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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Effect of Load Distribution on Shear Resistance of Haunched Beam: Experimental Study.

A Thesis in Structural Engineering

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A Thesis

Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science

The undersigned have examined the thesis entitled '**Effect of Load Distribution on Shear Resistance of Haunched Beam: Experimental Study.**' presented by **Betremariam Tesfaye**, a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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UNDERTAKING

I certify that research work titled "Effect of Load Distribution on Shear Resistance of Haunched Beam: Experimental Study" is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged/referred.

Betremariam Tesfaye

ABSTRACT

Major parameters that have an influence on the shear strength of beams without shear reinforcements identified by many researchers are shear span to depth ratio (a/d), tensile strength of concrete (ft), longitudinal reinforcement ratio (ρ), size of beam and presence of axial force. Effects of load distribution and tapered/variable depth of beam has not given great concern in many studies as well as in building design codes. Results of experimental tests on twelve shear critical beam specimens were presented to study whether the shear strength of beam is affected by type of load distribution as well as tapered/variable depth. The specimens are comprised of six types of beam namely the PC(prismatic beam subjected to central concentrated), HUC(haunched up beam subjected to central concentrated load), HDC(haunched down beam subjected to central concentrated load), PU(prismatic beam subjected to two point load), HUU(haunched up beam subjected to two point load) and HDU(haunched down beam subjected to two point load). Two types of load distribution have been taken into consideration; the former three types of beams are subjected to central concentrated load and the later three types of beam are subjected to two-point loads. The varied/tapered shape under consideration is haunched beam. Two types of haunched beams are studied; the haunched up beam and the haunched down beam. Each type of beam is casted and tested twice for the perfection of the study. All specimens have the same size h (200 mm), same shear span to depth ratio a/d (2.75) and reinforcement ratio. The study showed that type of loading has significant effect on the shear resistance of haunched beams. The type of haunches also has a significant effect on shear resistance.

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NOTATIONS

A_s	Area of longitudinal reinforcement
b, b_w	The width of the beam
DIN	Deutsche Institute for Norms
EBCS	Ethiopian Building Code Standard
ES EN	Ethiopian Standard based on Euro Norm
f_{ck}	Characteristic cylindrical concrete compressive strength
f_{cu}	Characteristic cubic concrete compressive strength
f_{yd}	Design yield stress of the reinforcement
f_{yk}	Characteristic tensile strength of the longitudinal reinforcement
HDC	Haunched Down Concentrated (Haunched Down beams subjected to Concentrated load)
HDU	Haunched Down Uniform (Haunched Down Uniform beams subjected to Uniformly distributed load)
HUC	Haunched Up Concentrated (Haunched Up beams subjected to Concentrated load)
HUU	Haunched Up Uniform (Haunched Up Uniform beams subjected to Uniformly distributed load)
M_{max}	Maximum moment capacity
P	Point load applied on a beam
PC	Prismatic Concentrated (Prismatic beams subjected to concentrated load)
PU	Prismatic Uniform (Prismatic beams subjected to Uniformly distributed load)
V_{max}	Maximum shear capacity

CHAPTER 1 INTRODUCTION

1.1 General background

Some special structures such as bridge decks and slabs were built using a beam having variable depth which are usually known as a tapered or haunched beam. But there are few international design codes such as the Mexican 2004 and the German DIN 1045 2001 that gives some guidance on variable depth structures. (Nghiep,2011)

Structures such as retaining walls, earth-covered structures, footings, silos or the support regions of some tapered slab bridges and deck slabs of girder bridges are usually without shear reinforcements and in practice, they are subjected to distributed loading, in many cases their design is governed by shear. (Caldentey et al.,2012)

Research has been done on beams subjected to distributed loading. However, they mostly focused on simply supported beams of constant thickness. Some research has been done on tapered members without shear reinforcement. However, it discusses mostly with beams subjected to point loadings.

1.2 Significance

The design of structural members without shear reinforcement subjected to distributed loading is governed by shear. The design formulas of many codes of practice, however, are based on tests of constant thickness beams subjected to point loading.

For empirical models based on such an experimental background, this shows that some effects significantly influencing shear strength may not be properly accounted for. This may result in the placement of unnecessary shear reinforcement or, potentially, to unsafe designs.

This study presents the result of test series on slender haunched beam that allows a direct comparison between the shear strength of members subjected to central concentrated load or to a two-point load for both constant thickness (prismatic) or haunched beams.

1.3 Objectives

1.3.1 General Objectives

- To study the shear strength and load-carrying capacity of different slender haunched beams subjected to central concentrated and two-point loading arrangements.

1.3.2 Specific Objectives

- To study the significant effect of type of loading on shear strength of slender haunched beams without shear reinforcement.
- To study the significant effect of type of haunches on shear strength of slender haunched beams without shear reinforcement.
- Comparison of the load-carrying capacity of different slender haunched beams.
- Comparison of the load-carrying capacity of slender haunched beam under different types of loading.

1.4 Scope

This research will include only normal slender prismatic and haunched beams subjected to central-concentrated load and two-point loads which result in the internal force of flexure and shear. Deep beams, beam-columns, pre-stressed beams and beams made of high strength concrete will not be covered in this document.

1.5 Methodology

The overall process of this research includes a literature review, designs of beam, examinations of building codes and experimental program.

Firstly, the previous works by different authors on the effect of variable/tapered depth beams and works on the effects of load distribution were reviewed. Basic shear design notes were studied and checked if variable/tapered depth of beams, as well as the loading types, affects the shear strength of beams. Then the most widely used building codes were reviewed for their shear design provisions. Slender beams for central concentrated loading and for two-point loading were designed and the specimens for the experimental program were decided.

After the desk analysis is finished, the experimental investigation was conducted on specimens selected to meet the objective. Finally, the results obtained from the experiment were discussed and compared with the desk studies and the conclusions and recommendations were drawn.

CHAPTER 2 LITERATURE REVIEW

2.1 Haunched beams

Haunched beams are a beam whose cross-section is thicker at the supports than in the middle of the span. They are a 1D members varies in cross-section along the length of the 1D member. These 1D members may contain some parts a constant cross-section and the remaining parts a haunch. Therefore, there are many types of haunched beam parameters.

Haunched beams are one form of non-prismatic beams who have not the same cross-sectional properties from one end to the other and having reinforcements over parts of length and do not have a straight axis. The most common forms of structural members that are non-prismatic have haunches that are either stepped or tapered or parabolic in shape. Non-prismatic concrete beams can provide steel and concrete savings when used to replace equivalent strength prismatic elements. (Jolly and Vijayan 2016)

The non-prismatic members having varying depths are frequently used in the form of haunched beams. The cross-section of the beam can be made non-prismatic by varying width, depth, or by varying both depth and width continuously or discontinuously along their length. Variation in width causes difficulty in construction. Therefore, beams with varying depth are generally provided. Either the soffit or top surface of the beam can be inclined to obtain varying cross-section, but the former practice is more common. The soffit profile may have triangular or parabolic haunches. The effective depth of such beams varies from point to point and the internal compressive and tensile stresses resultants are inclined. It makes the analysis of such beams slightly different from prismatic beams.

2.1.1 Reinforced concrete haunched beams

Reinforced concrete haunched beams (RCHBs) are used in cantilever retaining wall, framed buildings, simply supported and continuous bridges for economic and aesthetic reasons.

They favor more efficient use of materials to clear a given span or to provide a reasonably clear height for the stories of buildings. They provide the following advantages with respect to prismatic beams under lateral loading:

- More efficient use of concrete and steel reinforcement,
- The weight of the building can be reduced for a given lateral stiffness,
- Eases the placement of different facilities or equipment (electrical, air conditioning, sewage, etc.)
- Aesthetic reasons.

The main disadvantages of RCHBs are that special formwork and qualified workers are needed to build them. However, as workmanship is relatively cheap in developing countries, mid-rise haunched beam framed buildings are often built.

2.2 Effect of load distribution on shear resistance of beams

Shear tests on specimens without shear reinforcements and having the same maximum shear to maximum moment ratio were done at the University of Texas at Austin. Half of the specimens are subjected to the central concentrated load and the other half specimens were subjected to uniformly distributed load. The results from the experiment shows that the shear strength of beams is affected by the load distribution. Beams subjected to uniformly distributed load had an average increase in first diagonal cracking shear capacity when compared to the beams subjected to central concentrated load. (Dassow,2014)

Shear tests were performed on slender beams without stirrups on the specimens having two types of effective beam depth, one having double effective depth than the other. The specimens were subjected to central concentrated loads and uniformly concentrated loads to study the effect of load distribution as the effective depth of the beams increase. The ratio between the maximum shear to the maximum bending moment is maintained to the same to make directly comparable tests. The results from the experiment show that load distribution has an effect on the shear strength of beams. Beams subjected to central concentrated loads have averagely increase in shear strength at first diagonal cracking than beams subjected to uniformly distributed loads. (Klein,2015)

2.3 Effect of variable depth on shear resistance of beams

Simply supported haunched and prismatic reinforced concrete beams which are designed to develop a shear failure under static loading were studied in two groups. The first group is the beams with shear reinforcement and another group is beams without shear reinforcement satisfying the minimum required for prismatic beams by the concrete norms of Mexico's Federal District Code (MFDC). From the test, the conclusion had made as haunched beams have a different behavior with respect to prismatic beams. Haunched beams tend to an arching action in the haunched length as the main resisting failure mechanism. (Tena-Colunga,et al,2007).

The shear behavior of reinforced concrete beams of varying depth compared to prismatic beams was investigated experimentally under the same loading conditions by varying some parameters such as main longitudinal reinforcement, presence of stirrups, depth of beams at supports and the slope of the beams. From the study conclusion has made as the effect of the slopes at the top surface is less than that of beams having slopes at the bottom faces as compared to the rectangular beams. (Stefanou,1983)

Experimental studies have been done on beams of constant depth and beam having linearly varying depths along the shear span to study the behavior and the shear strength of haunched beams. From the tests and the proposed formula conclusion has been made as beams of different haunch inclinations has no significant change of load capacity at initial cracks. Haunched beams having smaller depth at the support did not suffer large reduction in ultimate shear strength and haunched beams having large depth at the support did not have a significant increase in ultimate shear strength. (Debaiky and Elniema,1982)

2.4 Shear resistance of beams.

A beam resists loads primarily by means of internal moments, M, and shears, V. In the design of a reinforced concrete member, flexure is usually considered first, leading to the size of the section and arrangement of reinforcement to provide the necessary moment resistance. Limits are placed on the amounts of flexural reinforcement which can be used to ensure that if failure were ever to occur, it would develop gradually, giving warning to the occupants. The beam is then proportioned for shear. Because a shear failure is frequently sudden and brittle, as suggested by the damage sustained by the building. The design for shear must ensure that the shear strength equals or exceeds the flexural strength at all points in the beam.

The manner in which shear failures can occur varies widely depending on the dimensions, geometry, loading, and properties of the members. For this reason, there is no unique way to design for shear.

2.4.1 Factors affecting the shear strength of beams without web reinforcement

Beams without web reinforcement will fail when inclined cracking occurs or shortly afterward. For this reason, the shear capacity of such members is taken equal to the inclined cracking shear. The inclined cracking load of a beam is affected by five principal variables, some included in design equations and others not (MacGregor and Bartlett, 2000).

2.4.1.1 Tensile strength of concrete.

The inclined cracking load is a function of the tensile strength of the concrete. The stress state in the web of the beam involves biaxial principal tension and compression stresses. A similar biaxial state of stress exists in a split cylinder tension test, and the inclined cracking load is frequently related to the strength from such a test. The flexural cracking which precedes the inclined cracking disrupts the elastic stress field to such an extent that inclined cracking occurs at a principal tensile stress, based on the un-cracked section, of roughly a third of fct.

