ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING



Effects of Coarse Aggregate Size on Shear Strength of Reinforced Concrete Beam with Web Reinforcement

A Thesis in Structural Engineering

By

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UNDERTAKING

I certify that research work titled **"Effects Of Course Aggregate Size on Shear Strength of Reinforced Concrete Beams with Web Reinforcement"** 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.

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ABSTRACT

Mechanism of flexural failure of a rectangular reinforced concrete beam was much simpler to understand as compared to shear failure. In fact shear failure of reinforced concrete beams is a very complex phenomenon due to involvement of too many parameters.

In spite of many decades' experimental research and use highly sophisticated analytical tools shear failure is not yet fully understood. Furthermore if a beam is not properly designed shear reinforcement is overloaded to failure shear collapse is likely to occur suddenly with no advance warning of distress. This is in strong contrast with the nature of flexural failure. Though aggregate interlock is one of the determinant behavior in shear, its behaviors and contributions are not clearly stated yet on shear. But scholars have figured out that the shear stress that can be transmitted along cracks is related to concrete strength, crack width and aggregate size. From among factors that determines the shear performance of RC beam we want to identify the effect of aggregate size. Here we are always controlling the other parameters that determines the shear performance of RC beam.

Objective of this research was to identify how aggregate size will influence the shear strength of reinforced beam with shear reinforcement. And with specific objective to analyze the effects of aggregate size on shear strength. To determine size effects numerically (i.e. the effective diameters of aggregate for shear performance). To recommend the most effective aggregate size for shear performance of reinforced beam with shear reinforcement.

This research paper is based on the experimental results of eight (8) RC beams test as specimen by changing the aggregate size and by controlling the other parameters to analyze the impact of aggregate size on the shear performance of RC beam with web reinforcement.

After performing an experimental program at AAiT construction material laboratory on different nominal aggregate sizes; it was noticed that, as the aggregate size increase there is a significant increase in shear performance and an increase in the ductility of RC beams that has been under examinations.

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CHAPTER ONE

1. INTRODUCTION

1.1 General Background

Reinforced concrete belongs to the type of structure which are assumed to be cracked under normal service condition this assumption is due to the fact that plain concrete has comparatively smaller fracture strain in tension and therefore will crack at low stress, once cracked the reinforcement become active and take over the force from concrete.

Shear failure of reinforced concrete more properly called diagonal tension failure is one example which is difficult to predict accurately. In spite of many decades' experimental research and use of highly sophisticated analytical tools it is not fully understood yet. Furthermore if a beam is not properly designed, shear reinforcement is overloaded to failure shear collapse is likely to occur suddenly with no advance warning of distress. This is in strong contrast with the nature of flexural failure.

For rational analysis of reinforced concrete interfaces, mechanical behaviors of interacting components across the interfaces should be properly conceptualized. Knowledge and modeling related to shear forces in cracked concrete are still not fully understood, because of the fact that shear loading leads to complicated physical mechanisms, such as multi-axial stress conditions, interlocking of cracks, dowel action and reduced bond resistance of embedded bars.

Mechanism of flexural failure of a rectangular reinforced concrete beam was much simpler to understand as compared to shear failure. In fact shear failure of reinforced concrete beams is a very complex phenomenon due to involvement of too many parameters. Factors influencing the shear capacity of beams are shear span to depth ratio (a/d), tension steel ratio (ρ), compressive strength of Concrete (f_c'), size of coarse aggregate, density of concrete, use of fibers in concrete, size of beam, position and geometry of haunches, tensile strength of concrete, support conditions, clear span to depth ratio (L/d), number of layers of tension reinforcement, grade of tension reinforcement and end anchorage of tension reinforcement

From time to time, shear must be transferred across an interface between two members that can slip relative to one another. The shear-carrying mechanism is known variously as aggregate interlock, interface shear transfer, or shear friction. Aggregate interlock is a natural mechanism effective in transferring loads across discontinuities, such as joints and cracks. To address such a valuable and unforeseen importance we need to know the properties of aggregate and understand their effective size to cope up with. Hillerborg (1985) reported that mortar shows smaller fracture energy (GF) values than concrete and there is a trend of concrete GF increase with an increase of the maximum aggregate size.

The degree of mechanical interlocking and internal stress concentration around the aggregate, related to the aggregate surface characteristics, are found to have significant influence on micro cracking behavior of concrete composites. Furthermore, aggregate surface characteristics become more pronounced with increased matrix quality. For HSC (low w/c) with large aggregate, the degree of mechanical interlock is more dominant than the uniformity in stress concentration. For LSC (high w/c) with large aggregate, uniformity in stress concentration is more significant than the degree of mechanical interlock.

Elzanaty et al. (1986) looked at the problem of shear in high-strength concrete and observed a smoother failure plane in the higher strength concrete specimens. Their study was performed on a total of 18 beams with concrete strengths f_c', ranging from 21 to 83 MPa. Apart from concrete strength, test variables included ρ and the shear span-to-depth ratio,a/d .The conclusions drawn from these tests were that the shear strength increased with increasing f_c' but less than that predicted using the ACI Code equations. These equations predict an increase in shear strength in proportion to $\sqrt{f_c'}$. Elzanaty et al. concluded that an increase in the steel ratio led to an increase in the shear capacity of the specimens regardless of concrete strength.

The importance of aggregate interlock, or shear-friction, across shear cracks has been recognized for quite some time as one of the mechanisms of shear resistance in reinforced concrete (RC) beams. ACI code provisions for shear (ACI 318-95 1) do not take explicit account of shear friction, but rather lump it together with other factors, such as dowel effects and shear carrying capacity of the compressed part of the beam, into the concrete contribution term Vc.

Several well -established theories based on equilibrium considerations (strut -and-tie models and stress fields) can be applied when shear reinforcement is provided, leading to safe design solutions. Theories also considering Compatibility conditions the tensile strength of concrete (compression field -based theories and fixed-angle softened-truss model) have also been developed allowing accurate predictions of the shear response of transversely reinforced members. Some general theories, such as the modified compression field theory (MCTF), have been successfully applied to members without shear reinforcement but not yet justified for members with web reinforcement [9].

In lieu of the latter simplified shear design equations, the CSA Standard (CSA 1994) proposes a more rational method of approach to the shear design "problem" based more on fundamental principles than on empirical equations. This method treats the stress-strain characteristics of the cracked concrete using average stresses and strains in the concrete and utilizes equilibrium and compatibility of strains. The crack pattern is also idealized as a series of parallel cracks occurring at an angle θ to the longitudinal direction. The theory considers that the shear strength of concrete at a crack location is dependent on the width of the crack as well as the maximum aggregate size used (i.e., it looks at the crack roughness). This method accounts for the strain softening of the diagonally cracked concrete in compression and also accounts for the tensile stresses in the cracked concrete (Vecchio and Collins 1982). The modified compression field theory is explained in detail by Collins and Mitchell (1997)

The Ethiopian Building Code Standard 2 (Structural Use of Concrete) provisions on shear design of Reinforced concrete members doesn't Consider effect of aggregate size. Thus, this research might answer the question of whether or not size of an aggregate has an effect on the shear behavior of beams. Furthermore if the effect of aggregate size is found to enhance the capacity, results of this research might suggest an improved way of choosing aggregate size while designing reinforced concrete members.

1.2 Research objective

1.2.1 General objective

- Objective of this research is to identify how aggregate size will influence the shear behavior of reinforced concrete beam with shear reinforcement.

1.2.2 Specific objectives

- To analyze the effects of aggregate size on shear strength
- To determine size effects specifically (i.e. the effective diameters of aggregate for shear performance)
- To recommend the most effective aggregate size for shear performance of reinforced beam with shear reinforcements.

