stress-strain of reinforcements

fy	εt
0	0
466	0.002108597
466	0.005
466	0.04
484.6484375	0.045
502.09375	0.05
518.3359375	0.055
533.375	0.06
547.2109375	0.065
559.84375	0.07
571.2734375	0.075
581.5	0.08
590.5234375	0.085
598.34375	0.09
604.9609375	0.095
610.375	0.1
614.5859375	0.105
617.59375	0.11
619.3984375	0.115
620	0.12

APPENDIX B

RC BEAMS DESIGN RESULTS

Table B. 1: Calculation of required nominal concrete cover for reinforcement steel

Defined Parameters	Values	Units
Concrete characteristic strength, fck	30	Mpa
The maximum longitudinal reinforcement diameter	16	mm
Exposure classes related to environmental conditions	XC1	
The Maximum aggregate size, dg	20	mm
The design working life of the structure	50	year
Structural class	S4	
The minimum cover for durability ,$c_{min,dur}$	10	mm
The minimum cover for bond ,$c_{min,b}$	16	mm
The minimum concrete cover c_{min}	16	mm
The allowance for deviation ,Δc_{dev}	10	mm
The required nominal concrete cover, c_{nom}	26	mm
Use c_{nom}	30	mm

Table B. 2: Determination of design anchorage length for longitudinal reinforcement

Defined Parameters	Values
Concrete characteristic strength, fck	30 N/mm^2
Steel characteristic yield strength, f_{yk}	460 N/mm^2
Coefficient taking account of long term effects and loading effects on the tensile strength of concrete, α_{ct}	1
Concrete partial material safety factor	1.5
Reinforcement steel partial material safety factor	1.15
Coefficient α1 accounts for the effect of shape of the bar (straight bars)	1 mm
Bottom Longitudinal Reinforcement diameter	16 mm
coefficient η_1 for 'good' bond conditions	1
coefficient η_2 takes into account the effect of large bar diameters $\Phi > 32$	1
the design yield strength of the bar, fyd	400 mm
mean tensile strength fctm	2.9 N/mm^2

5% fractile tensile strength, $f_{ctk,0.05}$	2.03 N/mm^2
the design tensile strength of concrete, fctd	1.35 N/mm^2
The design value of ultimate bond stress for ribbed bars,fbd	3.04 N/mm^2
The basic required anchorage length, $l_{b,rqd}$	526.32 mm
Minimum anchorage length lb,min for anchorages in tension	160 mm
Minimum anchorage length lb,min for anchorages in compression	316 mm
The design anchorage length, l_{bd}	526 mm

Design summary for reinforced concrete beam with 4m span length

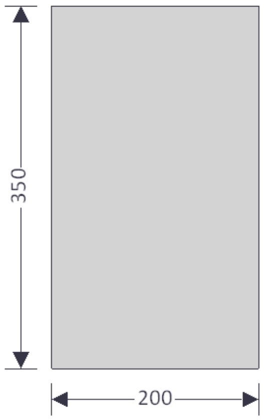


Figure B. 1: RC beam cross-section for 4m span length

Moment Design

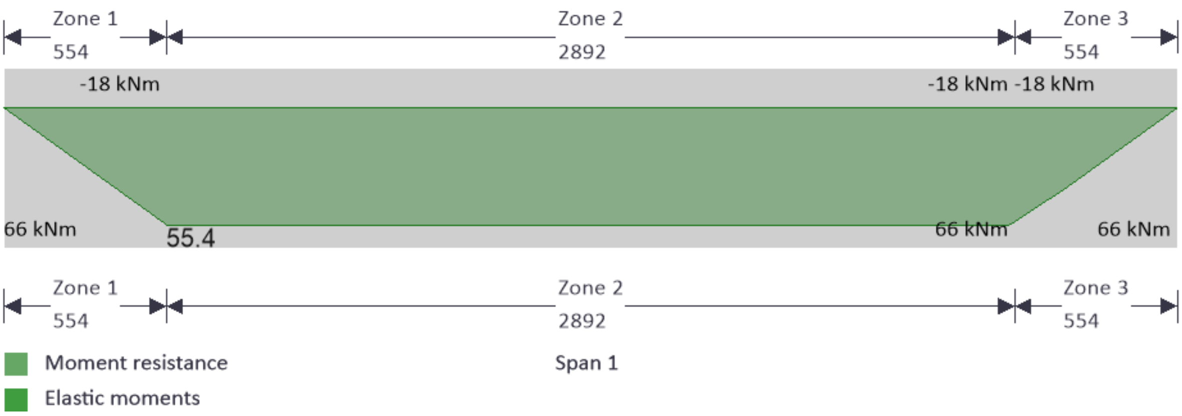


Figure B. 2: Moment resistance and elastic moments result for 4m span length

Table B. 3: Determination of RC beams shear span in 4m span length

Defined Parameters	**Values**
Height of the beam cross-section, h(mm)	350
The required nominal concrete cover, cnom(mm)	30