2.4.1.2 Longitudinal reinforcement ratio, ρ.

The shear capacities of simply supported beams without stirrups as a function of steel ratio, ρ =As/bwd. The practical range of ρw for beams developing shear failures is about 0.0075 to 0.025.

$$V_{cu} = 0.167\sqrt{f'_c}b_w d \qquad (2\text{-}1)$$

2.4.1.3 Shear span-to-depth ratio, a/d.

The shear span-to-depth ratio, a/d or M/Vd, has some effect on the inclined cracking shears and ultimate shears of shear spans with a/d less than 2. Such shear spans are "deep" shear spans (D-regions). For longer shear spans where B-region behavior dominates, a/d has little effect on the inclined cracking shear and can be neglected.

2.4.1.4 Size of beam.

As the overall depth of a beam increases, the shear stress at inclined cracking tends to decrease for a given f'c, ρ, and a/d. As the depth of the beam increases, the crack widths at points above the main reinforcement tends to increase. This leads to a reduction in aggregate interlock across the crack, resulting in earlier inclined cracking.

2.4.1.5 Axial forces.

Axial tensile forces tend to decrease the inclined cracking load, while axial compressive forces tend to increase it. As the axial compressive force is increased, the onset of flexural cracking is delayed and the flexural cracks do not penetrate as far into the beam. As a result, a larger shear is required to cause principal tensile strength of the concrete.

2.4.2 Shear resistance of slender beams.

2.4.2.1 Truss model of the behavior of slender beams falling in shear

The behavior of beams failing in shear must be expressed in terms of a mechanical-mathematical model before designers can make use of this knowledge in design. The best model for beams with web reinforcement is the truss model. This is applied to slender beams and deep beams (MacGregor and Bartlett, 2000).

The truss analogy for the design of reinforced concrete for shear was independently published by the Swiss engineer Ritter and The German engineer Morsch independently.

These procedures provide an excellent conceptual model to show the forces that exist in a cracked concrete beam.

As shown in figure 2-1, a beam with inclined cracks develops compressive and tensile forces, C and T, in its top and bottom "flanges," vertical tensions in the stirrups and inclined compressive forces in the concrete "diagonals" between the inclined cracks.

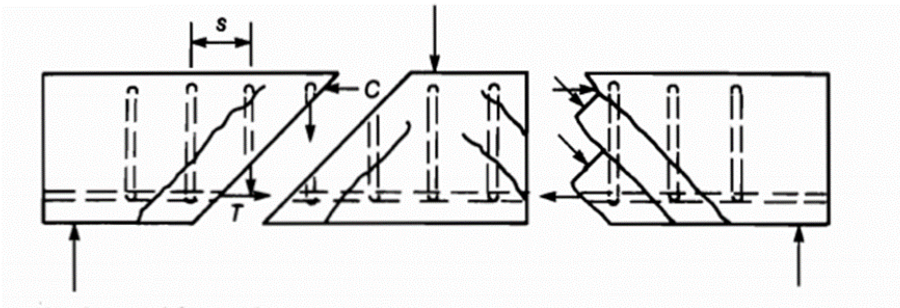


Figure 2-1: Internal forces in a cracked beam.
(Source: MacGregor and Bartlett, 2000 figure 6-20 a)

This highly indeterminate system of forces is replaced by an analogous truss. The simplest truss is shown in figure 2-2.

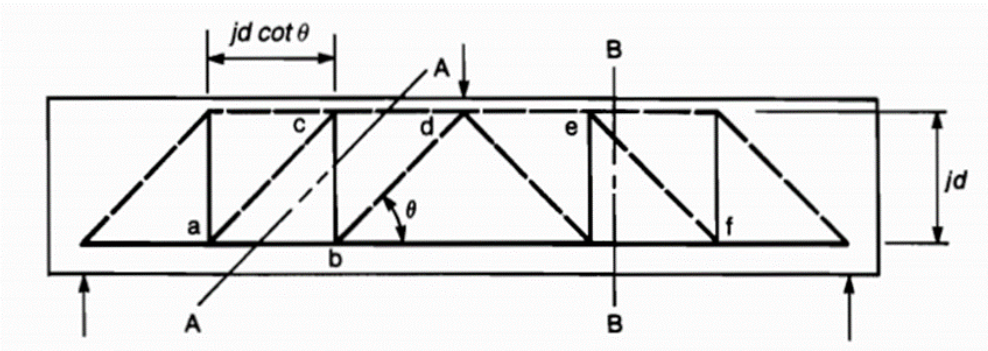


Figure 2-2: Pin ended truss.
(Source: MacGregor and Bartlett, 2000 figure 6-20 b)

2.4.3 Shear resistance of haunched beams.

2.4.3.1 Tapered beam

(MacGregor and Bartlett, 2000) In a prismatic beam, the average shear stress between two cracks is calculated as

$$\upsilon = \frac{v}{b_w \cdot j \cdot d} \tag{2-2}$$

Which is simplified to

$$\upsilon = \frac{v}{b_w \cdot d} \tag{2-3}$$

In the derivation of Eq. 6-4 of MacGregor and Bartlett, 2000, here Eq. 2.2 from it was assumed that jd was constant. If the depth of the beam varies, the compressive and/or tensile forces due to flexure will have vertical components.

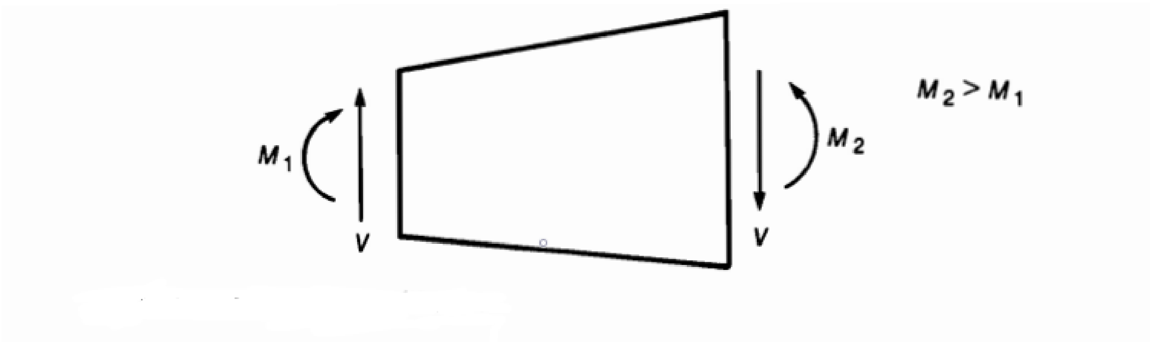


Figure a: forces on segments of a beam

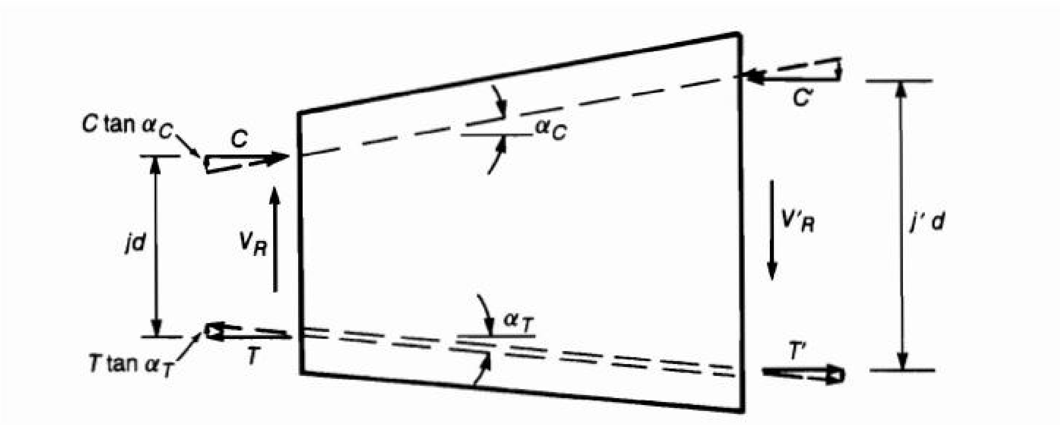


Figure b: Internal forces and shears

Figure 2-3: Reduced shear force in non-prismatic beam

(Source: MacGregor and Bartlett, 2000 figure. 6-51)

A segment of a tapered beam is shown in figure 2.3. The moment, M, at the left end of the section can be represented by two horizontal force components, C and T, separated by the lever arm jd. The tension force actually acts parallel to the centroid of the reinforcement and hence has a vertical component T tan α_T, where α_T is the angle between the tensile force and the horizontal. Similarly, the compressive force acts along a line joining the centroids of the stress blocks at the two sections and hence has a vertical component $C\ tan\ \alpha_C$. The shear force,V_f, on the left end of the element can be represented as

$$V_f = V_R + C\tan\alpha_c + T\tan\alpha_T \qquad (2\text{-}4)$$

Where V_R is the reduced shear force resisted by the stirrups and the concrete. Substituting $C = T = M/jd$ and letting $\alpha = \alpha_C + \alpha_T$

$$V_R = V_f - \frac{\|M\|}{jd}\tan\alpha \qquad (2\text{-}5)$$

Where $\|M\|$ represents the absolute value of the moment and α is positive if the lever arm $j_d$increases in the same direction as $\|M\|$ increases. The shear stresses in a tapered beam then become

$$v = \frac{V_R}{b_w d} \qquad (2\text{-}6)$$

Examples of the use of equation (6-50) on MacGregor and Bartlett, 2000, here equation (2-5) are shown in figure 2-4.

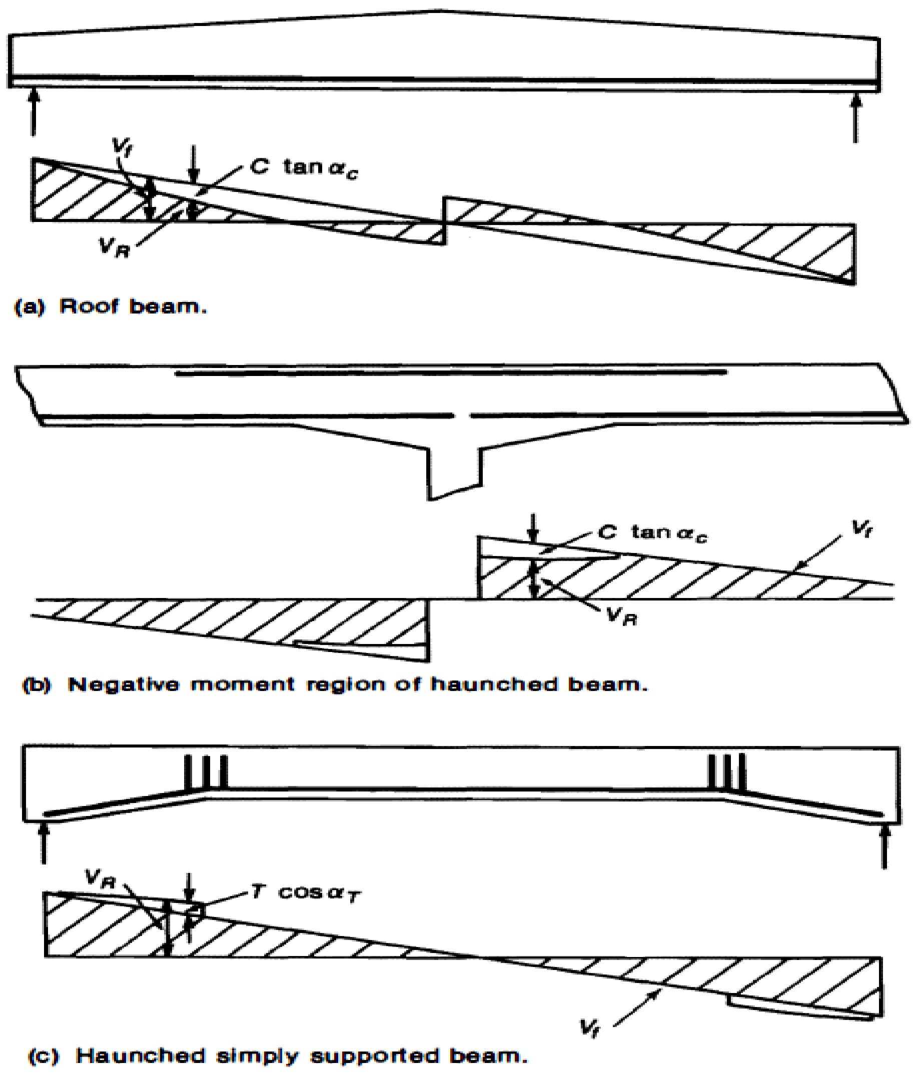


Figure 2-4: Examples of computation of V_R

(Source: MacGregor and Bartlett, 2000, figure 6-52)

Haunched up beams are used in circumstance where large depth at the support is required such as bridges or in circumstances where space in the middle span is required for functions such as lighting. Haunched up beams are the most widely used types of haunching. Haunched down beams may be used in circumstances such as smaller depth at the support is required.