1.3 Scope

This research will take considerations about only simply supported normal strength concrete (NSC) beams, which are subjected with flexure and shear which have web reinforcement in addition to longitudinal reinforcement. Beams with other types of loading and support condition, beam columns, very short beems, slender beems, pre-stressed concrete and beams made of high strength concrete (HSC) will not be included in this research.

1.4 Methodology

The overall overview of this research is based on literature review and experimental program at AAiT construction material laboratory.

First the basic shear design theories such as the simplified method and modified compression field approaches will be studied and checked if size of aggregate is a factor affecting the shear behavior of beams. Then, the most widely used building codes provisions and their Shear design provisions will be reviewed.

This research will also include experimental process on different RC beam specimens with different aggregate sizes, their response while they are loaded to shear will be forwarded as an experimental result.

CHAPTER TWO

2 LITERATURE REVIEW

2.1 Previous Investigations on shear strength and its parameters.

A beam resists loads primarily by means of internal moments, M, and shear, V, as shown in Fig. 2-1. In design of a reinforced concrete member, flexure is usually considered first, leading to the size of the section and arrangement of the section 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 a warning to the occupants. The beam is then proportioned for shear. Because a shear failure is frequently sudden and brittle, the design of shear must ensure that the shear strength equals or exceeds the flexural strength at all points in the beam [5].

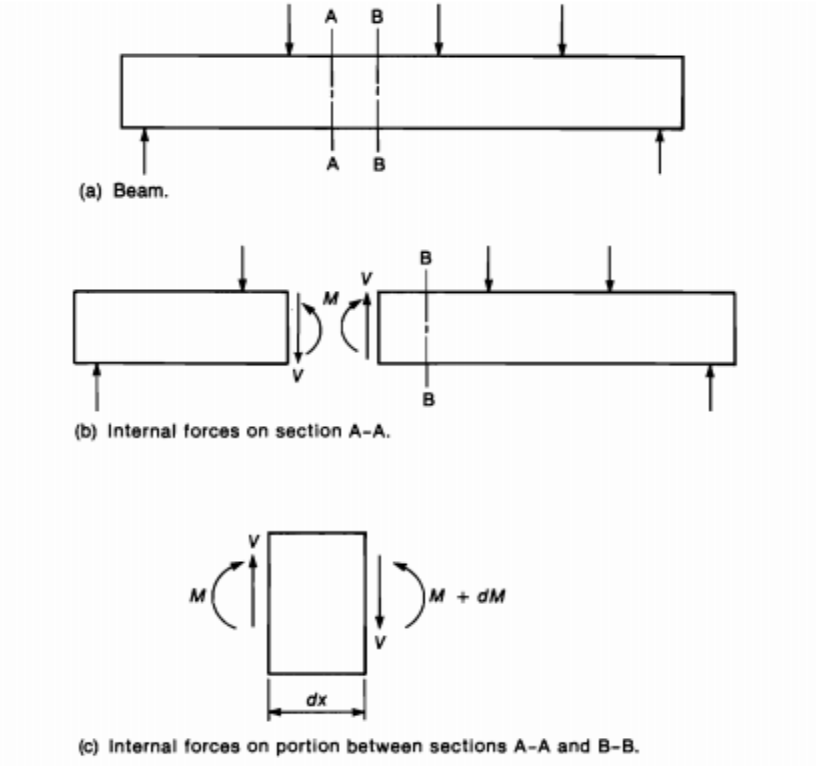


Figure 2.1. Internal forces in beam. from McGregore

2.1.1 Basic theory

Mohr's circle for stress.

Shear cracks develop due to principal tensile stress that exceed the tensile strength of the concrete. The most convenient way to determine the principal stresses at appoint is to use moher's circle for stress. We shall briefly review this method of calculation. Fig. 2.2 shows an element with a normal stress *f* acting on the vertical sides, and shearing stresses υ acting as shown. We shall refer to the sides of the element as planes **A** and **B** as shown. The shearing stresses on plane **A** and the opposite vertical plane act to form a counter-clockwise couple. These stress can be plotted in a Moher'circle for stress as shown in Fig. 2-2b. The normal stress on plane **A** is tensile and plots as positive *f* and, because the shear on **A** acts to form a clockwise couple. It plots a positive υ. The coordinates of the point in the circle representing plane **B** are f=0, and – υ. Points **A** and **B** are the opposite ends of a diameter of circle with its center on the horizontal axis. A circle can be drawn joining A and **B.** points on the circle represent stresses on plane through the element. Angles in the circle represent stresses on planes which are 90^0 apart in the element. In the circle, they plot as $2x90^0=180^0$ apart and represent the two ends of a diameter of the circle [5].

The algebraic maximum stress, on any plane through the element(where tensile stress is positive) is the principal tension stress, or maximum principal stress, f_1, located in the circle at point C at an angle of $2\theta cw$ from point **A**. in the element it acts on plane **C** oriented at θcw from plane **A** as shown in Fig. 2-2c. Since the angle from point **A** in the circle to the point representing f_1 is clockwise in the circle, it is clockwise in the element. The principal compression stress or minimum principal stress is f_2, which acts on a plane located at 180^0 from plane **C** in the circle, which corresponds to 90^0 from plane **C** in the element. There are no shear stresses acting on the planes of maximum and minimum principal stresses.

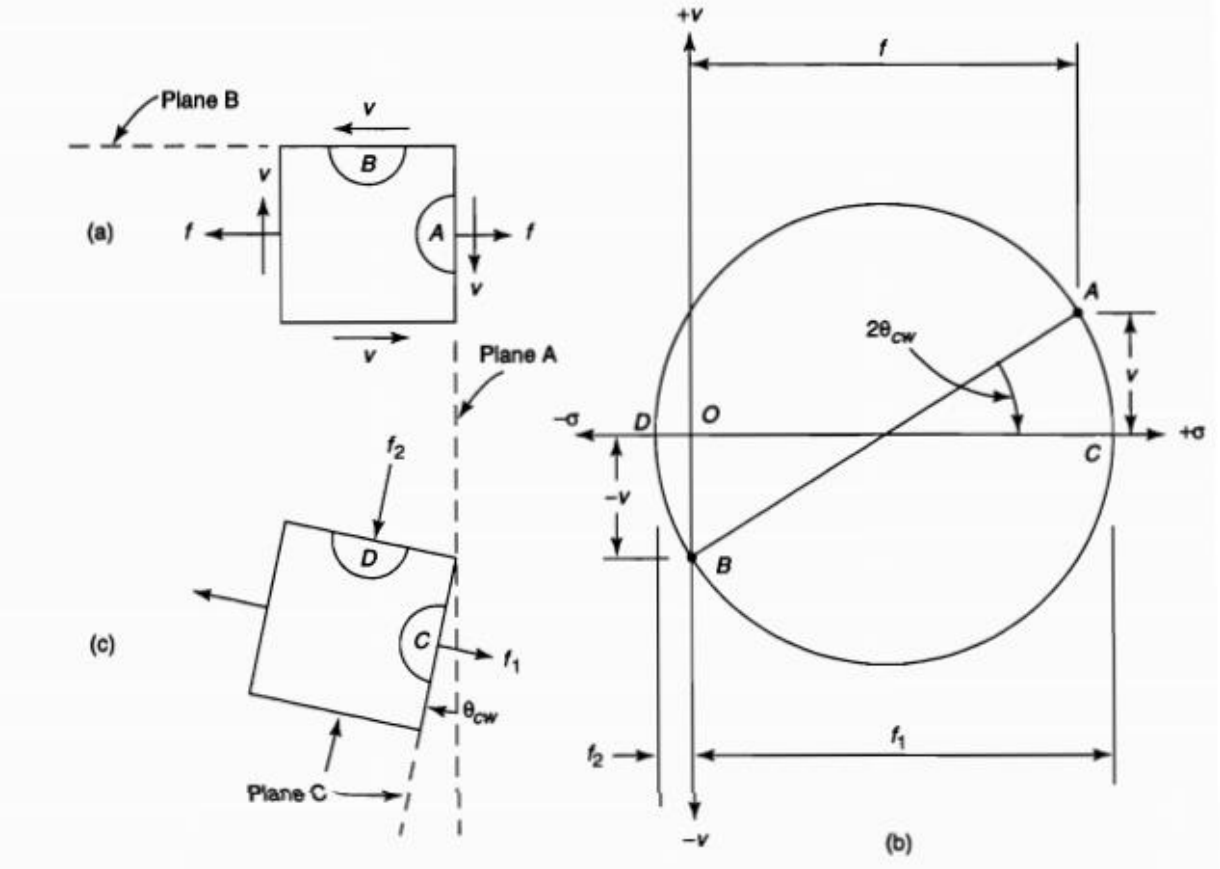


Figure 2.2; Mohr's circle for stress. From McGregore.