Width of the supporting element (mm)	200
Top Longitudinal Reinforcement diameter(mm)	10
Bottom Longitudinal Reinforcement diameter(mm)	16
Shear Reinforcement diameter(mm)	8
Effective depth of the beam cross-section using top diameter(mm)	307
Effective depth of the beam cross-section using bottom diameter(mm)	304
Clear Span of the beam(mm)	3800
a1(mm)	175
a2(mm)	100
the minimum value from a1 and a2(mm)	100
The effective span, l_{eff} of a member (mm)	4000
The minimum shear span, av(mm)	152
The maximum shear span, av(mm)	608
The effective span between top and bottom center of support plates (mm)	554

Table B. 4: Moment design summary for 4m span length

Defined Parameters	**Zone 1 (0 mm - 554 mm) Positive moment - section 6.1**	**Zone 1 (0 mm - 554 mm) Negative moment - section 6.1**	**Zone 2 (554 mm - 3446 mm) Positive moment - section 6.1**	**Zone 3 (3446 mm - 4000 mm) Positive moment - section 6.1**	**Zone 3 (3446 mm - 4000 mm) Negative moment - section 6.1**
PL(KN)	100	100	100	100	100
M(kNm)	55.4	13.9	55.4	55.4	13.9
d	304	307	304	304	307
δ	1	1	1	1	1
k	0.1	0.025	0.1	0.1	0.025
K'	0.207	0.207	0.207	0.207	0.207
Checking compression	No Required	No Required	No Required	No Required	No Required
Z(mm)	274.3	291.65	274.27	274.27	291.65
x(mm)	74	38	74	74	38

$As_{,req}$ (mm^2)	505	119	505	505	119
reinforcement provided;	3 x 16ϕ	2 x 10ϕ	3x 16ϕ	3 x 16ϕ	2 x 10ϕ
$As_{,prov}$ (mm^2)	603	157	603	603	157
$As_{,min}$(mm^2)	100	101	100	100	101
$As_{,max}$(mm^2)	2800	2800	2800	2800	2800
Checking	Pass	Pass	Pass	Pass	Pass
Crack control - Section 7.3					
wk	0.4	0.4	0.4	0.4	0.4
Es(Mpa)	200000	200000	200000	200000	200000
$f_{ct,eff}$(N/mm^2)	2.9	2.9	2.9	2.9	2.9
kc	0.4	0.4	0.4	0.4	0.4
k	1	1	1	1	1
s_{bar}(mm)	54	114	54	54	114
$_{\sigma s}$ (N/mm^2)	360	349	360	360	349
αcr	6.09	6.09	6.09	6.09	6.09
y(mm)	169.581	173.51	169.581	169.581	173.51
Act(mm^2)	33916	34702	33916	33916	34702
Asc,min(mm^2)	109	115	109	109	115
Checking	Pass	Pass	Pass	Pass	Pass
R_{PL}	0.65	0.65	0.65	0.65	0.65
σsr(mm^2)	218	197	218	218	197
sbar,max(mm)	277.9	300	277.9	277.9	300
Checking	Pass	Pass	Pass	Pass	Pass
ρm0			0.00548		
ρ_m			0.00831		
ρ^'m			0		
Kb			1		
$span\_to\_depth_{basic}$			16.418		
Ks			1.298		
F1			1		

F2			1		
span_to_depth$_{allow}$			21.31		
span_to_depth$_{actual}$			13.158		
Checking			Pass		
Minimum bar spacing (Section 8.2)					
s_{bar}(mm)		104	104		104
stop,min(mm)		25	25		25
Checking		Pass	Pass		Pass
s_{bot}(mm)	38		38	38	
sbot,min(mm)	25		25	25	
Checking	Pass		Pass	Pass	

Shear Design

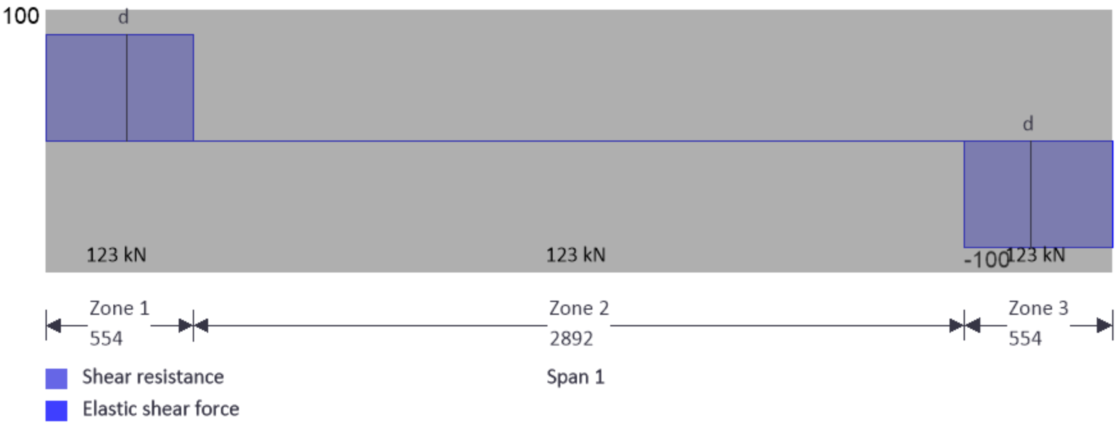


Figure B. 3: Shear resistance and elastic shear force result for 4m span length

The Shear design along Zone 1 (0 mm - 831 mm), Zone 2 (831 mm - 5169 mm) and Zone 3 (5169 mm - 6000 mm) using shear - section 6.2 shown in the table below