2.5 Code revision on the shear strength/resistance of slender beams having variable/tapered depth

2.5.1 Din 1045 2001 07 Section 10.3.2 (4)

(DIN-1045-1,2001) In members that are of variable effective depth or have inclined tendons, the design shear force, taking into account the shear component in the tension and compression flanges normal to the member axis, shall be obtained by means of this equation:

$$V_{ED} = V_{Edo} - V_{ccd} - V_{td} - V_{pd}$$

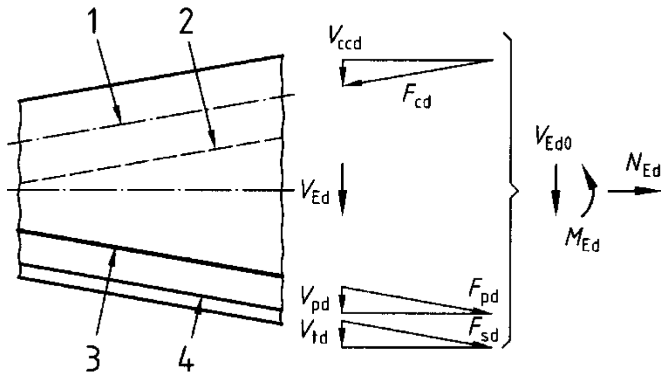


Figure 2-5: Shear in cross-sections of variable effective depth

Source: (DIN-1045-1,2001)

Key:

1 Line of action of compressive force in the concrete

2 Line of zero action

3 Centroid of tendons

4 Centroid of reinforcement

In the figure 2-5,

$\boldsymbol{V_{Ed}}$ is the design shear force;

$\boldsymbol{V_{Edo}}$ is the basic design shear force effective in the cross-section;

V_{ctd} is the design shear component in the compression zone;

V_{td} is the design shear component in the tension zone (reinforcement)

V_{pd} is the design shear force component in the prestressing steel at the ultimate limit state; however, the condition $P_{mt} \leq A_p.f_{pO,1k}/\gamma_s$ is to be satisfied.

2.5.2 Eurocode 2 2004 (Section 6.2.1)

From EN 1992-1-1: 2004, (EN, 2004) For the verification of the shear resistance the following symbols are defined:

$V_{Rd,c}$ is the design shear resistance of member without shear reinforcement.

$V_{Rd,s}$ is the design value of the shear force which can be sustained by the yielding shear reinforcement

$V_{Rd,\ max}$ is the design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts.

In the members with inclined chords, the following additional values are defined

V_{ccd} is the design value of the shear component of the force in the compression area, in the case of an inclined compression chord.

Vtd is the design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord.

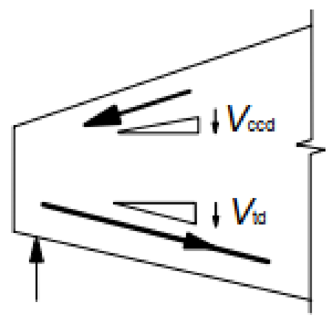


Figure 2-6: Shear component for members with inclined chords (source EN, 2004)

The shear resistance of a member with shear reinforcement is equal to:

$$\mathbf{V_{Rd} = V_{Rd,s} + V_{ccd} + V_{td}}$$

In regions of the member where $V_{Ed} \leq V_{Rd,c}$ no calculated shear reinforcement is required; minimum shear reinforcement should nevertheless be provided.

CHAPTER 3 BEAM DESIGN

Rectangular Beam Design

$$d = 157\ mm \qquad d_2 = 25 + 8 + 8 = 41mm$$

$$f_{ck} = 31.5\ MPa \quad f_{yk} = 574\ MPa \quad f_{cd} = 17.85\ MPa \quad f_{yd} = 499.13\ MPa$$

The cross-section is designed to carry a concentrated load of P =150 kN

3.1 Beam design for central concentrated load test

Shear span a = the distance from the concentrated load point up to the support or zero moment.

The desired shear span to effective depth ratio is

$$\frac{a}{d} = 2.75 \qquad a = 2.75 * d = 2.75 * 157 = 431.75mm$$

The support hangout is taken as 100 mm.

Self-weight = 0.2m*0.2m*25kN/m^3 =1 kN/m

Moment on the beam due to self-weight $\frac{wl^2}{8} = \frac{(1\frac{\mathrm{kN}}{\mathrm{m}})(0.8635)^2}{8} = 0.0932kNm$

Max moment due to concentrated load $M_{max} = \frac{PL}{4} = \frac{(150kN)*(0.8635m)}{4} = 32.38kNm$

$$M_{sd} = 0.0932kNm + 32.38kNm = 32.47kNm$$

3.2 Beam design for two-point load test

Shear span (a) in this case it can’t be the distance from the load to the support.

For simply supported beam

$$M_{max} = \frac{PL}{8}\ and\ V_{max} = \frac{P}{2}$$

Shear span (a) = $\frac{M_{max}}{V_{max}} = \frac{PL}{8} / \frac{P}{2} = \frac{L}{4}$

$$\frac{a}{d} = 2.75 \qquad \frac{\frac{L}{4}}{d} = 2.75$$

$$L = 2.75 * d * 4 = 2.75 * 157mm * 4 = 1727mm$$

$$support\ hang\ out\ taken\ as\ 150mm$$

$$The\ maximum\ moment\ due\ to\ two\ poin\ loading$$

$$M_{max} = \frac{PL}{8} = \frac{(150kN)(1.727m)}{8} = 32.38\ kNm$$

Self-weight = 0.2m*0.2m*25kN/m^3 =1 kN/m

Moment on the beam due to self-weight $\frac{wl^2}{8} = \frac{(1\frac{\text{kN}}{\text{m}})(0.8635)^2}{8} = 0.0932kNm$

$$M_{sd} = 0.0932kNm + 32.38kNm = 32.47kNm$$

For both of the cases, the cross-section is designed for the same design moment.

USING EBCS-EN-1992-1-1-2013 Design Table (Design Table for C12/15-C50/60)

(EBCS,2013)

$$\mu_{sd,lim} = 0.295\ (for\ 0\%\ moment\ redistribution)$$

$$\mu_{sd} = \frac{M_{sd}}{fcd * b * d^2} = \frac{32.47 * 10^6 N.mm}{17.85\frac{N}{mm^2} * 200\ mm * 157mm^2} = 0.369$$

$$\mu_{sd} > \mu_{sd,lim}$$

Therefore, the section is doubly reinforced section.

From the table for 0% moment redistribution

$$\mu_{sd,lim} = 0.295 \qquad \omega_{,lim} = 0.363 \qquad k_{x,lim} = 0.448$$

$$k_{z,lim} = 0.814 \qquad \varepsilon_{c,lim} = 3.5 \qquad \varepsilon_{s1,lim} = 4.313$$

$$\omega' = \frac{\mu_{sd,s} - \mu_{sd,lim}}{\left(1 - \frac{d_2}{d}\right)} = \frac{(0.369 - 0.295)}{\left(1 - \left(\frac{41}{157}\right)\right)} = 0.10016$$

Area of tension reinforcement

$$A_{s1} = (\omega_{,lim} + \omega')bd\frac{f_{cd}}{f_{yd}} = (0.363 + 0.10016) * 200 * 157 * \frac{17.85}{499.13} = 520.1\ mm^2$$

Check whether the compression reinforcement has yielded

$$\varepsilon_{s,2} = \left(1 - \frac{d_2/_d}{k_x}\right)\varepsilon_c = \left(1 - \frac{\frac{41}{157}}{0.448}\right) * 3.5 = 1.46$$

$$\varepsilon_{yd} = \frac{f_{yd}}{E_s} = \frac{499.13}{200000} = 2.496$$

$$\varepsilon_{s,2} < \varepsilon_{yd}$$

$$\sigma_{s2} = \varepsilon_{s,2}E_s = 1.46 * 10^{-3} * 200 * 10^3 = 292\ MPa$$

Calculate the stress in concrete at the level of compression reinforcement to avoid double counting of area.

$$\varepsilon_{s,2} = 1.46 < \varepsilon_2 = 2$$

$$\sigma_{cd,s2} = f_{cd}\left(1 - \left(1 - \frac{\varepsilon_{cs,2}}{\varepsilon_{c,2}}\right)^n\right) = 17.85 * \left(1 - \left(1 - \frac{1.46}{2}\right)^2\right) = 16.55\ MPa$$

Area of compression reinforcement

$$A_{s2} = \omega' bd \frac{f_{cd}}{(\sigma_{s2} - \sigma_{cd,s2})} = 0.10016 * 200 * 157 * \frac{17.85}{(292 - 16.55)} = 203.81mm^2$$

This will become using $\phi 16$ for compression and $\phi 20$ for tension at the mid-span region

$$\text{No. of tension reinforcement } = \frac{520.1\, mm^2}{314\, mm^2} = 1.66 \approx 2\phi 20$$

$$No. of\ compression\, reinforcement = \frac{203.81}{200.96} = 1.01 \approx 2\phi 16$$

To be more confident of shear failure the flexure capacity was increased. This is done by increasing the number of reinforcements in the tensile region. The cross-section used for the experimental program was provided 2 diameter 16 mm bars in the compression region. In the tension region, 1 additional bar of diameter 16 mm is added and the number of bars in the tension region is 2 diameter 20 mm and 1 diameter 16 mm.

3.3 Check the minimum shear reinforcement to decide "without shear reinforcement"

- The minimum shear reinforcement using EBCS-EN-1992-1-1-2013

$$\rho_{w,\min} = \frac{0.08\sqrt{f_{ck}}}{f_{yk}} \tag{3-1}$$

$$\rho_{w,\min} = \frac{0.08\sqrt{31.5}}{574} = 0.00078228$$

$$\rho_w = \frac{A_{sw}}{s * b_w * \sin\alpha} \tag{3-2}$$

Where:

ρ_w is the shear reinforcement ratio

ρ_w should not be less than **$\rho_{w,\min}$**

A_{sw} is the area of shear reinforcement within length **s**

s is the spacing of the shear reinforcement measured along the longitudinal axis of the member

b_w is the breadth of the web of the member

For the central-concentrated beam

$$A_{sw} = \frac{2 * 4 * 3.14 * 3^2}{3} = 75.36\, mm^2$$

$$s = \frac{863.5}{3} = 287.83\, mm$$

$$b_w = 200\, mm \qquad \alpha = 90^o$$

$$\rho_w = \frac{A_{sw}}{s * b_w * sin\alpha}$$

$$\rho_w = \frac{75.36\, mm^2}{287.83\, mm * 200\, mm * 1} = 0.00131$$

For the two-point beam

$$A_{sw} = \frac{2 * 4 * 3.14 * 4^2}{3} = 133.97\, mm^2$$

$$s = \frac{1727}{3} = 575.67\, mm$$

$$b_w = 200\, mm \qquad \alpha = 90^o$$

$$\rho_w = \frac{133.97\, mm^2}{575.67\, mm * 200\, mm * 1} = 0.00116$$

Excluding the stirrup at the support for both central-concentrated beam and for the two-point beam $\rho_{w,min} < \frac{0.08 * \sqrt{f_{ck}}}{f_{yk}}$, satisfying the requirement of without shear reinforcement.

CHAPTER 4 EXPERIMENTAL INVESTIGATION

4.1 General

This chapter describes the experimental program completed at the construction materials laboratory, Addis Ababa Institute of Technology to study the effect of load distribution and variable depth on the shear resistance of slender beams without stirrups.