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

Bazant and Kim (1984)[16] derived a shear strength equation based on the theory of fracture mechanics. This equation accounts for the size effect phenomenon as well as the longitudinal steel ratio and incorporates the effect of aggregate size. This equation was calibrated using 296 previous tests obtained from the literature and was compared with the ACI Code equations. It was noted after the comparison that the practice used in the ACI Code of designing for diagonal shear crack initiation rather than ultimate strength does not yield a uniform safety margin when different beam sizes are considered. It was also found. According to the new equation. That for very large specimen depths the factor of safety in the ACI Code almost disappears. However, no experimental evidence was available yet to confine that fact as all the tests performed up to that trine were on relatively small specimens. This equation was improved by Bazant and Sun (1987)[17] to

account for the maximum aggregate size distinctly from the size effect phenomenon and was extended to cover the influence of stirrups.

Because of the difference in behavior reinforced concrete beams are generally provided with special shear reinforcement to ensure that flexural failure would occur before shear failure if the member were severely overloaded. The shear stress in most beams are far below the direct share strength of concrete. The real concern is with diagonal tension stress resulting from the combination of shear stress and longitudinal flexural stress. (Design of concrete structurs,14 ed, Nilson)

Kani (1966 and 1967) was amongst the first to investigate the effect of absolute member size on concrete shear strength after the dramatic warehouse shear failures of 1955 (Collins and Kuchma, 1997 and Collins and Mitchell, 1997). His work consisted of beams without web reinforcement with varying number depths,d, longitudinal steel percentages, ρ , and shear span-to-depth ratios, a/d. He determined that member depth and steel percentage had a great effect on shear strength and that there is a transition point at a/d=2.5 at which beams are shear critical (i.e. the value of the bending moment at failure was minimum). Kani found this value of a/d to be the transition point between failure modes and is the same for different member sizes and steel ratios. Below a/d value of about 2.5 the test beams developed arch action and had a considerable reserve of strength beyond the first cracking point. For a/d values greater than 2.5 failure was sudden, brittle and in diagonal tension soon after the first diagonal cracks appeared. This transition point is more emphasized in test beams containing higher reinforcement ratios and almost disappears in specimens with lower reinforcement ratios. In addition. Kani found a clearly defined envelope bounded by limiting values of ρ anda/d. Inside this envelope diagonal shear failures are predicted to occur and outside of this envelope flexural failures are predicted to occur. These conclusions regarding the influence of both ρ and a/dwere similar for all beam depths tested. Kani also looked at the effect of beam width and found no significant effect on shear strength. Kani's work was summarized in the textbook "Kani on Shear in Reinforced Concrete" (Kani et al. 1979).

An extensive survey of the accessible literature on the mechanisms of shear transfer in cracks in concrete was published as Stevin report [16] in 1978*.

The main conclusion of this survey were the following:

- Many basically different types of formulation to describe shear transfer in cracks are found. Several of these would lead to irrational results if applied to the analysis of shear critical structures.
- When the crack faces are subjected to a shear displacement a wedging action is developed, resulting in compressive stress normal to the crack plane. Reinforcement crossing the crack or other restraint elements are activated, the forces in which must maintain equilibrium.
- Although the resistance to shear displacement of concrete interfaces, generally designated as aggregate interlock, has been investigated by a number of authors in a various way, hardly any attention has been given to the phenomena of wedging action which provides a link between normal and shear stress, on the one hand crack opening and shear displacement on the other hand.
- The crack width is generally considered the most important variable influencing the shear stress-shear displacement relation. The concrete strength is believed also to be of major influence. Several opinions exist with regard to the influence of maximum particle size and aggregate type (rounded, crushed, lightweight).
- Aggregate interlock and dowel action show similar relationships between shear stress and shear displacement. For small crack widths aggregate interlock dominates over dowel action.

Z P Bazant and J K Kim [10] in 1984 analyzed diagonal shear failure of longitudinally reinforced concrete beams using recent fracture mechanics approach. In addition to size effect a rational formula for the effect of steel ratio and shear span was derived.

$$v_u = \left[\frac{10\sqrt[3]{\rho}}{\left(1+\frac{d}{25da}\right)^{\left(\frac{1}{2}\right)}}\right]\left[f_c' + 3000\sqrt{\frac{\rho}{\left(\frac{a}{d}\right)^5}}\right] \qquad (1)$$

Where da is the Maximum aggregate size

The above equation (1) has better agreement with the test data. In this equation five parameters ($f_c{}'$, ρ, d/a, d and da) are correlated with ultimate shear strength of rectangular beams, especially the effect of aggregate size, which plays very important role in the shear strength.

2.1.2 A review of experimental studies

Stanik (1998) performed tests on a wide range of beam specimens at the University of Toronto. The specimens tested had varying depths, d, ranging from 125 mm to 1000 mm, varying amounts of longitudinal steel (0.76% to 1.31%) as well as varying concrete strengths, f_c, ranging from 37 MPa to 99 MPa. The longitudinal reinforcement was distributed in some specimens along the sides and some specimens contained the minimum amount of transverse reinforcement recommended by the CSA Standard (CSA 1994). In the series with longitudinal bars along the sides, a set of wider beams was also tested. The purpose was to evaluate the influence of the amount, as well as the distribution of the longitudinal steel on the shear strength. Stanik found that the size effect is very pronounced in lightly reinforced deep members. Members containing the minimum amount of transverse reinforcement or side distributed steel performed better than their counterparts with only bottom longitudinal reinforcing bars. Deep members with side distributed reinforcement performed nearly as well as the shallow members containing only bottom longitudinal reinforcement. As well, the wider members containing side distributed steel were weaker than the narrower ones with similar side distributed steel. Stanik concluded that the size effect is more related to measures controlling crack widths and crack spacing rather than the absolute depth of the member. Moreover, Stanik found very little gain in shear strength with the use of higher concrete strengths. In fact, he found that the shear strengths of the beams with 100 MPa concrete were only marginally greater than the shear strength of the 40 MPa beams. Stanik used the modified compression field theory proposed by the CSA Standard (CSA 1994) to predict the response of his test beams. He found good agreement between his experimental results and these predictions. He also proposed to use an effective aggregate size of zero in the modified compression field theory method for the very high-strength concretes in order to account for the insignificant gain in shear strength from the lower concrete strengths. Stanik also performed a comparison between his experimental results and the ACI Code (ACI committee 3 18-1995) expressions. He found that the ACI expressions substantially overestimate the shear contribution of concrete, notably in the deeper members.

Walraven and Reinhardt[9] and Walraven Ferny and Pruijssers[10] performed some important work at Delft university of technology on the constitutive relation of cracks in concrete loaded in shear. Their work accounts for aggregate interlock, dowel action and axial tension of the reinforcement crossing a crack, combines experiment and theory and shows a good agreement between the two. More recently Walraven[11] and Walraven and Stroband[12] showed high-strength concrete (HSC) exhibits marked reduction in shear friction compared with normal strength concrete.