Table B. 5: Shear design for 4m span length

Defined Parameters	**Vertical Stirrups (90º)**	**Inclined Stirrups (45º)**	**Inclined Stirrups (60º)**	**Inclined Stirrups (75º)**
θmax (deg)	45	45	45	45
v1	0.528	0.528	0.528	0.528
αcw	1	1	1	1

Asv,min	191	136	118	114
$V_{Ed,max}$ (kN)	100	100	100	100
z	274.3	274.3	274.3	274.3
V Rd,max(kN)	290	579	458	368
Checking	Pass	Pass	Pass	Pass
VEd	100	100	100	100
v_{Ed}(N/mm^2)	1.8228	1.8228	1.8228	1.8228
θ (deg)	21.8	21.8	21.8	21.8
Asv,des(mm^2/mm)	365	259	317	354
Asv,req(mm^2/mm)	365	259	317	354
2 x 8 legs @ c/c	225	450	350	275
Asv,prov(mm^2/mm)	447	223	287	366
Checking	Pass	Pass	Pass	Pass
svl,max	228	456	360	290
Checking	Pass	Pass	Pass	Pass

Table B. 6: Moment design reinforcement areas results for 4m span length

	Unit	Provided	Required	Utilisation	Result
Zone 1 top (-13.9 kNm)	mm^2	157	119	0.756	PASS
Zone 1 bot (55.4 kNm)	mm^2	603	505	0.837	PASS
Zone 2 top (0.0 kNm)	mm^2	157	0	0.000	PASS
Zone 2 bot (55.4 kNm)	mm^2	603	505	0.837	PASS
Zone 3 top (-13.9 kNm)	mm^2	157	119	0.756	PASS
Zone 3 bot (55.4 kNm)	mm^2	603	505	0.837	PASS
Span to depth ratio		13.16	21.32	0.617	PASS

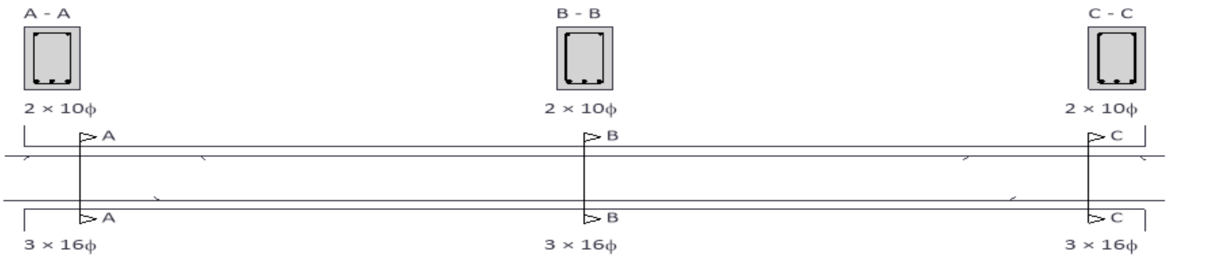


Figure B. 4: The top and bottom reinforcements provision result for 4m span length

Table B. 7: Shear design reinforcement areas results for 4m span length

	Unit	Provided	Required	Utilisation	Result
Zone 1 (100.0 kN)	mm²/m	447	365	0.816	PASS
Zone 2 (100.0 kN)	mm²/m	447	365	0.816	PASS
Zone 3 (100.0 kN)	mm²/m	447	365	0.816	PASS