Twelve reinforced concrete specimens were tested under six load configurations (each load configuration was tested using two specimens). The investigated parameters were:

- Influence of load distribution (central concentrated loading and two-point loading); and
- Influence of variable depth (haunches)

The six types of load configuration were named as follows:

- PC1/PC2: Prismatic beam with central concentrated load;
- PU1/PU2: Prismatic beam with two-point load;
- HUC1/HUC2: Haunched up beam with central concentrated load;
- HUU1/HUU2: Haunched up beam with two-point load;
- HDC1/HDC2: Haunched down beam with central concentrated load;
- HDU1/HDU2: Haunched down beam with two-point load.

4.2 Test specimens

To avoid bending failure prior to shear failure increased amount of reinforcement consisting of mild reinforcement steel with a nominal characteristic yield stress of 500 MPa is used as tensile reinforcement. To avoid anchorage failures, sufficient anchorage length was provided. The reinforcement of all specimens was identical, consisting of two 20 mm and one 16 mm bars on the tensile face. On the compression face, two 16 mm bars consisting of mild reinforcement steel with a nominal characteristic yield stress of 500 MPa were placed. Concrete cover for all specimens was 25 mm with an effective depth d at supports equal to 157 mm and reinforcement ratio $\rho = As/(bw.d)$ equal to 4.64% for all specimens.

The geometry of the specimens was selected so that the center of gravity of the applied loads remains the same in all cases, with a constant ratio between the resultant of the applied forces and the effective depth (a/d) equal to 2.75.

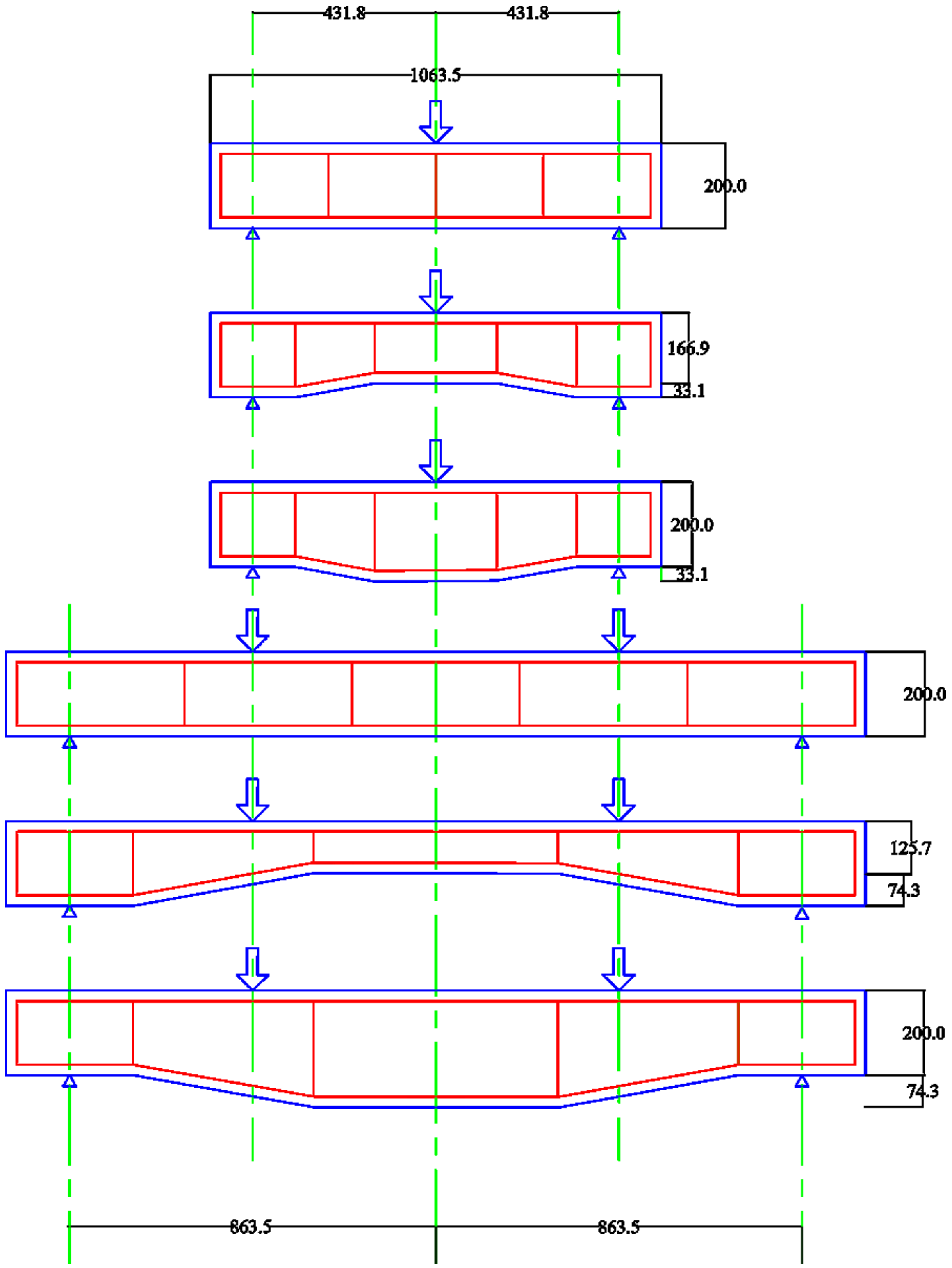


Figure 4-1: The test specimens

4.3 Material property

4.3.1 Concrete property.

Table 4-1: The cube test values

CUBE	STRESS (MPa)	LOAD (kN)
MIX-1	49.82	1121.2
MIX-1 OUT	42.11	947.6
MIX-1	39.66	892.4
Mix 1 Average	**43.86**	**987.06**
MIX-2	43.78	985.1
MIX-2	39.93	898.6
MIX-2 OUT	31.57	710.4
Mix 2 Average	**38.43**	**864.7**
MIX-3	35.8	805.4
MIX-3 OUT	35.34	795.1
MIX-3	36.4	819
Mix 3 Average	**35.85**	**806.5**

The term "OUT" in the table denotes the cube that maintained out of water tank and placed in the same condition as the beam specimens.

Mix 1 Average = (49.82+42.11+39.66)/3 = 43.86 **MPa** **Cylindrical = 34.29 MPa**

Mix 2 Average = (43.78+39.93+31.57)/3 = 38.43 **MPa** **Cylindrical = 30.89 MPa**

Mix 3 Average = (35.8+35.34+36.4)/3 = 35.85 **MPa** **Cylindrical = 29.28 MPa**

Overall Average = (43.86+38.43+35.85)/3 = 39.38 **MPa** **Cylindrical = 31.49 MPa**

4.3.2 Reinforcement property

Table 4-2: Reinforcement test results from lab

Specimen NO.	D1 (mm)	D2 (mm)	Yield Load (kN)	Failure Load (kN)	Elongation (%)	Mass/Length kg/m	Remark
8-1	734	864	23.2	28.9	4.9	280	73.5
8-2	726	849	23.1	29.2	4.8	285	73.1
8-3	742	854	23.3	29.2	4.9	281	73.3
Average	734	855.67	23.2	29.1	4.87	282	73.3
16-1	1536	1721	112.7	134.6	9.9	1395	89
16-2	1544	1763	114.8	129.5	9.2	1390	89
16-3	1530	1746	116.4	128.4	9.7	1415	92
Average	1536.7	1743.3	114.6	130.8	9.6	1400	90
20-1	1931	2207	180.3	200.4	12.2	2185	90
20-2	1947	2190	182.1	202.7	12.2	2160	89
20-3	1920	2184	180.1	200.6	12.7	2170	89.9
Average	1932.7	2193.7	180.83	201.2	12.37	2171.67	89.63

Table 3-2 shows the result of reinforcement tensile strength test in the laboratory using Universal Testing Machine.

Table 4-3: Calculation of reinforcement strengths from reinforcement test result

FOR DIA.16	FOR DIA.20	FOR DIA. 8
$a_s = 201.06mm^2$	$a_s = 314.16mm^2$	$a_s = 50.27mm^2$
$f_y = \frac{114.63*10^3 N}{201.06mm^2} = 570.13MPa$	$f_y = \frac{180.83*10^3 N}{314.16mm^2} = 575.6MPa$	$f_y = \frac{23.2*10^3 N}{50.27mm^2} = 461.51MPa$
$f_u = \frac{130.83*10^3 N}{201.06mm^2} = 650.7MPa$	$f_u = \frac{201.23*10^3 N}{314.16mm^2} = 640.53MPa$	$f_u = \frac{29.1*10^3 N}{50.27mm^2} = 578.87MPa$

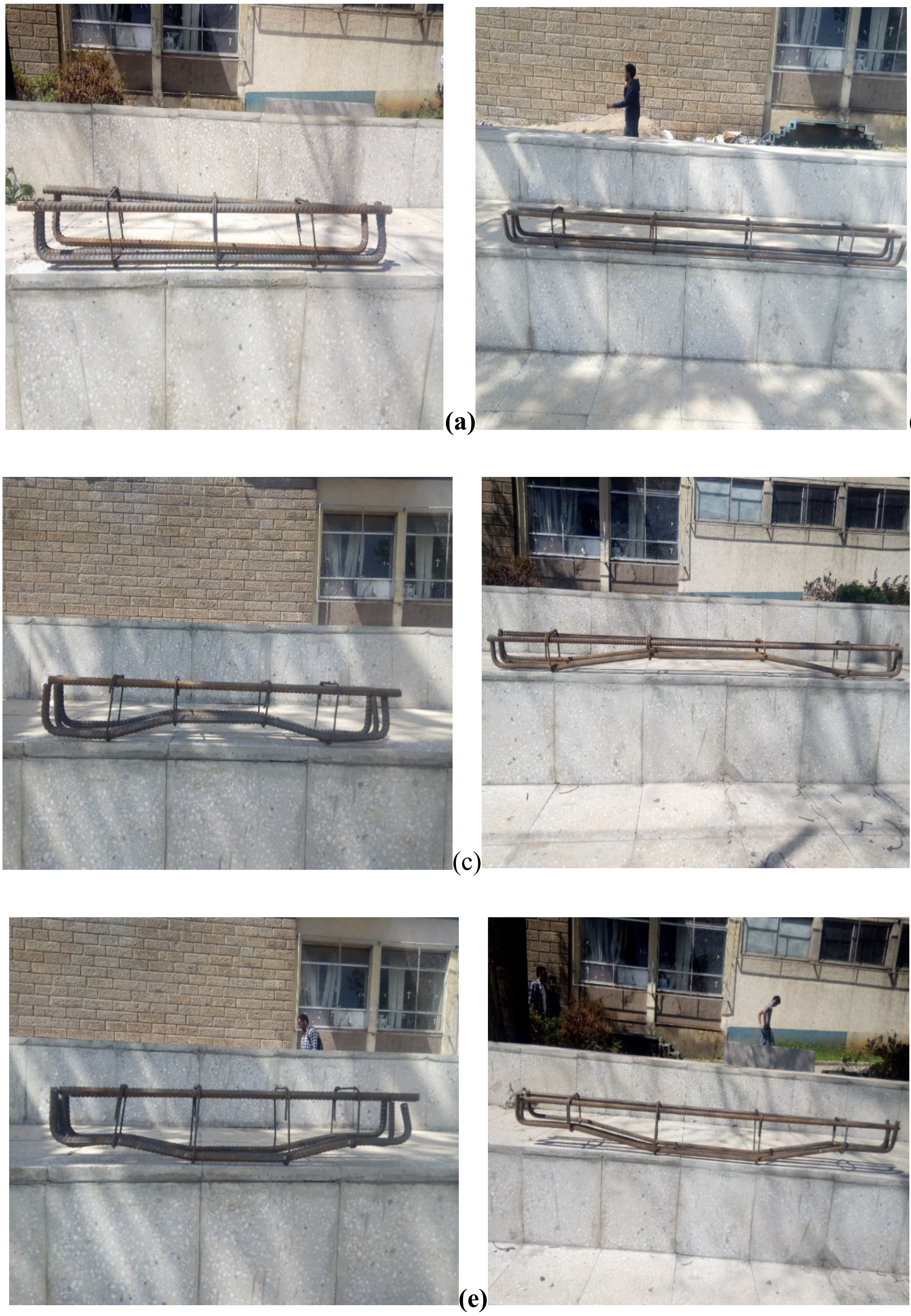
4.4 Specimen fabrication

The material parameters used in the mix design were measured in the laboratory following Abebe Dinkus' construction materials laboratory manual (Dinku,2002). The procedures followed to select and produce concrete mix is provided in Appendix B of this document.