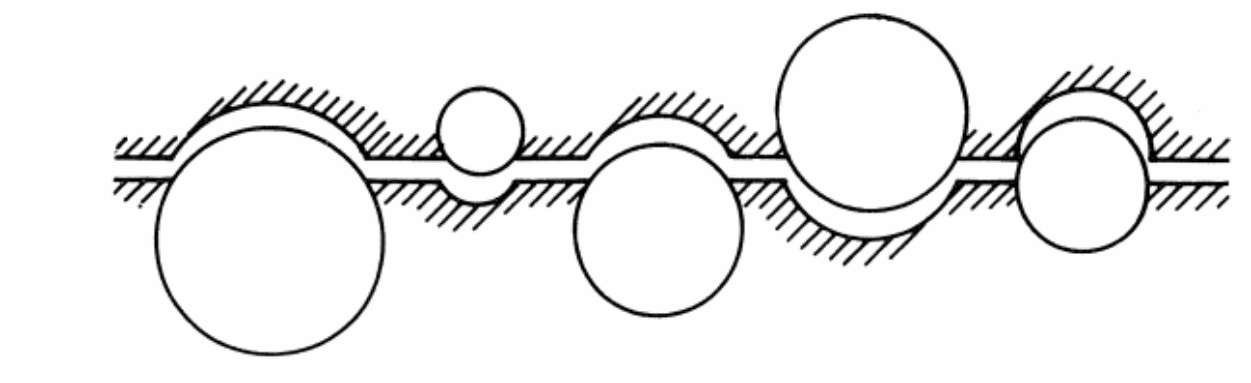


Figure 2.3: Designation of crack plane after Walraven and Reinhardt.

Concrete can represented as two phase-system: in matrix (hardened cement past) a collection of aggregate particles are embedded. Generally the strength and stiffness of the aggregate particles are greater than the matrix. However, the contact area between the two materials, the bond zone, is the weakest link of the system. Hence cracking occurs commonly through the matrix but along the circumference of the aggregate particles. These particles are simplified to spheres which can be intersected by the crack plane at all depths with the same probability.

Hardened cement past is visco-elastic material: the deformations provoked by stress are only partially elastic; for the other part they are plastic. Under multi-axial stresses, in the area between the aggregate particles in concrete, large plastic deformations are expected to predominate over the elastic deformations, the stress-strain relations of the matrix material, consisting of hardened cement past with aggregate particles smaller than 0.25 mm is assumed rigid plastic.

These studies showed that the shear stress that can be transferred across the cracks depends on the compressive stress, the crack width, the crack slip, the strength of the mortar and concrete, the coefficient of friction between the mortar and the aggregate, the maximum size and the volumetric percentage of aggregate, aggregate type and the distribution of longitudinal and lateral reinforcement.

2.1.3 Properties of coarse aggregates

2.1.3.1 Aggregate type

The properties of coarse aggregate type used in concrete mixture affects the modulus of elasticity for few reasons. A higher aggregate modulus will result in a concrete having higher modulus. As expected a light weight aggregate will have a lower modulus than the mortar paste. Conversely a stronger aggregate produces a concrete that is stronger than the mortar paste. Strong aggregates, such as felsite, trap rock, or quartzite, are needed to make very-high strength concretes. Weak aggregates include sandstone, marble, and some metamorphic rocks, while limestone and granite aggregates have intermediate strength. Normal-strength concrete made with high-strength aggregates fails due to mortar cracking, with very little aggregate failure.

2.1.3.2 Aggregate size

As the size (diameter) of the coarse aggregate increases, the roughness of the crack surfaces increases, allowing higher shear stresses to be transferred across the cracks.

2.1.3.3 Aggregate shape

- ✓ Flat shape aggregate
- ✓ Elongated aggregates
- ✓ Round aggregates
- ✓ Angular aggregates

2.2 Modes of shear failure

Depending upon the shear span, shear failure may be classified into three types

- diagonal tension failure
- diagonal compression failure
- splitting or true shear failure

2.2.1 Diagonal tension failure

The diagonal crack starts from the last flexural crack and turns more and more inclined under shear loading. The diagonal crack encounters resistance as it moves up into the compression zone. With further increase in load, the tension crack gradually extends at a very flat slope until sudden failure at point 2, the subsequent arch mechanism is not capable of sustaining the load beyond cracking load. This type of failure is also called beam action failure. Diagonal tension failure always occur when a > 2d(Figure 2.4). But in very limited cases it may occur for a < 2d. When load and reaction are far apart then diagonal tension failure is possible.

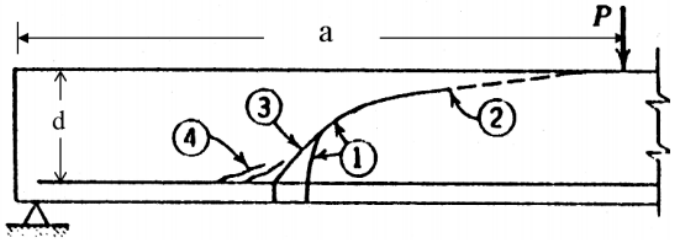


Figure 2.4; shear tension failure from F art 1.

2.2.2 Diagonal compression failure

When reaction and load are closely spaced then vertical compressive stresses under the load reduces the possibility of further tension cracking. Similarly vertical compressive stresses over the reaction also limit the bond splitting and diagonal cracking along the steel. Hence a large shear in a short length may initiate 45^{o} crack (web shear crack) across the neutral axis before a flexural crack appears. Such cracks restrict the shear resistance into a smaller un-cracked depth, thus increasing the shear stress. Hence the beam acts as a tied arch. Propagation of inclined crack reduces the compression zone. In diagonal compression failure the

specimen is able to carry additional load after formation of the first fully developed inclined tension crack . This type of failure occur for shear span (a) to depth (d) ratio 1 to 2.5(d < a < 2.5d). In the vicinity of load point it becomes too small to resist the compression force, and it crushes (shaded area in Figure 2.5). Such failure of compression zone above diagonal cracking is also called arch action.

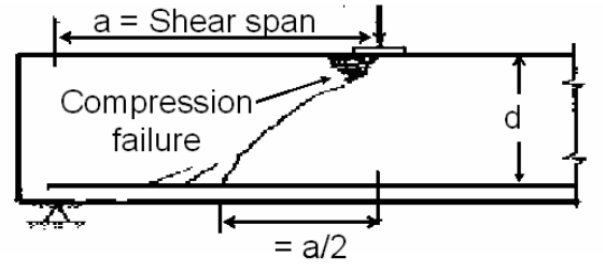


Figure 2.5: shear compression failure from F art 1.

2.2.3 Splitting or true shear failure

In this case shear is carried as an inclined thrust between load and reaction, which almost eliminates ordinary diagonal tension concepts. Splitting or true shear failure occurs when shear span is too short i.e. a < d (Figure 2.5). Shear strength is much higher in such cases. The final failure is splitting or it may fail in compression at the reaction.

2.3 Shear Strength Evaluation of RC Beams

2.3.1 The Simplified Method

The simplified method is permitted for flexural members which are not subjected to significant axial tension. The basic design equation for shear capacity of slender concrete beams (beams with shear span containing B-regions) is

$$V_r \geq V_f \qquad 2-2$$

Where V_f the shear force due to the factored loads and V_r is a factored shear resistance given by

$$V_r = V_c + V_s \qquad 2-3$$

WhereV_c , is the shear carried by the concrete and V_s is the shear carried by the stirrups.