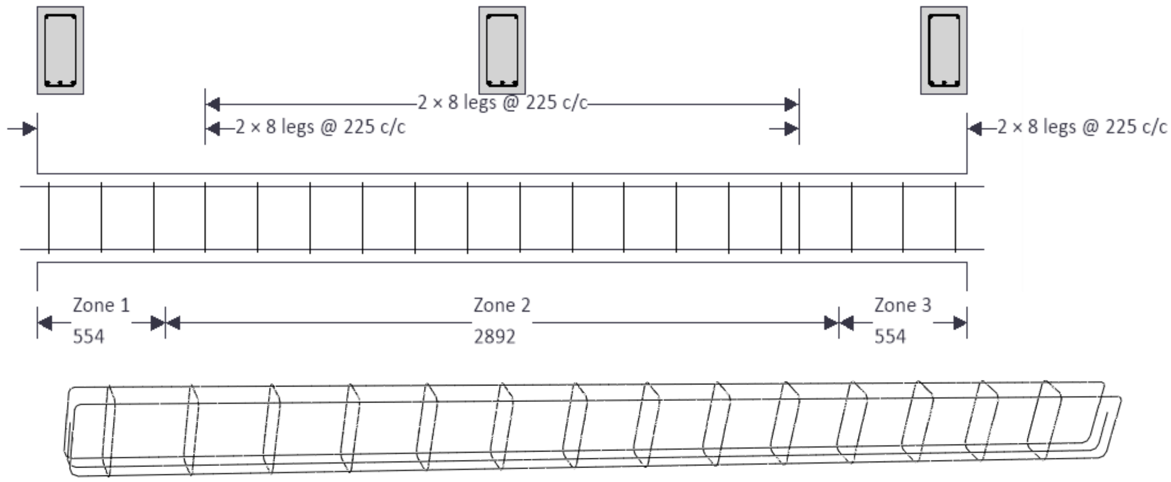


Figure B. 5: Vertical stirrups provision in 4m span length of RC beam

Figure B. 6: 45^0 inclined stirrups provision in 4m span length of RC beam

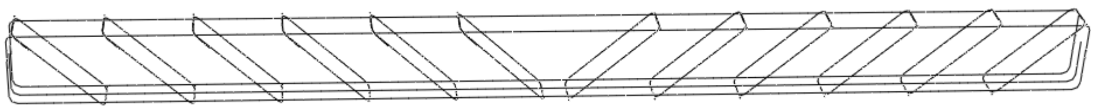


Figure B. 7: 60^0 inclined stirrups provision in 4m span length of RC beam

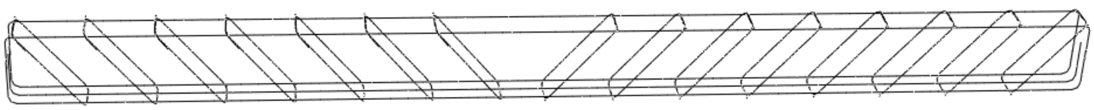


Figure B. 8: 75^0 inclined stirrups provision in 4m span length of RC beam

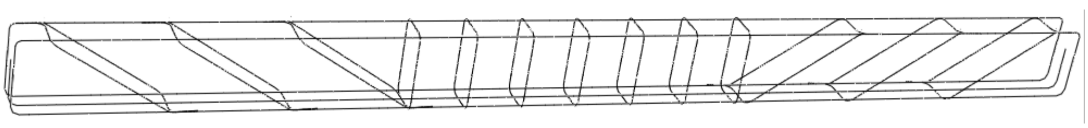


Figure B. 9: 45^0 Swimmer bars provision in 4m span length of RC beam

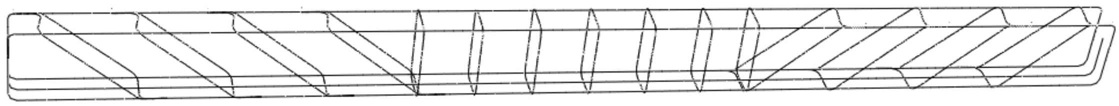


Figure B. 10: 60^0 Swimmer bars provision in 4m span length of RC beam

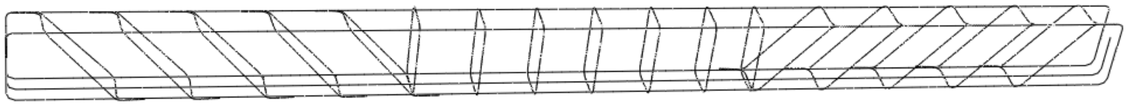


Figure B. 11: 75^0 Swimmer bars provision in 4m span length of RC beam

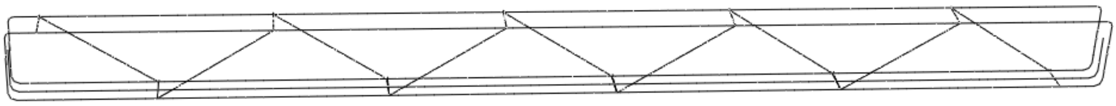


Figure B. 12: 45^0 Rectangular spiral bars provision in 4m span length of RC beam

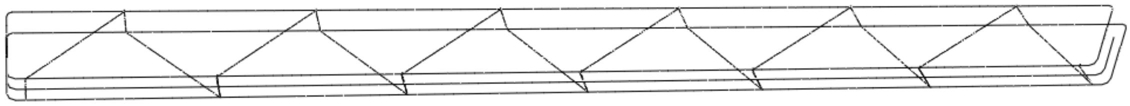


Figure B. 13: 60^0 Rectangular spiral bars provision in 4m span length of RC beam

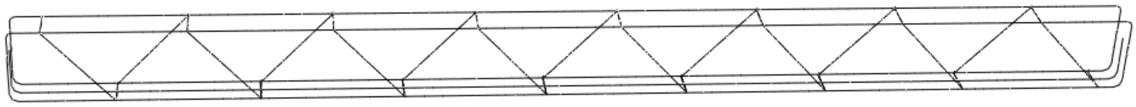


Figure B. 14: 75^0 Rectangular spiral bars provision in 4m span length of RC beam