Longitudinal reinforcements were provided according to the analyzed area and determined shape in order to prevent the anchorage failure and produce the required shape of the beam.

The production of formworks was very tedious work especially for the haunched type of beams. Four out of twelve formworks were prismatic and requires lesser carpenters' effort and production time than the rest of eight haunched type formworks. Even the four haunched formworks out of the eight were needed to support the four relatively short haunched beam that will be loaded with concentrated loads needed much effort to accurately provide the required varying shape. Another challenge of non-prismatic formworks was providing bottom support for the beam without losing the intended shape due to the heavyweight of fresh concrete.

Figure 4-2: The formworks



(a) (b) (c) (d) (e) (f)

Figure 4-3: Reinforcement cage for: - a). PC b). PU c). HUC d). HUU e). HDC f). HDU

Spacers were placed between the reinforcement cages and the formwork to provide sufficient space for the concrete cover.

Mixes were mixed three times using a concrete mixer machine to cast all the twelve beams. The three mixes have the same mix proportions and their workability was controlled to be uniform using the slump test. Mix 1 is for prismatic beam, mix 2 is for haunched down beam and mix 3 is for haunched up beam.

Three cubes and three-cylinder samples were taken from each mix to test the compressive and tensile strength respectively of the concrete at the testing date.

The beams were assumed to be without shear reinforcements (stirrups). However, for the purpose of holding the longitudinal reinforcement in place three and four plain dia.6 mm shear reinforcements were provided on the concentrated loaded and uniformly loaded beams respectively.

The side formworks were removed after 24 hours. The soffits were removed after the concrete was cured for 28 days continuously and gain its concrete strength.


(a)


(b)


(c)


(d)


(e)


(f)


(g)


(h)


(i)


(j)


(k)


(l)

Figure 4-4: Beams after removal of formworks: - a) PC1, b) PC2, c) PU1 d) PU2 e) HUC1 f) HUC2, g) HUU1, h) HUU2, i) HDC1, j) HDC2, k) HDU1, l) HDU 2

The load was applied using a heavy vehicle hydraulic jack of maximum capacity 300 kN manually. The test setup was made carefully to prevent sliding of beams, maintain vertical application of the loads, maintain the center of hydraulic jack and load cell to measure the maximum intensity of the load accurately. Gypsum was mixed with water and put between load plates and the beam surface to smoothen the rough surface of the beam and also to fix in place the load plate after it dries.

The surface of the beams was made white and grids were drawn with a space of 50 mm to easily notice the crack dimensions and directions.

Sample of the test setup during testing is shown in the figure 4-6 and 4-7. Three points on each beam (two points on the supports points of axis of rotation and one point at the center of the beam) were drilled to fix the steel frame attached to a transducer to measure the central deflection.

Concentrated load was applied using small point load plates having an area of 50*200 mm^2 and thickness 30 mm. The hydraulic jack was placed at the center of this plate to apply concentrated load.

Uniformly distributed load (Using two-point load) was applied using two-point load plates having an area of 50*200 mm^2 and thickness 30 mm. Two points were placed at a distance that resembles uniformly distributed load effect. The concentrated load coming from the hydraulic jack was distributed to the two-point load plates using strong I section shown in the figure 4-7.

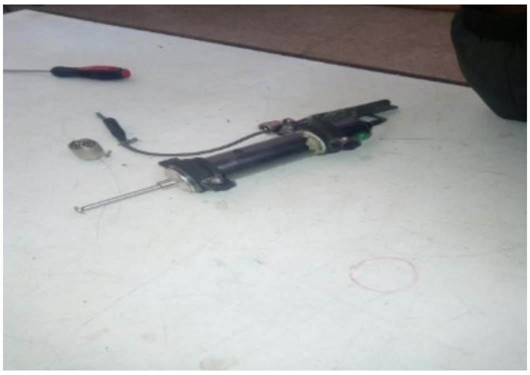


Figure 4-5: Transducer (Electronic deflection measuring instrument)

Figure 4-6: Test set up for concentrated load test

Figure 4-7: Test set up for uniformly distributed (two-point load) load test

4.5 Instrumentation

All beams were fully instrumented to measure the applied loads and deflections on the beams. In brief, the instrumentation consisted of a load cell which measures the applied load, deflection measurement to measure the mid-span deflection. All of the instrumentations were connected to a data logger and the experimental data was directly obtained in a USB Flash Disk.

4.6 Test procedure

The specimens were placed on a roller support above the couple of I sections of the beam testing machine. For the smaller beams, three or four persons were enough to lift the specimens to the beam testing machine. For the bigger one crane was used. The specimens were painted with white color and gridded to 50*50 mm squares on one face to easily observe the crack patterns.

Points of loading were marked on the top surface of the specimen to place the concentrated load plate. On the opposite side of the painted face of the specimen displacement measuring tool was fixed. The steel bar holding the displacement measuring tool was fixed on the two points at the supports that have zero rotation. Small steel plate was fixed to the top mid-point of the specimen where there is maximum deformation.

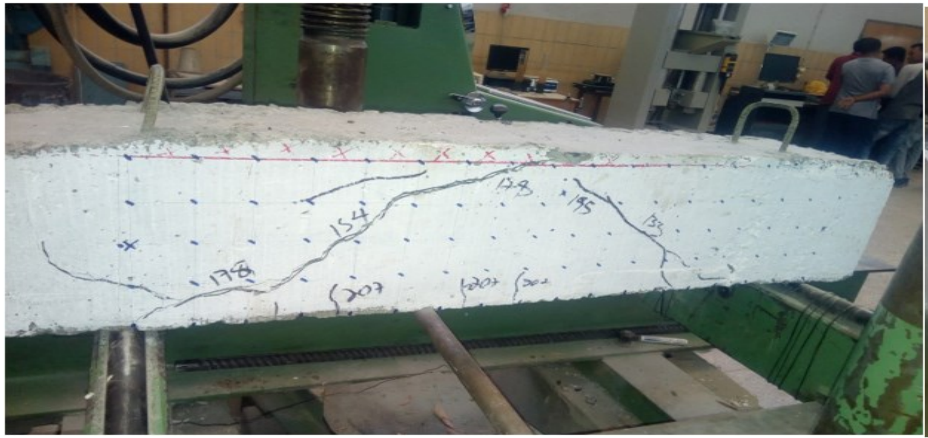
The load was applied using hydraulic jack manually in the range of -5 to 11.25 kN/s loading rate. The load cell above the loading setup and the displacement measuring tool under the small steel plate sends the data to the connected datalogger every second. Pictures and videos were taken using a smartphone camera during the testing of specimens.

CHAPTER 5 RESULTS AND DISCUSSION

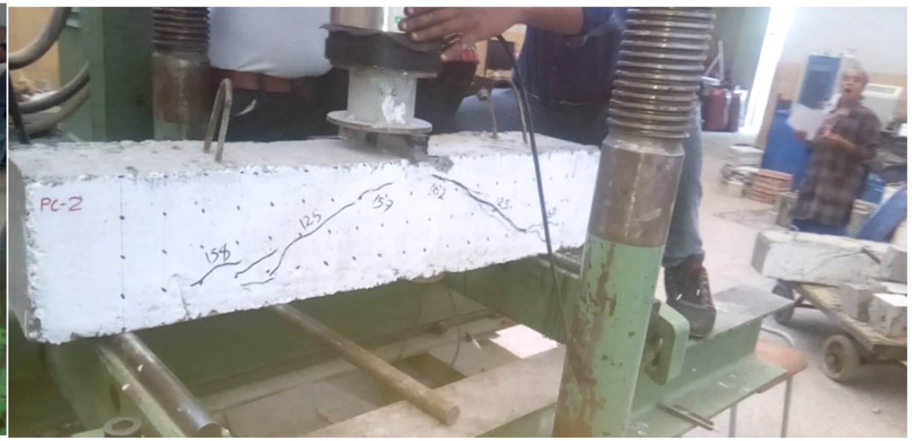
All of the beams were intentionally designed to fail in shear by keeping the moment or flexure capacity of the section of the beam higher than the shear capacity of the section All beams have been failed in diagonal shear failure type. The pictures shown in figure 5-1 shows the photos that have been taken during the test. In all picture, diagonal shear failure was noticed as the shear failure of the beams. The beams subjected to central concentrated load has a clearer crack or diagonal shear failure than the beams subjected to the two-point loading.

The beam type HDC-2 have different post-peak graphs as shown from table 5-1. The graph shows a horizontal straight line in the post-peak region. This result due to the beam cracks before the failure of the beam. This does not create a problem in the study since the shear strength considered in the study is the ultimate shear strength which corresponds to the ultimate load.

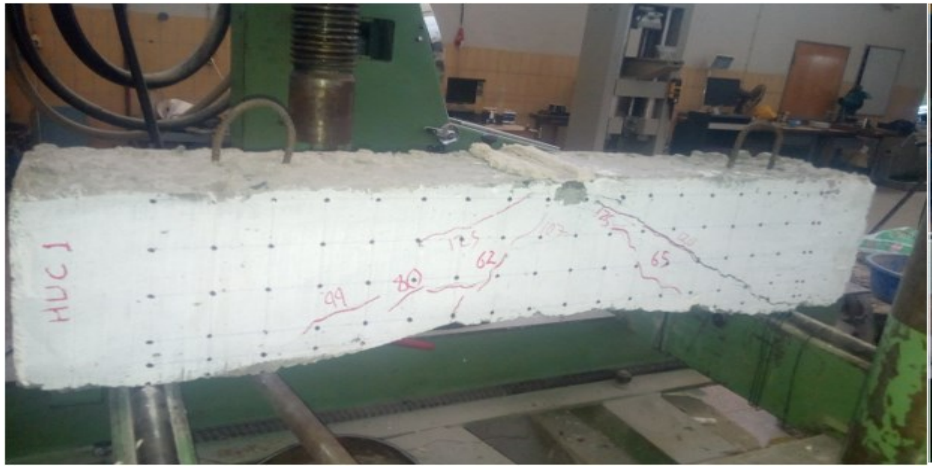
The ultimate load and the corresponding shear strength were used to compare the shear strength of beams of different variable shapes as well as the shear strength of beams subjected to different loading conditions.