2.3.1.1 Beams without web reinforcement

Slender beams without web reinforcement will fail when inclined cracking occurs or shortly afterward. For this reason the shear strength of such members is taken equal to the inclined cracking shear. In 1962, the ACI-ASCE committee on shear and diagonal tension presented an equation for calculating the shear at inclined cracking in beams without web reinforcement.

The SI equivalent of this equation is:

$$V_c = \left(0.158\sqrt{f_c'} + 17.2\rho_w \frac{V_f\, d}{M_f}\right) b_w d \qquad Mpa \qquad 2-4$$

More recent studies have suggested that Eq. 2-4 underestimate the effect of ρ_w for beams without web reinforcement and is not entirely correct in its treatment of the variable a/d, expressed as $\frac{V_f\, d}{M_f}$ in Eq. 2-4.

For the normal range of variables the second term in the parentheses in Eq. 2-4 will be equal to about $0.008\sqrt{f_c'}$. If this is substituted into Eq. 2-4 results

$$V_c = 0.167\sqrt{f_c'} b_w d \qquad 2-5$$

The metric edition of ACI code presents Eq. 2-5 for computing V_c and multiplies V_c by strength reduction factor (resistance factor), ϕ, equals 0.85 for shear to obtain factored resistance for design. In A23.3 separate resistance factors are applied to concrete ($\phi_c = 0.6$) and steel ($\phi_s = 0.85$) and the load factor for dead and live loads are about 0.89 times those in ACI code because it was not intended that the design for shear be more conservative in CSA 1994 A23.3 than ACI, the constant 0.167 is multiplied by 0.89(0.85/0.6) giving 0.21. This was arbitrarily rounded off to 0.2 giving

$$V_c = 0.2\lambda(\phi_c\sqrt{f_c' b_w d} \qquad 2-6$$

Where λ is a factor to account for lower tensile strength of structural low-density concrete but Eq. 2-6 is not used to compare with experimental results because the constants has been adjusted but Eq. 2-5 can.

Since as it was discussed the shear at failure in test of larger beam without web reinforcement deceases as the depth of the beam increases. For this reason CSA 1994 A23.3 limit the use of Eq. 2-6 to beams having either:

a. At least a minimum amount of web reinforcement specified in CSA 1994 A23.3 or
b. An effective depth, *d,* not exceeding 300mm.

In ACI code an increase in V_c is permitted for members subjected to axial compression. A23.3 does not allow such an increase unless design is carried out using the general method. In addition the simplified method is not permitted for members subjected to significant axial compression.

2.3.1.2 Beams with web reinforcement

Due to inclined cracking, the strength of beams drops below the flexural capacity. The purpose of web reinforcement is to ensure that the full flexural capacity can be developed. Prior to cracking the strain in the stirrup is equal to the corresponding stain in the concrete. Since the concrete cracks at a very small stain the stress in the stirrup prior to inclined crack will not exceed 20 to 40 Mpa. Thus the stirrups do not prevent the inclined cracks from forming; they come on to play after the cracks have formed.

The forces in a beam with stirrups and an inclined crack are shown in figure 2.6, the shear transferred by tension in the stirrup is V_s. Since V_s do not disappear when the crack opens, there will always be a compression force C_1' and a shear force V_{cz}' acting on a part of the beam bellow the crack. As a result, T_2 will be less than T_1, the difference depending on the amount of web reinforcement. The force T_2 will, however, be larger than $T = M/jd$ based on the moment C.

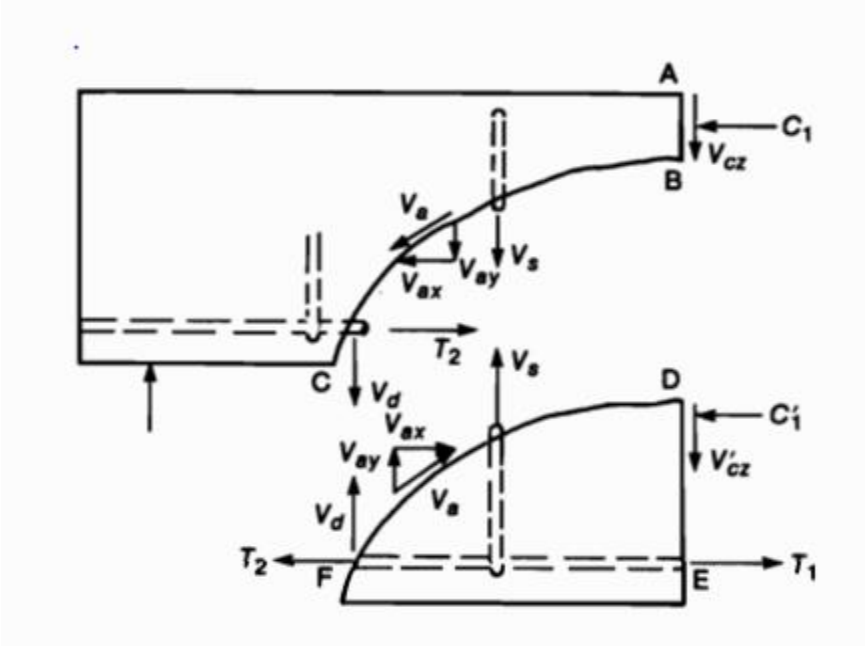


Figure 2.6: From MacGregor internal forces in a cracked beam with stirrups

Shear failure limit states for beams with web reinforcement

1. **Failure due to the yielding of the stirrups**. In fig 2.6 shear was transferred across the surface A-B-C by the shear in the compression zone, V_{cz}, the vertical component of aggregate interlock, V_{ay}, dowel action, V_d, and the stirrups, V_s. In simplified method V_{cz}, V_{ay}, and V_d are lumped together as V_c, which is referred to as "shear carried by the concrete" thus the factored shear resistance, V_r is assumed to be

$$V_r = V_c + V_s \qquad 2-7$$

A23.3 further assumes that V_c is equal to the shear strength of the beam without stirrups. Which is in turn, based on the inclined cracking load as given by Eq. 2-5 or 2-6. It should be emphasized that taking V_c equal to the shear at inclined cracking is empirical observation from test, which approximately true if it is assumed that the horizontal projection of the inclined crack is d, if flatter crack is used so that($jdcot\theta$) is greater than d, a smaller value of V_c must be used.

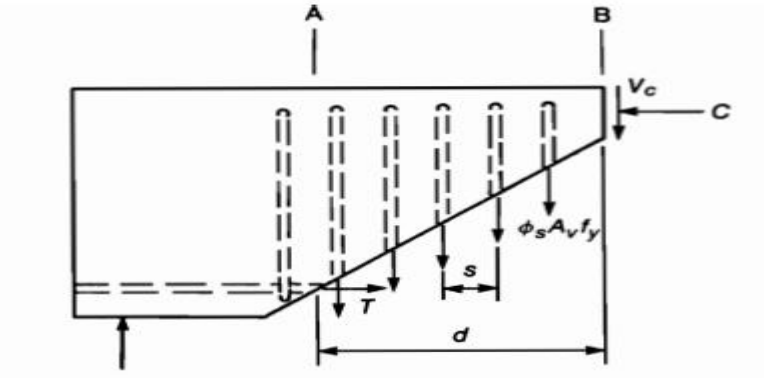


Figure 2.7: From MacGregor shear resisted by stirrups

Figure 2-7 shows a free body between the end of a beam and an inclined crack. The horizontal projection of the crack is taken as d, suggesting that the crack is slightly flatter than 45^0. If s is the stirrup spacing, the number of stirrups cut by the crack is d/s . Assuming that all the stirrups yield at failure and resistance factor for steel, the shear resisted by the stirrup is

$$V_s = \frac{\phi_s A_v f_y d}{s} \qquad 2-8$$

Stirrups are unable to resist shear unless they are crossed by the inclined cracks. It is possible that 45^0 crack to cross the web without intersecting a stirrup if the stirrup spacing exceeds d.