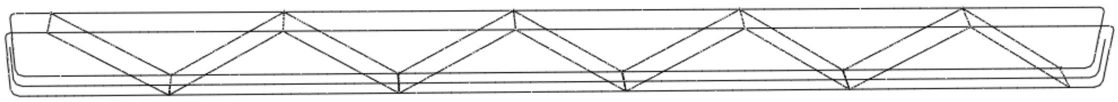


Figure B. 15: 45^0 Warren truss reinforcement's provision in 4m span length of RC beam

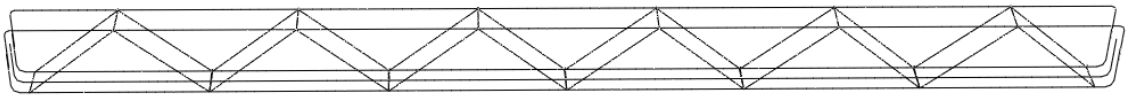


Figure B. 16: 60^0 Warren truss reinforcement's provision in 4m span length of RC beam

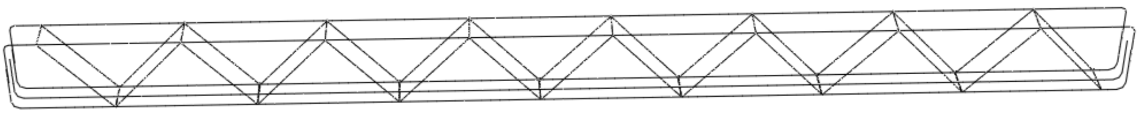


Figure B. 17: 75^0 Warren truss reinforcement's provision in 4m span length of RC beam

Design summary for reinforced concrete beam with 6m span length

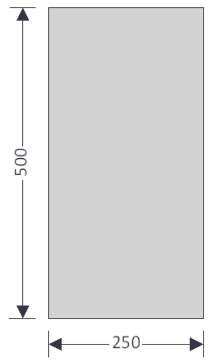


Figure B. 18: RC beam cross-section for 6m span length

Moment Design

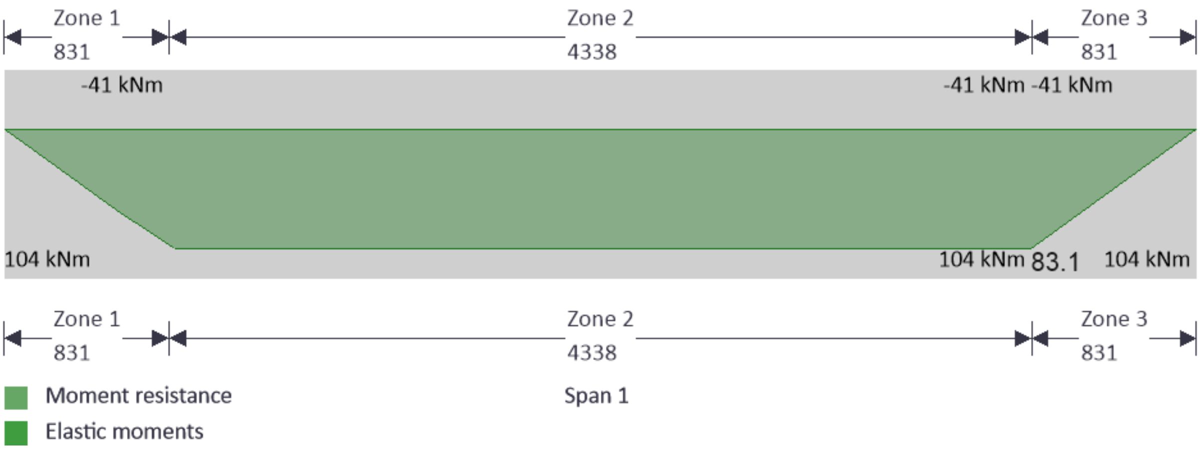


Figure B. 19: Moment resistance and elastic moments result for 6m span length

Table B. 8: Determination of the RC beams shear span in 6m span length

Defined Parameters	Values
Height of the beam cross-section,h (mm)	500
The required nominal concrete cover, cnom(mm)	30
Width of the supporting element (mm)	300
Top Longitudinal Reinforcement diameter(mm)	10
Bottom Longitudinal Reinforcement diameter(mm)	16
Shear Reinforcement diameter(mm)	8
Effective depth of the beam cross-section using top diameter(mm)	457
Effective depth of the beam cross-section using bottom diameter(mm)	454
Clear Span of the beam(mm)	5700
a1(mm)	250
a2(mm)	150

the minimum value from a1 and a2(mm)	150
The effective span, l_{eff} of a member (mm)	6000
The minimum shear span, av(mm)	227
The maximum shear span, av(mm)	908
The shear span between top and bottom center of support plates (mm)	831