PC-1



PC-2



HUC-1

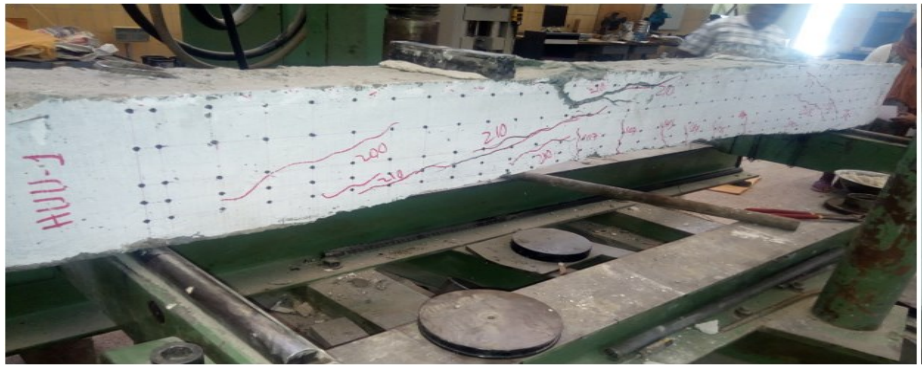
HUC-2

HDC-1

HDC-2

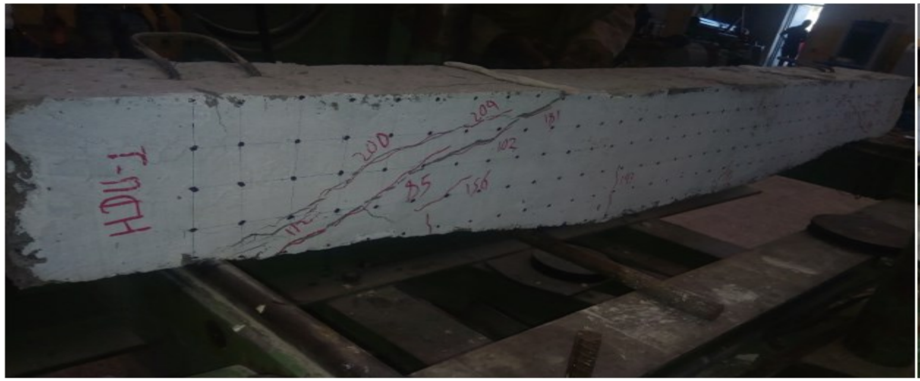
PU-1

PU-2



HUU-1

HUU-2



HDU-1

HDU-2

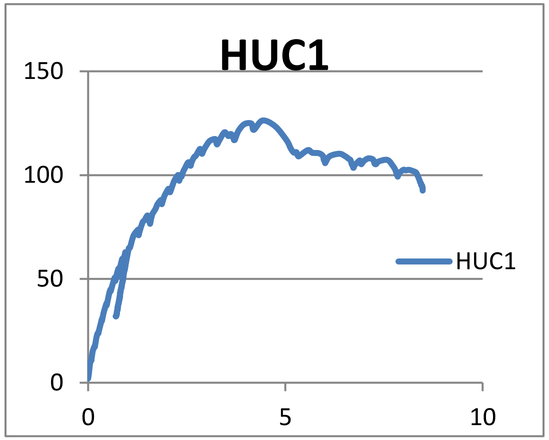
Figure 5-1: Crack patterns of the specimens

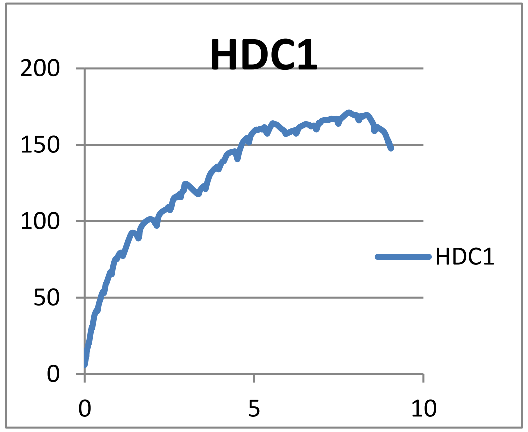
Table 5-1: Load deformation graph from central concentrated load tests

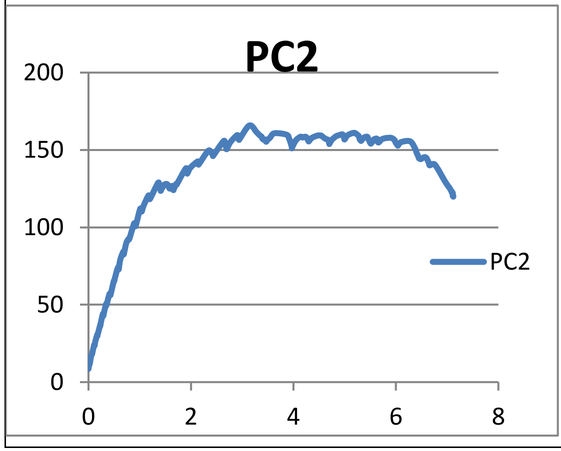
PC1

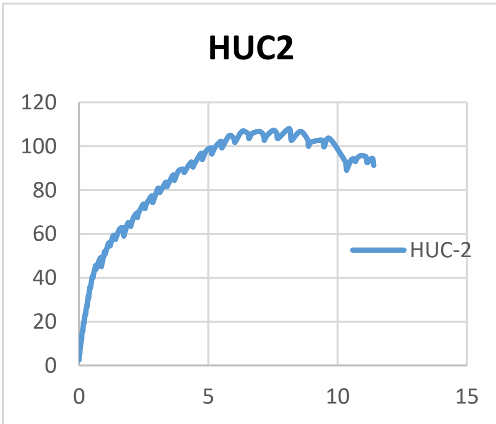
PC1 was missed

The maximum load and the failure load can be obtained from the video









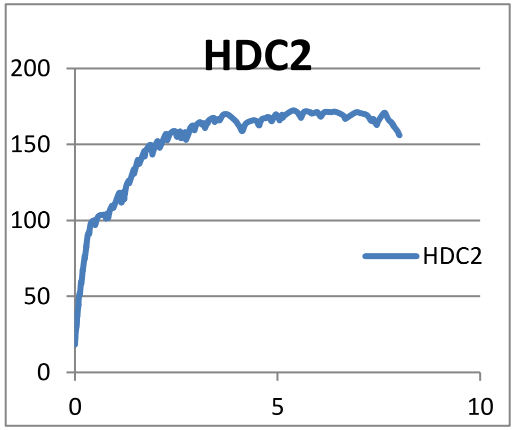
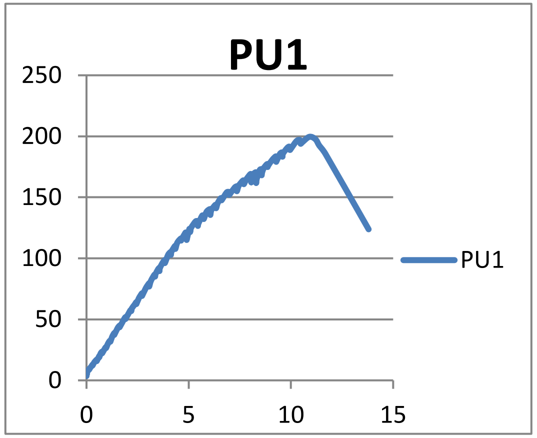
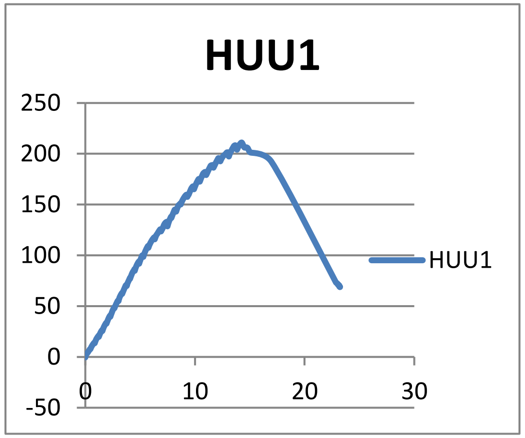
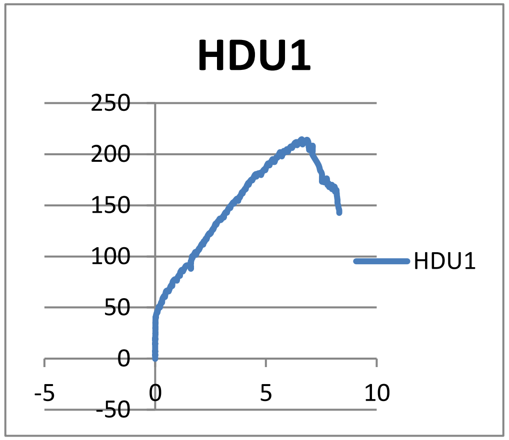
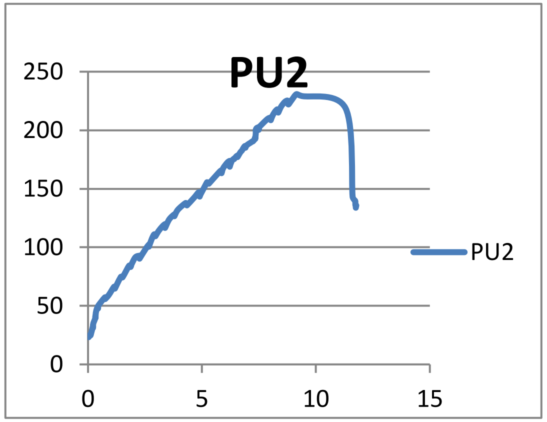


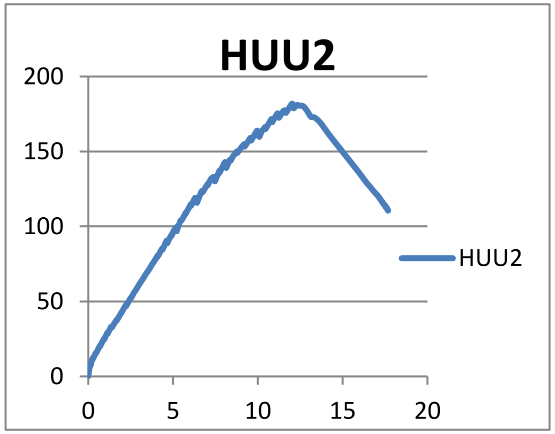
Table 5-2: Load deformation graph from two-point load tests

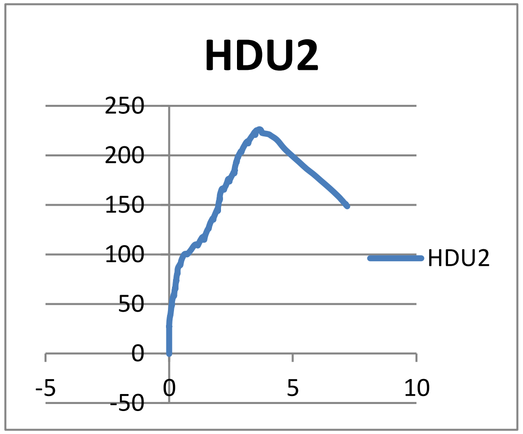












5.1 Results from central concentrated load tests

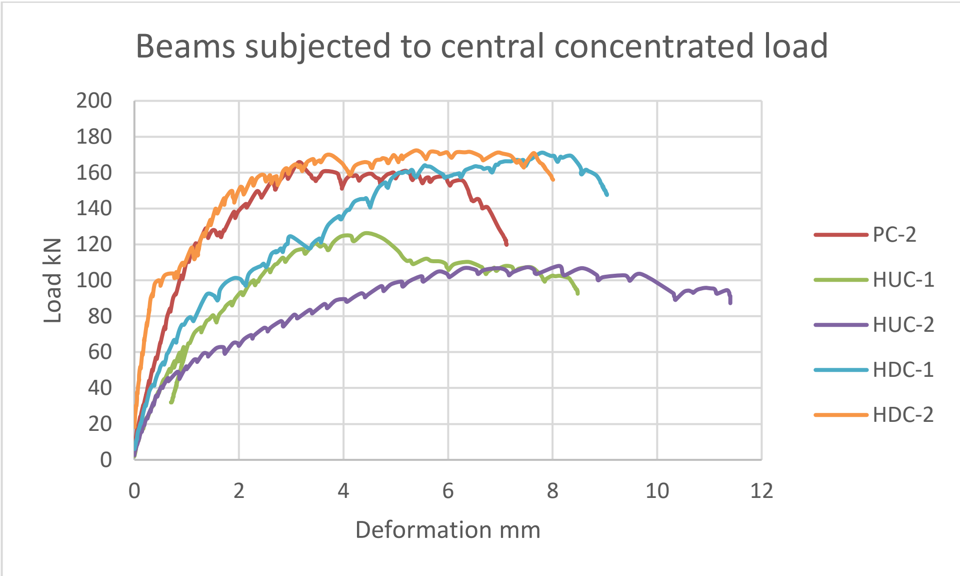


Figure 5-2: Load deformation diagram of specimens subjected to concentrated load

Table 5-3: Peak and ultimate load and deformation from concentrated load tests

	Ultimate load		Failure load	
	Deformation (mm)	Load (kN)	Deformation (mm)	Load (kN)
PC-1		207		
PC-2	3.14	165.82	7.12	119.8
HUC-1	4.42	126.3	8.48	93.79
HUC-2	8.12	108.04	11.4	91.29
HDC-1	7.78	171.07	9.04	148.06
HDC-2	5.37	172.32	8.01	156.06

In the graph plotted in figure 5-2, the results of five beams tested by subjecting to central concentrated load are presented. The values PC-1, an identical beam with PC-2 is lost before saving to the USB flash disk in the material testing laboratory. However, the peak value was obtained from the video and photographs captured during the testing. The values from the graph in figure 5-2 as well as peak values from the table 5-3 show both the haunched up as well as the haunched down beams have lower load-carrying capacity hence lower shear strength. The haunched-up beams instead have higher load-carrying as well as shear strength than over the haunched down beams.