2. **Shear failure initiated by failure of the stirrup anchorage.** Eq. 2-8 is based on the stirrups will yield at the ultimate. This true only if the stirrups are well anchored.
3. **Serviceability failure due to excessive crack width at service load.** Wide inclined cracks in beams are unsightly and may allow water to penetrate the beam, possibly causing corrosion of the stirrups.
4. **Failure due to crushing of the web**. Compression stresses exist in the compression diagonals in the web of the beam. Since the diagonal compression stress is related to the shear stress, v, a number of codes limit the ultimate shear stress to 0.2-0.25 times the compression strength of concrete.
5. **Shear failure initiated by failure of the tension chord.** The truss analogy shows that the force in the longitudinal tensile reinforcement at a given point in the shear span is a function of a moment at a section located approximately *d* closer to the nearest section of maximum moment. Partly for this reason CSA 1994 A23.3 requires that flexural

reinforcement extend the larger of *d* or 12 times bar diameter past the point where it is no longer required for flexure.

Types of web reinforcement.

a. Stirrups or ties perpendicular to the axis of the member. This is by far the most common type of web reinforcement.
b. Welded wire fabrics perpendicular to the axis of the member serving as shear reinforcement. This may be in the form of sheet bent in to U shape, or may be a ladder-like sheet with ranges of a ladder transverse to the member.
c. Stirrups inclined at an angle of 45^0 or more to the longitudinal reinforcement.
d. Bent up bars. A portion of longitudinal reinforcement may be bent up where no longer needed for flexure
e. Spirals.

2.3.2 Modified Compression Field Theory

The web of the concrete beam cracks due to principal tensile stress and subsequently resist a shear by means of compression struts between the cracks and the tension in the stirrups similar with that of truss analogy. Away from the load and reaction this load carrying mechanism is referred to as the compression field. This theory gives a good understanding of the behavior of cracked web but tend to underestimate the strength. A compression field theory is similar to the truss analogy except that the cracked web is idealized as a cracked continuum and the crushing strength of the web is taken as a function of the transverse principal tensile strains in the cracked web. This are computed using relationships derived using Mohr's circle for strain. Vecchio and Collins [7] developed the modified compression field theory which extended the compression field theory to include the effect of tension in stirrups.

Small transverse tension stresses can develop in the concrete struts between the cracks due to the bond between the stirrups and the struts, similar to the tension stresses in the concrete between the cracks. These tends to increase the shear carrying capacity of the beam. The modified compression field theory is the basis of the general method.

Apportion of the web of a beam is shown in Fig. 2.9a. The stress shown are those acting at a crack. It is assumed that the stirrups are yielding at the crack. Solving vertical force equilibrium a the crack gives

$$V = v_{ci}\left(b_w \frac{d_v}{sin\theta}\right) sin\theta + A_v f_y \left(\frac{d_v cot\theta}{s}\right) \quad (9)$$

Where v_{ci} is the shear stress transferred across the crack, and d_v the effective depth for shear, equal to flexural lever arm, jd. In the simplified method.

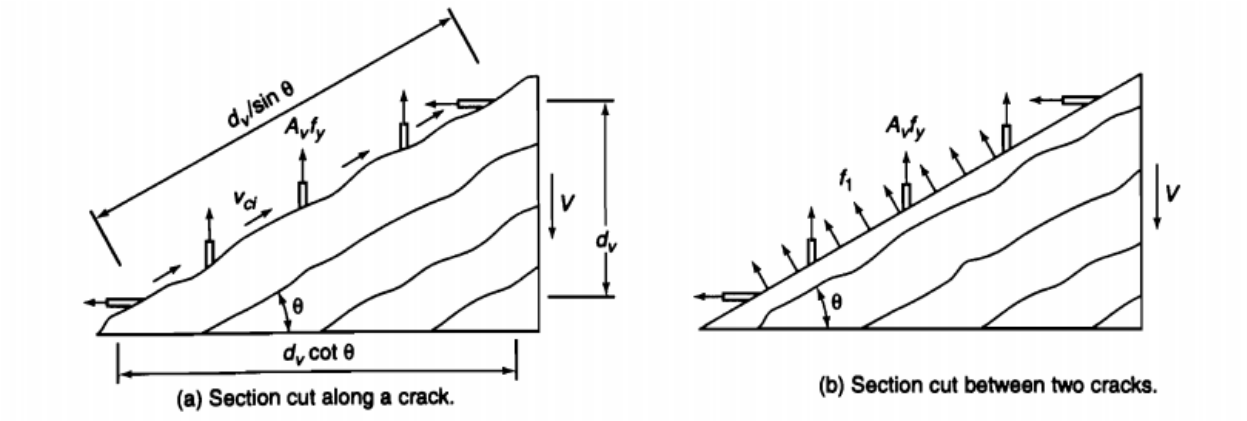


Figure 2.8: Stress at and in between inclined cracks from J G MacGregore.

$$V_r = V_c + V_s \quad (10)$$

Where

$$V_c = v_c b_w d \quad (11)$$

And v_c is the shear stress acting on a vertical section through the beam. A comparison of equation 9 and 11 suggests that for $d_v \approx d$, $v_{ci} \approx v_c$. Thus, the shear stress v_c on the vertical section in the simplified method is approximately equal to v_{ci},the shear stress transferred across an inclined crack.

A section cut halfway between two cracks is shown in Fig. 2.8b. The average tensile stress transverse to the struts is f_1 which gives rise to a force perpendicular to the inclined section in Fig. 2.8b. Summing vertical forces half way between two cracks

$$V = f_1 b_w d_v cot\theta + \frac{A_v f_v d_v cot\theta}{s} \quad (12)$$

Where f_v is the average stress in a stirrup between two cracks.

The two sets of stresses shown in Fig. 2.9a and b must equilibrate the same vertical force in both cases, thus

$$v_{ci} b_w d_v + \frac{A_v f_y d_v cot\theta}{s} = f_1 b_w d_v cot\theta + \frac{A_v f_v d_v cot\theta}{s} \quad (13)$$

Where f_v is less than f_y. To maintain equilibrium, f_1 must be limited to

$$f_1 = v_{ci} tan\theta + \frac{A_v}{s b_w}(f_y - f_v) \quad (14)$$

Modes of failure and limiting strength

a. By crushing the compression struts in the web.
b. By the average value of sustainable tensile stress transverse to the cracks in the web.
c. By stress at which slip occurs along the crack.

Sustainable value of average tensile stress f_1 after Collins and Mitchell [8] the relationship is

Prior to cracking, $\epsilon_1 \leq \epsilon_{cr}$

$$f_1 = E_c \epsilon_1 \quad (15)$$

After cracking, based on shear panel test reported by Vecchio and Collins [7] $\epsilon_1 > \epsilon_{cr}$

$$f_1 = \frac{\alpha_1 \alpha_2 f_{cr}}{1 + \sqrt{(500\epsilon_1)}} \quad (16)$$

Where ϵ_{cr} is the strain at cracking, α_1 is factore accounting for the bond characteristics of the reinforcement which equals 1.0 for deformed bars, α_2 a factore accounting for the type of the

loading which is equals to 1.0 for monotonic short-term loading to failure, and f_{cr} is the tensile stress at cracking.

Slip along the cracks.