Table B. 9: Moment design summary for 6m span length

Defined Parameters	**Zone 1 (0 mm - 831 mm) Positive moment - section 6.1**	**Zone 1 (0 mm - 831 mm) Negative moment - section 6.1**	**Zone 2 (831 mm - 5169 mm) Positive moment - section 6.1**	**Zone 3 (5169 mm - 6000 mm) Positive moment - section 6.1**	**Zone 3 (3446 mm - 4000 mm) Negative moment - section 6.1**
PL(KN)	100	100	100	100	100
M(kNm)	83.1	20.8	83.1	83.1	20.8
d	454	457	454	454	457
δ	1	1	1	1	1
k	0.054	0.013	0.054	0.054	0.013
K'	0.207	0.207	0.207	0.207	0.207
Checking compression	No Required	No Required	No Required	No Required	No Required
Z(mm)	431.2	434.15	431.23	431.23	434.15
x(mm)	57	57	57	57	57
$As_{,req}$ (mm^2)	482	120	482	482	120
reinforcement provided;	3 x 16ϕ	3 x 10ϕ	3 x 16ϕ	3 x 16ϕ	3 x 10ϕ
$As_{,prov}$ (mm^2)	603	236	603	603	236
$As_{,min}$(mm^2)	186	187	186	186	187
$As_{,max}$	5000	5000	5000	5000	5000
Checking	Pass	Pass	Pass	Pass	Pass
Crack control - Section 7.3					

wk	0.4	0.4	0.4	0.4	0.4
Es(Mpa)	200000	200000	200000	200000	200000
$f_{ct,eff}(N/mm^2)$	2.9	2.9	2.9	2.9	2.9
kc	0.4	0.4	0.4	0.4	0.4
k	1	1	1	1	1
s_{bar}(mm)	79	82	79	79	82
σ_s (N/mm^2)	360	360	360	360	360
αcr	6.09	6.09	6.09	6.09	6.09
y(mm)	245.111	248.03	245.111	245.111	248.03
$Act(mm^2)$	61278	62008	61278	61278	62008
$Asc,min(mm^2)$	197	200	197	197	200
Checking	Pass	Pass	Pass	Pass	Pass
R_{PL}	0.65	0.65	0.65	0.65	0.65
$\sigma sr(mm^2)$	208	132	208	208	132
sbar,max(mm)	290.5	300	290.5	290.5	300
Checking	Pass	Pass	Pass	Pass	Pass
ρm0			0.00548		
ρ_m			0.00425		
ρ^'m			0		
Kb			1		
$span\_to\_depth_{basic}$			24.322		
Ks			1.36		
F1			1		
F2			1		
$span\_to\_depth_{allow}$			33.08		
$span\_to\_depth_{actual}$			13.216		
Checking			Pass		
Minimum bar spacing (Section 8.2)					
s_{bar}(mm)		72	72		72
stop,min(mm)		25	25		25

Checking		Pass	Pass		Pass
s_{bot}(mm)	63		63	63	
sbot,min(mm)	25		25	25	
Checking	Pass		Pass	Pass	

Shear Design

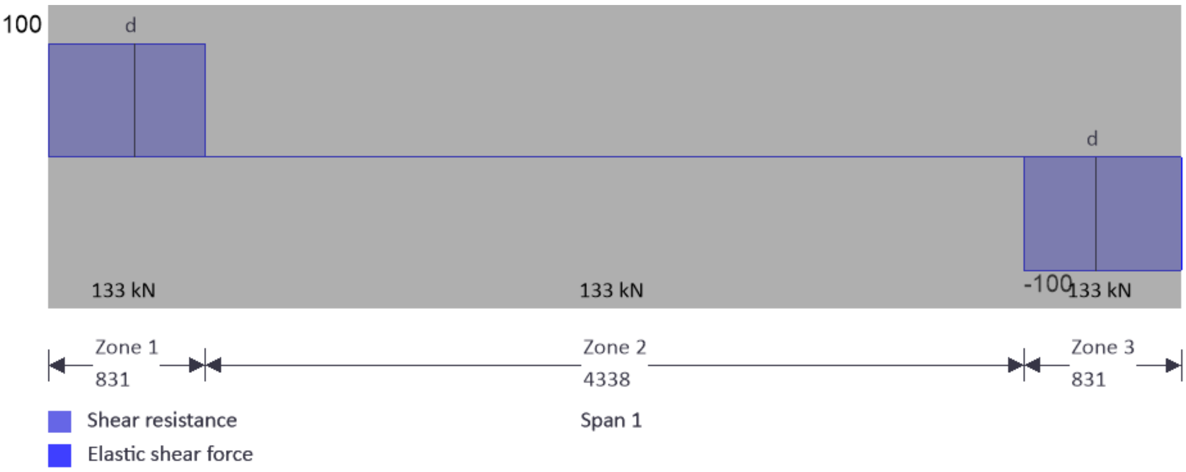


Figure B. 20: Shear resistance and elastic shear force result for 6m span length

The Shear design along Zone 1 (0 mm - 831 mm), Zone 2 (831 mm - 5169 mm) and Zone 3 (5169 mm - 6000 mm) using shear - section 6.2 shown in the table below