5.2 Results from the two-point load test

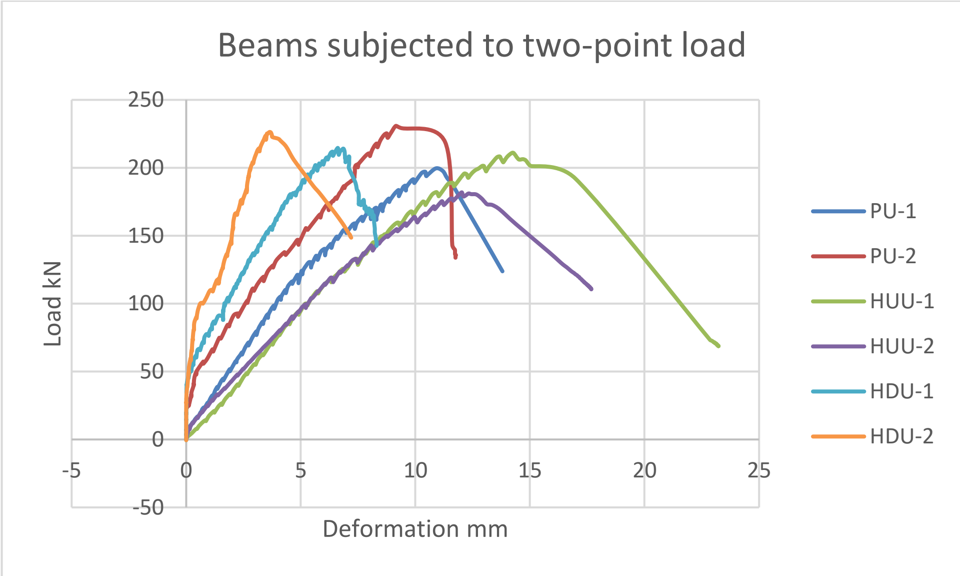


Figure 5-3: Load deformation diagram of specimens subjected to two-point load

Table 5-4: Peak and ultimate load and deformation from two-point load tests

	Ultimate load		Failure load	
	Deformation (mm)	Load (kN)	Deformation (mm)	Load (kN)
PU-1	10.9	199.58	13.8	123.8
PU-2	9.15	230.84	11.77	135.8
HUU-1	14.24	211.08	23.23	68.78
HUU-2	12.01	181.82	17.68	110.79
HDU-1	6.61	214.59	8.31	144.81
HDU-2	3.64	226.34	7.2	148.56

As plotted in the graph in figure 5-3, the load-deformation values of six beams from three types of beams according to the tapering shape are shown. All the six beams were subjected to two-point loading seeking to produce uniformly distributed load behavior. The ultimate and failure load values with their corresponding deformation values were summarized in table 5-4. As seen from the values the prismatic type of beam has the largest load-carrying capacity as well as shear strength than both haunched up and haunched down beams. Haunched down beams have higher load carrying and shear strength than the haunched up beams.

In both cases, in beams subjected to central concentrated load and in beams subjected to two-point load, the prismatic beams have highest load-carrying capacity and shear strength than the haunched up and haunched down beams and the haunched down beams, in turn, have higher load carrying as well as shear strength than the haunched up beams.

5.3 Effects of load distribution

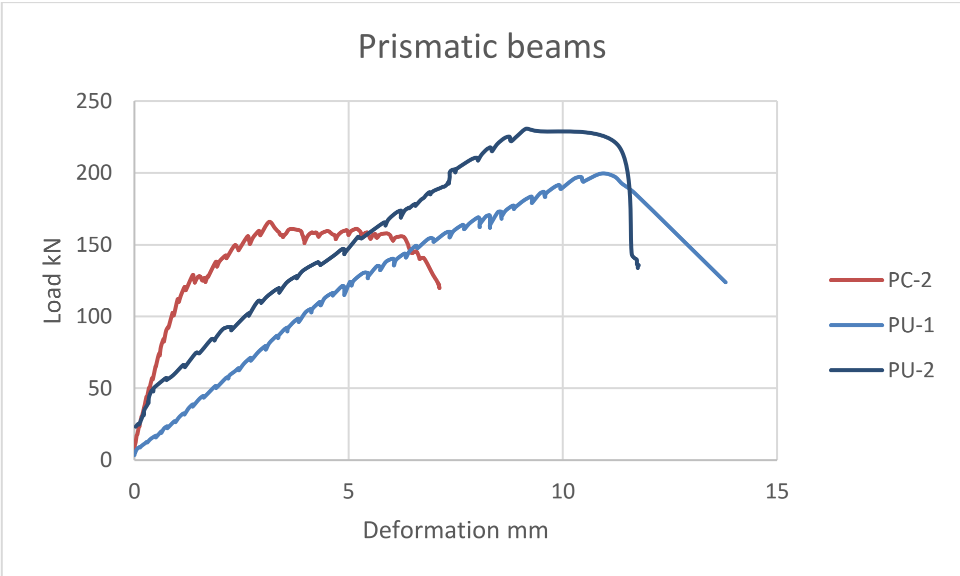


Figure 5-4: Load deformation diagram of prismatic specimens

Table 5-5: Prismatic beams

	Ultimate load		Failure load	
	Deformation (mm)	Load (kN)	Deformation (mm)	Load (kN)
PC-1	no result	207	no result	no result
PC-2	3.14	165.82	7.12	119.8
PU-1	10.9	199.58	13.8	123.8
PU-2	9.15	230.84	11.77	135.8

The values plotted in the graphs in figure 5-4 and tabulated in the Table 5-5 shows the prismatic beams subjected to the two-point load have higher load-carrying capacity as well as shear strength than the prismatic beams subjected to central concentrated load.

Figure 5-5: Load deformation diagram of haunched up specimens

Table 5-6: Haunched up beams

	Ultimate load		Failure load	
	Deformation	Load	Deformation	Load
HUC-1	4.42	126.3	8.48	93.79
HUC-2	5.37	172.32	8.01	156.06
HUU-1	14.24	211.08	23.23	68.78
HUU-2	12.01	181.82	17.68	110.79

The graph plotted in figure 5-5 and the table 5-6 show the values of the load-deformation relation of the haunched-up beam subjected to central concentrated load and two-point load. As shown from the results the load-carrying capacity, as well as the shear strength of the haunched-up beams subjected to two-point loading, was higher than the haunched-up beams subjected to central concentrated load.

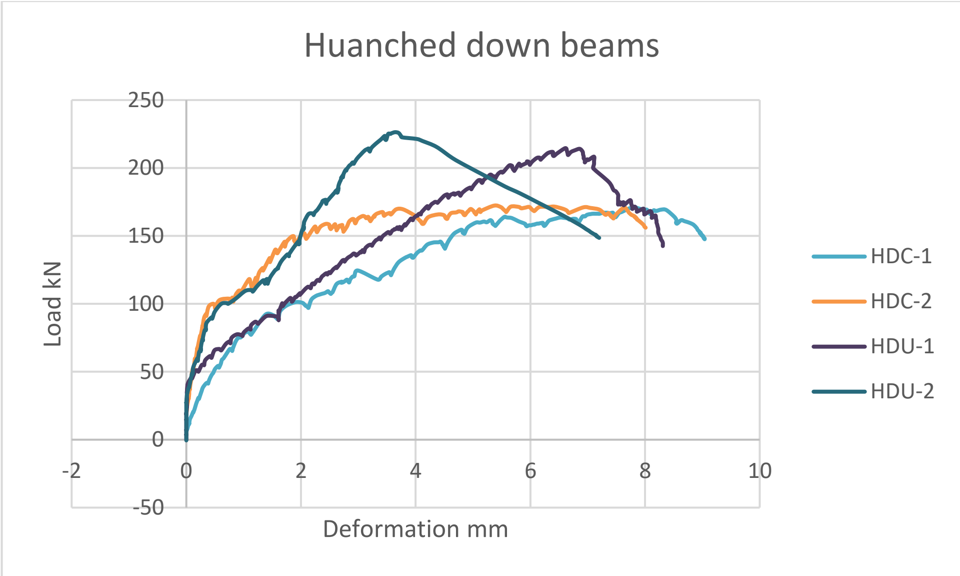


Figure 5-6: Load deformation diagram of haunched down specimens

Table 5-7: Haunched down beams

	Ultimate load		Failure load	
	Deformation	Load	Deformation	Load
HDC-1	7.78	171.07	9.04	148.06
HDC-2	5.37	172.32	8.01	156.06
HDU-1	6.61	214.59	8.31	144.81
HDU-2	3.64	226.34	7.2	148.56

The load-deformation values of the haunched down beams were plotted in the graph plotted in figure 5-6 and the peak values were tabulated in table 5-7. The maximum/ultimate load-carrying capacity of the haunched down beams subjected to the two-point loading have higher load-carrying as well as shear strength than the haunched beams subjected to central concentrated load.

The type of load distribution has a clear effect on the shear strength as shown in the experiment.

The prismatic beam subjected to two-point load (PU) have 20.35 to 39.21% more load-carrying capacity as well as shear strength over the prismatic beam subjected to central concentrated load (PC).

The haunched-up beam subjected to two-point load (HUU) have 43.95 to 67.12% more load-carrying capacity as well as shear strength over the haunched-up beam subjected to central concentrated load (HUC).

The haunched down beam subjected to two-point load (HDU) have 25.43 to 32.30% more load-carrying capacity as well as shear strength over the haunched down beam subjected to central concentrated load (HDC).

Generally, from the discussion above it was observed that the load-carrying capacity, as well as the shear strength of beams subjected to two-point load, is greater than the beams subjected to central concentrated load.

5.4 Effects of variable depth

Haunched/tapered beams have also different shear strength from the prismatic beams as shown from the results of the experiment.

The prismatic beam subjected to central concentrated load (PC) have 31.29 to 63.89% more load-carrying capacity as well as shear strength over the haunched-up beam subjected to central concentrated load (HUC).

The prismatic beam subjected to central concentrated load (PC) have -3.06 to 21.0% more load-carrying capacity as well as shear strength over the haunched down beam subjected to central concentrated load (HDC).

The haunched down beam subjected to central concentrated load (HDC) have 58.33 to 59.49% more load-carrying capacity as well as shear strength over the haunched-up beam subjected to central concentrated load (HUC).

The prismatic beam subjected to two-point load (PU) have 9.77 to 26.96% more load-carrying capacity as well as shear strength over the haunched-up beam subjected to two-point load (HUU).

The prismatic beam subjected to two-point load (PU) have -6.99 to 7.57% more load-carrying capacity as well as shear strength over the haunched down beam subjected to two-point load (HDU).

The haunched down beam subjected to two-point load (HDU) have 18.02 to 24.48% more load-carrying capacity as well as shear strength over the haunched-up beam subjected to two-point load (HUU).

According to the (MacGregor and Bartlett, 2000), V_R decrease as the lever arm jd increases in the same direction as the magnitude of moment increases. For simply supported beams the magnitude of moment increases as one moves from the support to the mid span. From this it is indicated that haunched beams having a lever arm increasing toward a mid-span will have small shear resistance V_R and the haunched beams having a lever arm decreasing towards the support will have a higher shear resistance V_R. This implies that haunched down beams will have lesser shear resistance V_R than haunched up beams.