Slip occurs along the cracks if the value of f_1 computed using equation 14 exceeds the value of f_1 from equation 16. This is a function of the ability of the cracks to transmit the shear stress v_{ci} which intern is function of the crack width, w and the maximum aggregate size a. The limiting value of v_{ci} is given by

$$v_{ci} = \frac{0.18\sqrt{f_c'}}{0.3 + \frac{24w}{a + 16}} \qquad (17)$$

The crack width w used in equation 17 is the product of the tensile strain $\epsilon_1$and the mean spacing of the inclined cracks, $s_{m\theta}$, measured perpendicular to the cracks.

$$w = \epsilon_1 s_{m\theta} \qquad (18)$$

Where $s_{m\theta}$ is a function of the spacing of vertical crack, s_{mx}, and horizontal cracks, s_{mt}.(see Ref. 7-8)

In lieu of the latter simplified shear design equations, the CSA Standard (CSA 1994) proposes a more rational method of approach to the shear design "problem" based more on fundamental principles than on empirical equations. This method treats the stress-strain characteristics of the cracked concrete using average stresses and strains in the concrete and utilizes equilibrium and compatibility of strains. The crack pattern is also idealized as a series of parallel cracks occurring at an angle θ to the longitudinal direction. The theory considers that the shear strength of concrete at a crack location is dependent on the width of the crack as well as the maximum aggregate size used (i.e., it looks at the crack roughness). This method accounts for the strain softening of the diagonally cracked concrete in compression and also accounts for the tensile stresses in the cracked concrete (Vecchio and Collins 1982). The modified compression field theory is explained in detail by Collins and Mitchell (1997) and by Collins et al. (1996).

2.3.3 Aggregate size effects in shear

The ability of the crack interface to transmit shear stress τ depends on crack width ***w.*** Vecchio and Collins [7] allowed for the possibility of local compressive normal stress σ across crack. Based on Walraven and Reinhardt's [9] experimental result they suggested the following parabolic equation to relate σ to τ.

$$\frac{\tau}{\tau_{max}} = 0.18 + \frac{1.64\sigma}{\tau_{max}} - 0.82\left(\frac{\sigma}{\tau_{max}}\right)^2 \quad (19)$$

$$\tau_{max} = \frac{\sqrt{f_c'}}{0.3 + \frac{24w}{a+16}} \quad N, mm \quad (20)$$

It turns out that the value of σ in beams failing in shear is negligible and was simplified in latter version of MCFT to Collins and Mitchell [8].

$$\tau = 0.18\tau_{max} \quad (21)$$

Thus the shear stress that can be transmitted along cracks is related to concrete strength, crack width and aggregate size in addition with dowel action and shear reinforcement.

2.4 Design code reviews

Developing a comparable theory for the shear strength of reinforced concrete members has been the objective of a major worldwide research effort for more than 40 years. For members with significant amounts of shear reinforcement, a variable angle truss model based on the lower bound theory of plasticity has been developed and is incorporated into Euro code 2.

While flexural design is concerned with ensuring that the two sides of a member can resist the appropriate magnitudes of tensile or compressive longitudinal forces, shear design is intended to ensure that the two sides of the member continue to act as a unit. This involves identifying where shear reinforcement is required to link together the two sides of the member and determining how much of this reinforcement is needed to prevent a premature shear failure. For regions not containing shear reinforcement, a shear failure can occur without warning and typically involves the opening of a major diagonal crack. Traditional American shear design procedures assumed that such failures of members without shear reinforcement would not occur if the

calculated shear stress,$V/(b_w jd)$, at service loads was less than about 0.03fc′. If the shear stress was higher than this allowable stress, than shear reinforcement would be added to carry the excess. The capacity added by the links was given by the 45^0truss model of Ritter and Morsch as $\rho_w f_s$ where V_s was the allowable tensile stress in the links. Such shear reinforcement controls the opening of diagonal cracks and permits much higher shear stresses to be resisted. Irrespective of the amount of links, however, an upper limit of $0.12f_c'$ was placed on the shear stress at service load. In traditional German shear design, by contrast, once the shear stress at service load exceeded about 0.4 MPa, shear reinforcement was provided to carry the entire shear using the 45^0truss model. Because of these differences in shear design requirements, American reinforced concrete beams typically contained much less shear reinforcement than comparable German beams.[16]

2.4.1 American concrete institute, ACI 318-01(2011)

The current ACI design procedure for shear defines the nominal shear strength as the sum of the shear strength provided by shear reinforcement, V_s, and the shear strength provided by concrete, V_c, which is assumed to be the same for beams with and without shear reinforcement and is taken as the shear causing significant inclined cracking. The value of V_c, is defined for members subject to shear and flexure only and for members subject to axial compression separately. Two alternatives are presented for calculation of V_c,. The first alternative has a simple formula. Unlike its European counterpart, the ACI provision for calculation of the concrete contribution doesn't contain the longitudinal reinforcement ratio as a factor. It only depends on the compressive strength of the concrete and the size of the member. The contribution of the shear reinforcement is based on the modified truss analogy. The truss analogy assumes that the total shear is carried by shear reinforcement. However, considerable research has indicated that shear reinforcement needs to be designed to carry only the shear exceeding that which causes inclined cracking provided the diagonal members in the truss are assumed to be inclined at 45 degrees. Thus, the ACI approach of calculating the shear reinforcement contribution can be generalized as modified 45^0 truss analogy .Shear reinforcement restrains the growth of inclined cracking. Ductility is increased and a warning of failure is provided. In an unreinforced web, the sudden formation of inclined cracking might lead directly to failure without warning. Such reinforcement is of great value if a

member is subjected to an unexpected tensile force or an overload. Accordingly, due to the reason stated above, a provision for providing minimum shear reinforcement for all members except for solid slabs, footings and joists is stipulated in ACI-318.

2.4.2 Euro code 2(2004)

In the current European Code, the shear resistance of a member with shear reinforcement is defined to be the shear resistance offered by the web reinforcements. Shear reinforcement is not necessary if the resulted shear force is less than the member design shear resistance without shear reinforcement. However minimum shear reinforcement is required except in slabs and members of minor importance.

An empirical formula is given for calculation of the contribution from the concrete in resisting shear. The empirical formula for concrete contribution takes into account the longitudinal reinforcement ratio, the compressive stress capacity of the concrete and the presence of axial force.

The calculation of shear force resistance contribution of the transverse reinforcement is based on the variable angle Truss analogy. Unlike the ACI, this code doesn't have a specific value for the crack inclination angle θ. But it gives a recommended limit of $\boldsymbol{1 < \cot \theta < 2.5}$ On the other hand, following the Truss analogy, it gives a maximum limit for the shear force to insure that the diagonal compression force will not exceed the diagonal crushing force of the concrete, V_{fcd} where v is 0.6 for concrete not grades greater than 60 MPa and 0.9-fck/200>0.5 for concrete grades greater than 60 MPa.

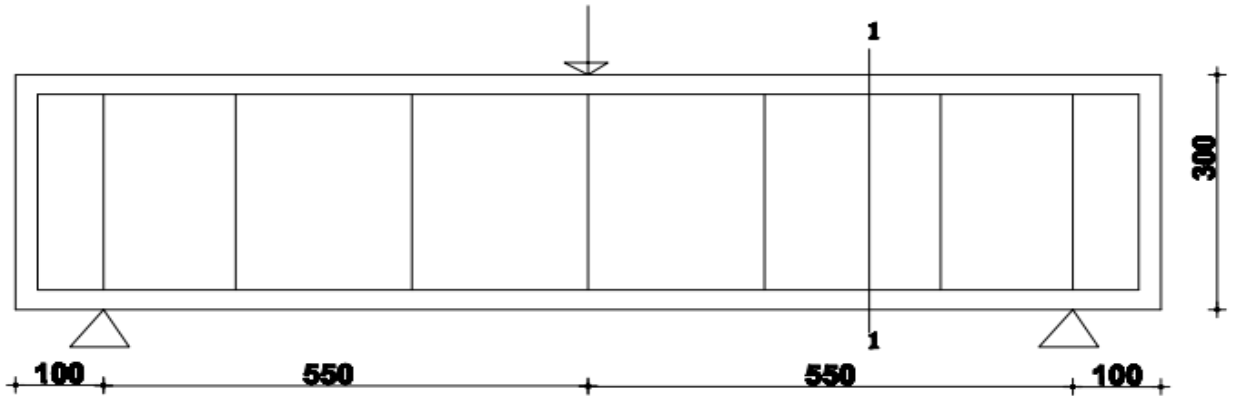
CHAPTER THREE

3 EXPERIMENTAL PROGRAM

This chapter describes the experimental program completed at the Construction Materials Laboratory, Addis Ababa Institute of Technology to study the effect of aggregate size on the shear behavior of reinforced concrete beams.