Table B. 10: Shear design for 6m span length

Defined Parameters	Vertical Stirrups (90º)	Inclined Stirrups (45º)	Inclined Stirrups (60º)	Inclined Stirrups (75º)
θmax (deg)	45	45	45	45
v1	0.528	0.528	0.528	0.528
αcw	1	1	1	1
Asv,min	238	169	147	143
$V_{Ed,max}$ (kN)	100	100	100	100
z	431.2	431.2	431.2	431.2
V Rd,max(kN)	569	1138	899	723
Checking	Pass	Pass	Pass	Pass
VEd	100	100	100	100
v_{Ed}(N/mm^2)	0.9276	0.9276	0.9276	0.9276
θ (deg)	21.8	21.8	21.8	21.8

Asv,des(mm^2/mm)	232	165	202	225
Asv,req(mm^2/mm)	238	169	202	225
2 x 8 legs @ c/c	325	575	525	425
Asv,prov(mm^2/mm)	309	175	191	237
Checking	Pass	Pass	Pass	Pass
svl,max	341	681	538	432
Checking	Pass	Pass	Pass	Pass

Table B. 11: moment design reinforcement areas results for 6m span length

	Unit	Provided	Required	Utilisation	Result
Zone 1 top (0.0 kNm)	mm^2	236	200	0.847	PASS
Zone 1 bot (83.1 kNm)	mm^2	603	482	0.799	PASS
Zone 2 top (0.0 kNm)	mm^2	236	0	0.000	PASS
Zone 2 bot (83.1 kNm)	mm^2	603	482	0.799	PASS
Zone 3 top (0.0 kNm)	mm^2	236	200	0.847	PASS
Zone 3 bot (83.1 kNm)	mm^2	603	482	0.799	PASS
Span to depth ratio		13.22	33.14	0.399	PASS

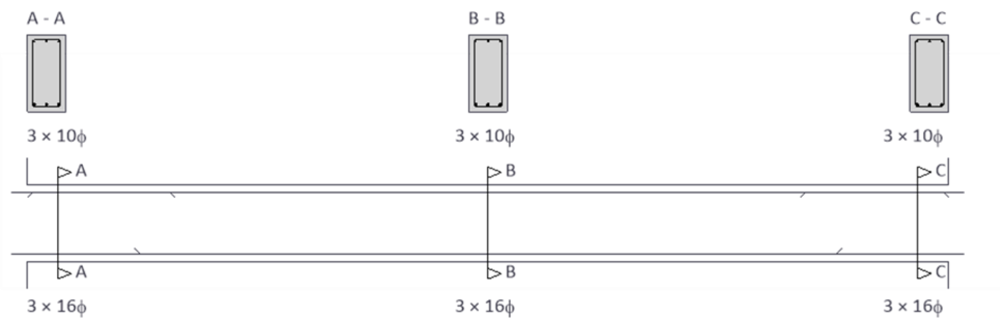


Figure B. 21: The top and bottom reinforcement provision result for 6m length span

Table B. 12: Shear design reinforcement areas results for 6m length span

	Unit	Provided	Required	Utilisation	Result
Zone 1 (100.0 kN)	mm^2/m	309	238	0.770	PASS
Zone 2 (100.0 kN)	mm^2/m	309	238	0.770	PASS
Zone 3 (100.0 kN)	mm^2/m	309	238	0.770	PASS