The German code (DIN-1045-1,2001) shows the contribution of the tensile reinforcement to the design shear force. The design shear component of the tensile reinforcement V_{td} has negative effect for haunched down beams and has positive effect for the haunched up beams. This also implies that haunched down beams has lesser design shear resistance than haunched up beams.

Here in this study the specimens used for comparisons are the prismatic beam, the haunched up beam and the haunched down beam having the same depth at the support where maximum shear force is expected and the results not support the theoretical background. It is important to see if those specimens in the study with the same depth at the maximum moment or with the same depth at the point of load application and the depth varies increasingly or decreasingly to the support to compare with the theoretical background. This may be studied in the further works.

The haunched beam types used in this study are that types of beams having haunching only in the tensile zone of the beam only and the compressive face of the beam is straight. Therefore, the contribution of the tensile reinforcement for the difference in the shear strength is significantly higher than the contribution of the compressive concrete.

The variety in the shear strength capacity comes from the variety of the shear force and bending moment diagrams for the central concentrated load and for two-point loading cases.

CHAPTER 6 CONCLUSIONS AND RECOMMENDATION

6.1 Conclusion

The following conclusions are drawn from the studies on the effect of load distribution on shear resistance of haunched beam.

1. It is concluded that the shear resistance of beams having different tapered/variable shapes in their shear span have different shear resistance. In both groups of beams that have been subjected to central concentrated load in the first group or the beams subjected to two-point load test, which intended to behave as uniformly distributed load behavior, the shear resistance of the three beams differ significantly from each other. In the study, it has been concluded as the prismatic beams have a good shear resistance than either the haunched beams or than haunched down types of beams whether they are subjected to the central concentrated loads or the two-point loads. It is also concluded from the study the haunched down type of beams in both central concentrated and two-point loading case have higher shear resistance than the haunched up beams.
2. Investigating the effect of load distribution on the shear strength of tapered beams subjected to two types of load distribution, central concentrated load, and two-point loading, shows a difference in the shear resistance. Beams subjected to two-point loadings have higher shear resistance than beams subjected to central-concentrated load cases in all types of beam tapering/variety of depths.

6.2 Recommendation

In the process of designing a beam, one has to significantly consider the loading arrangement that may be applied to the beam in many circumstances uniformly distributed loads are applied on the structures. It is good to follow the two-point loading approach than the central concentrated loading.

The effect of load arrangement must be checked for three-point, four-point, etc. until the arrangement of loads resembles the real loads on the beams usually uniformly distributed load.

The load-carrying capacity and the shear strength of other tapered shapes should be studied since tapered/haunched beams have different shear strength behaviors and rarely elaborated on the standard building design codes.

This paper is focused only on simply supported beam, one has also to further study the behavior of tapered beams in a continuous structure.

Since the production of formwork for tapered shape need much effort and caution strong materials such as steel formwork must be provided to maintain the shape of the beam until gets the strength.

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APPENDIX A: CONCRETE MIX DESIGN

The concrete mix design was done using the ACI method. The specified strength of concrete is 30 MPa at 28 days measured on standard cylinders.

The slump size = 50 mm

The maximum size of aggregate = 25 mm

Bulk specific gravity of Coarse aggregate (CA) = 2.84%

Absorption of Coarse aggregate (CA) = 1.373%

Moisture of Coarse aggregate (CA) = 0.83%

Unit weight of Coarse aggregate (CA) = 1653.64 kg/m^3

Bulk specific gravity Fine aggregate (FA) = 2.47

Absorption Fine aggregate (FA) = 2.81%

Moisture Fine aggregate (FA) = 2.51%

Fineness Modulus (FM) Fine aggregate (FA) = 3.06

Step 1: Choice of Slump

The slump of the concrete is selected depending on the type of construction and degree of workability. An appropriate value can be chosen from Table 1.

Table 1. Recommended Slumps for Various Types of Construction

Type of construction	Slump	
	(mm)	**(inches)**
Reinforced foundation walls and footings	25-75	1.0-3.0
Plain footings, caissons and substructure walls	25-75	1.0-3.0
Beams and reinforced walls	25-100	1.0-4.0
Building columns	25-100	1.0-4.0
Pavements and slabs	25-75	1.0-3.0
Mass concrete	25-50	1.0-3.0

Step 2: selection of maximum aggregate size

The maximum size of the coarse aggregate used is 25 mm.

Step 3: Estimation of mixing water and Air content

The beams are not exposed to special exposure conditions. Therefore, Pozzolana Portland Cement without air entrainment was used.

The amount of mixing water and entrapped air required for non-air entrained concrete for the selected slump (50 mm) and maximum size of course aggregate (25 mm) from Table 2 is 179 kg/m^3 and 1.5% respectively.

Table 2. Approximate Mixing Water and Air Content Requirements for Different Slumps and Maximum Aggregate Sizes

	Mixing Water Quantity in kg/m^3 (lb/yd^3) for the listed Nominal Maximum Aggregate Size							
Slump	9.5 mm (0.375 in.)	12.5 mm (0.5 in.)	19 mm (0.75 in)	25 mm (1 in.)	37.5 mm (1.5 in)	50 mm (2 in.)	75 mm (3 in.)	100 mm (4 in)
Non-Air-Entrained PCC								
25-50 (1-2)	207 350	199 335	190 315	179 300	166 275	154 260	130 220	113 190
75-100 (3-4)	228 385	216 365	205 340	193 325	181 300	169 285	145 245	124 210
150-175 (6-7)	243 410	228 385	216 360	202 340	190 315	178 300	160 270	. .
Typical entrapped air (percent)	3	2.5	2	1.5	1	0.5	0.3	0.2
Air-Entrapped PCC								
25-50 (1-2)	181 305	175 295	168 280	160 270	148 250	142 240	122 205	107 180
75-100 (3-4)	202 340	193 325	184 305	175 295	165 275	157 265	133 225	119 200
150-175 (6-7)	216 365	205 345	197 325	184 310	174 290	166 280	154 260	. .
Recommended Air Content (percent)								
Mild Exposure	4.5	4	3.5	3	2.5	2	1.5	1
Moderate Exposure	6	5.5	5	4.5	4.5	4	3.5	3
Sever Exposure	7.5	7	6	6	5.5	5	4.5	4

Step 4: Determination of water to cement ratio

The water-cement ratio was obtained based on the dual criterion of durability and strength using table 3 and 4. The estimated water to cement ratio based on the exposure/durability conditions from Table 3 is = 0.5

The estimated water-cement ratio for an average strength of 30 MPa cylindrical strength for non-air entrained concrete from Table is = 0.55. Therefore, the lower value from the two ratios is water-cement ratio = 0.5. w/c = 0.5 is used in the design.

Table 3. Maximum Permissible W/C Ratios for Different Types of Structures and Degrees of Exposure

Exposure Condition	Maximum w/c ratio, normal density aggregate concrete	Minimum design strength, low-density aggregate concrete MPa
I. Concrete Intended to be watertight		
(a) Exposed to freshwater	0.5	25
(b) Exposed to brackish or seawater	0.45	30
II. Concrete exposed to freezing and thawing in a moist condition:		
(a) curbs, gutters, guard rails or thin sections	0.45	30
(b) other elements	0.5	25
(c) in the presence of de-icing chemicals	0.45	30
III. For corrosion protection of reinforced concrete exposed to de-icing salts, brackish water, seawater or spray from these sources.	0.4	33

Table 4. Relationship between W/C Ratio and Compressive Strength of Concrete

Cylinder compressive strength at 28 days (N/mm)	Water/cement ratio by weight	
	Non-air entrained concrete	Air-entrained concrete
45	0.38	.
40	0.43	.
35	0.48	0.4
30	0.55	0.46
25	0.62	0.53
20	0.7	0.61
15	0.8	0.71

Step 5: Determination of cement content

The amount of cement content per unit volume of the concrete is obtained by dividing the estimated water content by the w/c ratio.

Cement content = (179/0.5) kg/m^3 = 358 kg/m^3

Step 6: Determination of coarse aggregate (CA) content

Dry bulk volume of coarse aggregate per unit volume of concrete for 25 mm coarse aggregate and 3.0 Fineness modulus from Table 5 is = 0.65

The weight of coarse aggregate = Dry bulk volume * Dry rodded bulk volume

The weight of coarse aggregate = 0.65*1653.64 kg/m^3 = 1074.86 kg/m^3

Table 5. Dry Bulk Volume of Coarse Aggregate per Unit Volume of Concrete

Maximum size of aggregate	Dry bulk volume of rodded coarse aggregate per unit volume of concrete for fineness modulus of sand of:				
Mm	**In**	**2.4**	**2.6**	**2.8**	**3**
10	3/8.	0.5	0.48	0.46	0.44
12.5	1/2.	0.59	0.57	0.55	0.53
20	3/4.	0.66	0.64	0.62	0.6
25	1	0.71	0.69	0.67	0.65
40	1. 1/2.	0.75	0.73	0.71	0.69
50	2	0.78	0.76	0.74	0.72
70	3	0.82	0.8	0.78	0.76
150	6	0.87	0.85	0.83	0.81

Step 7: weight of fine aggregate (FA) content

The first estimate of density of fresh concrete for 25 mm maximum size of coarse aggregate and for non-air entrained concrete from Table 6 = 2380 kg/m^3

The weight of fine aggregate is the difference between the weight of fresh concrete and the total weight of all other ingredients.

The weight of the known ingredients of the concrete are:

Weight of coarse aggregate = 1074.86 kg/m^3

Weight of the cement = 358 kg/m^3

Weight of mixing water = 179 kg/m^3

The weight of fine aggregate = (2380-1074.86-358-179) kg/m^3 = 768.14 kg/m^3

Table 6. First Estimate of Density (Unit Weight) of Fresh Concrete

Maximum size of aggregate		First estimate of density (unit weight) of fresh concrete			
		Non-air-entrained		Air-entrained	
Mm	**In**	**kg/m^3**	**lb/yd^3**	**kg/m^3**	**lb/yd^3**
10	3/8.	2285	3840	2190	3690
12.5	1/2.	2315	3890	2235	3760
20	3/4.	2355	3960	2280	3840
25	1	2380	4010	2285	3850
40	1. 1/2.	2415	4070	2320	3910
50	2	2445	4120	2345	3950
70	3	2495	4200	2400	4040
150	6	2530	4260	2440	4110

The weight of fine aggregate is also calculated using absolute volume method which is a more accurate method;

	INGREDIENTS	WEIGHT (kg/m^3)	ABSOLUTE VOLUME (m^3)
1	CEMENT	358	358/3.15 = 113.65
2	WATER	179	179/1.0 = 179
3	COARSE AGGREGATE	1074.86	1074/2.84 = 378.17
4	AIR	1.5%	(1.5/100) *1000 = 15
	TOTAL		685.82

The absolute volume of fine aggregate = (1000-685.82) m^3 = 314.2 m^3

The weight of fine aggregate

= absolute volume of fine aggregate* Bulk specific gravity Fine aggregate

= 314.2 m^3*2.47= 776.07 kg/ m^3

Step 8: Adjustment for Aggregate Moisture

The proportion of coarse aggregate, fine aggregates and amount of mixing water was adjusted based on the absorption capacity of aggregates and the moisture contents of the aggregates:

The adjusted weight of fine aggregate = 780 + 2.51/100 (780) = 800 kg/m^3

The adjusted weight of coarse aggregate = 1074.86 + 0.83/100 (1074.86) = 1084 kg/m^3

Water from fine aggregate = 780*(2.51-2.81) = -2.34 kg/m^3

Water from coarse aggregate = 1074.86*(0.83-1.373) = -5.84 kg/m^3

The adjusted weight of water = (179+2.34+5.84) kg/m^3 = 187.2 kg/m^3

MIX PROPORTIONS				
INGREDIENTS	CEMENT	FINE AGGREGATE	COARSE AGGREGATE	WATER
QUANTITY kg/m^3	358	800	1084	187.2
RATIO	1	2.235	3.03	0.523