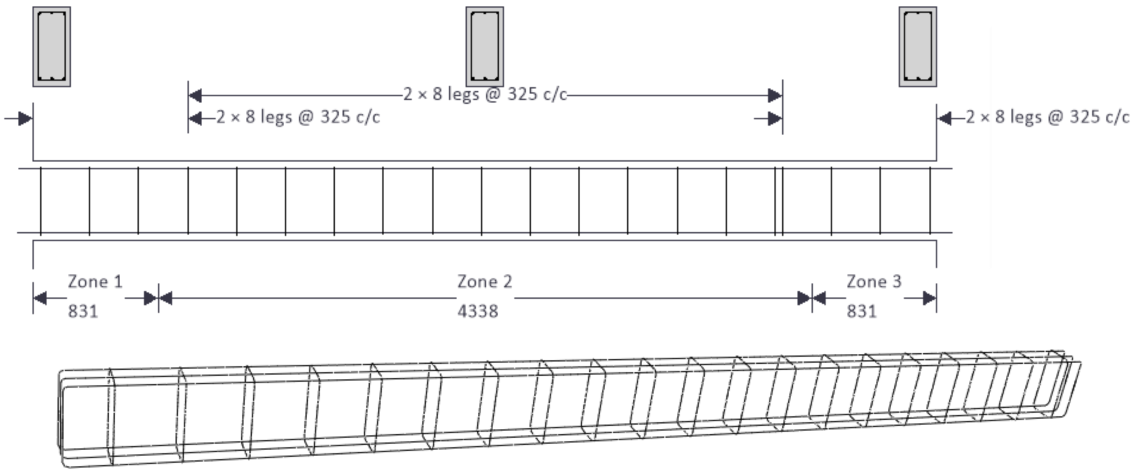


Figure B. 22: Vertical stirrups provision in 6m span length of RC beam

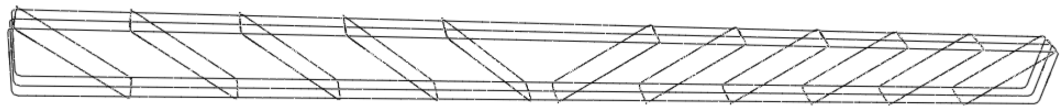


Figure B. 23: 45^0 inclined stirrups provision in 6m span length of RC beam

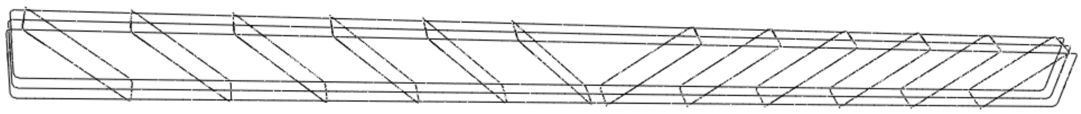


Figure B. 24: 60^0 inclined stirrups provision in 6m span length of RC beam

Figure B. 25: 75^0 inclined stirrups provision in 6m span length of RC beam

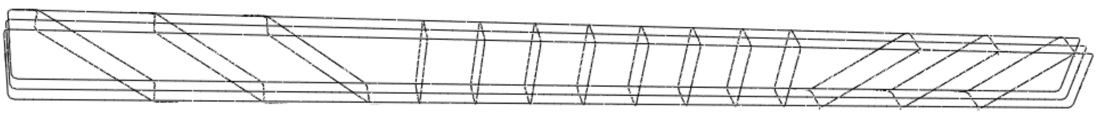


Figure B. 26: 45^0 Swimmer bars provision in 6m span length of RC beam

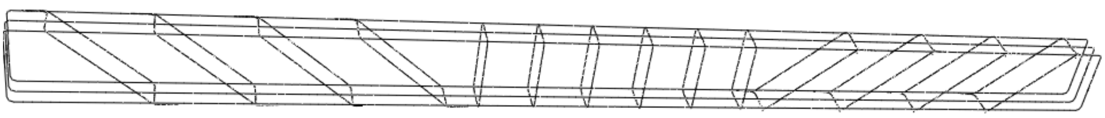


Figure B. 27: 60^0 Swimmer bars provision in 6m span length of RC beam

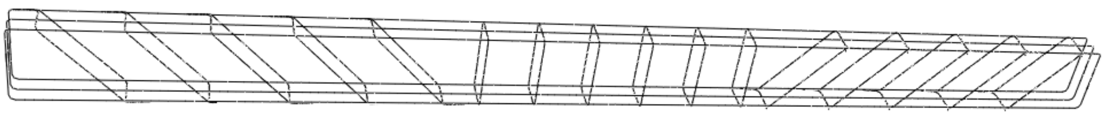


Figure B. 28: 75^0 Swimmer bars provision in 6m span length of RC beam

Figure B. 29:45^0 Rectangular spiral bars provision in 6m span length of RC beam

Figure B. 30: 60^0 Rectangular spiral bars provision in 6m span length of RC beam

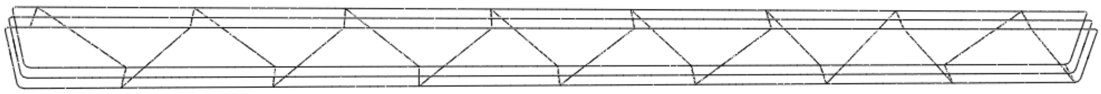


Figure B. 31: 75^0 Rectangular spiral bars provision in 6m span length of RC beam

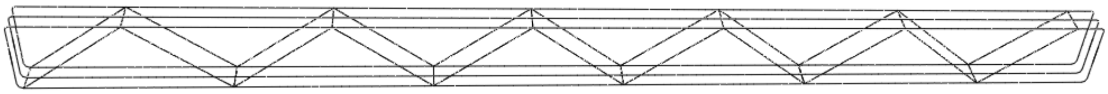


Figure B. 32: 45^0 Warren truss reinforcement's provision in 6m span length of RC beam

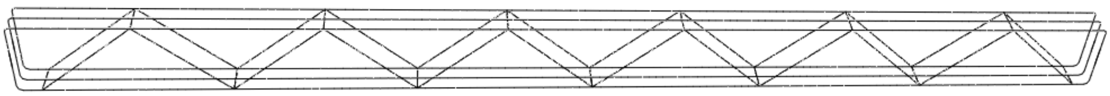


Figure B. 33: 60^0 Warren truss reinforcement's provision in 6m span length of RC beam

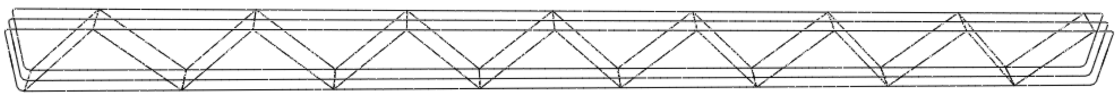


Figure B. 34: 75^0 Warren truss reinforcement's provision in 6m span length of RC beam