The main variable used in the current study is course aggregate size. A 1.3m (Based on a/d ratio)short beam was selected for the experimental program and shear failure was ensured by choosing short beam (a/d<2.5),with nominal web reinforcement and by providing sufficient longitudinal reinforcement bar. Prior to the experiment, shear failure was further confirmed by comparing the flexural and shear capacity using hand calculation

3.1.1 Design and detailing of beam specimen



Beam specimen

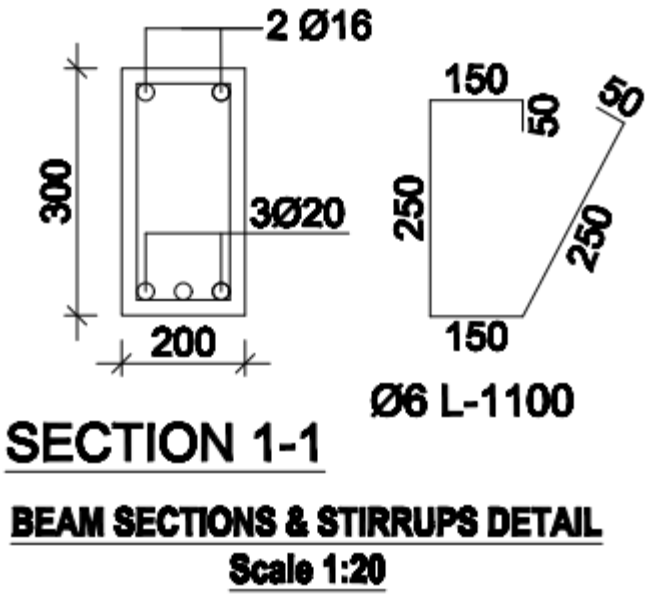


Figure 3.1; design and detailing of beam section

3.1.2 Specimen identification

Experimental program consists of eight rectangular reinforced concrete beams with a cross section and the detail drawings are given in figure 3.1. The experiments uncontrolled variable is maximum aggregate size while other parameters that can involve in the shear resistance were kept constant so that their impact can be controlled.

It was the course aggregate size used for the classification of the specimens. There were four types of maximum aggregate sizes and two specimens for each aggregate sizes which are,

1. BNAS 37.5mm:beam with nominal aggregate size of 37.5mm
2. BNAS 25mm:beam with nominal aggregate size of 25mm
3. BNAS 12.5mm:beam with nominal aggregate size of 12.5mm
4. BNAS 4.75mm. :beam with nominal aggregate size of 4.75mm

Were the fore groups of specimens that were under experiment.

Aggregate size.

It was chosen that four different maximum aggregate sizes for the experiment in which the aggregates were sieved and blinded to achieve the requirements of ASTMC33 gradation for course aggregate size. Which is depicted on the table3.1, the category of the aggregates are

- BNAS 37.5mm, No.4 (37.5mm to 19mm)
- BNAS 25mm, No.5 (25mm to 12.5mm)
- BNAS 12.5mm, No.7 (12.5mm to 4.75mm)
- BNAS 4.75mm, No.9 (4.75mm to 1.18mm)

		amount finer than each laboratory seive (square-opening), mass percentage														
seive number	nominal size(seives with square openings)	100mm(4in)	90mm(3.5in.)	75mm(3in.)	63mm(2.5in.)	50mm(2in.)	37.5mm(1.5in)	25mm(1in)	19mm(0.75in	12.5mm(0.5m)	9.5mm(0.375in.)	4.75mm(No.4)	2.36mm(No.8)	1.18mm(No.16)	300µm(No.50)
1	90to 37.5mm	100	90-100	....	25 to 60	…	0 to15	…	0 to 5	....	....	....	....	....	....
	3.5 to 1.5 in.														
2	63 to 37.5mm	....	....	100	90 to 100	35 to70	0 to 15	....	0 to 5	....	....	....	....	....	....
	2.5 to 1.5 in.														
3	50 to 25mm	....	....	....	100	90 to 100	35 to70	0 to 15	....	0 to 5	....	....	....	....	....
	2 to 1 in														
357	50 to4.75mm	....	....	....	100	95 to 100	....	35 to70	....	10 to 30	....	0 to5	....	....	....
	(2in. To No.4)														
4	**37.5 to 19mm**	**....**	**....**	**....**	**....**	**100**	**90 to 100**	**20 to 55**	**0 to 15**	**....**	**0 to 5**	**....**	**....**	**....**	**....**
	(1.5 to0.75in)														
467	37.5 to 4.75mm	....	....	....	....	100	95 to 100	....	35 to70	....	10 to 30	....	0 to 5	....	....
	1.5in to No.4														
5	**25 to 12.5mm**	**....**	**....**	**....**	**....**	**....**	**100**	**90 to100**	**20 to 55**	**0 to 10**	**0 to 5**		**....**	**....**	**....**
	1 to 0.5in												…		
56	25 to 9.5mm	....	....	....	....	....	100	90 to 100	40 to 85	10 to 40	0 to 15	0 to 5	....	....	....
	1 to 0.375in										....		....		
57	25 to 4.75mm	....	....	....	....	....	100	90 to 100	....	25 to 60	....	0 to 10	0 to 5	....	…
	1in. To No.4														
6	19 to 9.5mm	....	....	....	....	....	....	100	90 to 100	20 to 55	0 to 15	0 to 5	....	....	…
	0.75 to 0.375in.											....	....		
67	19 to 4.75mm	....	....	....	....	....	....	100	90 to 100	....	20 to 55	0 to 10	0 to 5	....	....
	0.75in toNo.4														
7	**12.5 to 4.75mm**	**....**	**....**	**....**	**....**	**....**	**....**	**....**	**100**	**90 to 100**	**40 to 70**	**0 to 15**	**0 to 5**	**....**	**....**
	0.5in. To No.4														
8	9.5 to 2.36mm	....	....	....	....	....	....	....		100	85 to 100	10 to 30	0 to 10	0 to 5	....
	0.375in. To No. 8														
89	9.5 to1.18mm	....	....	....	....	....	....	....		100	90 to 100	20 to 55	5 to 30	0 to 10	0 to5
	0.375in. To No. 16														
9^	**4.75 to 1.18mm**	**....**	**....**	**....**	**....**	**....**	**....**	**....**			**100**	**85 to 100**	**10 to 40**	**0 to 10**	**0 to 5**
	No.4 to No.16														

Table 3.1 grading requirements of aggregates.

3.1.3 Material properties

3.1.3.1 Concrete properties

The concrete for the specimens were casted metal and wooden form works. A maximum due care has been given to controlled the controlled variables and to set free the uncontrolled variables. Six cylinders of diameter 150mm and depth of 300mm for tensile strength test and six cubes of 150mmx150mm fore compressive strength test were casted with the beams for each category of aggregate sizes. Three of them were tested at 7^{th} day and three of them were tested at test date. The mix design for producing target c-25 concrete by using ordinary Portland cement(OPC) were performed with a great concern and the detail is just attached on the annex of this document.

Table 3.2; mix design

Material	BNAS 37.5	BNAS 25	BNAS12.5	BNAS 4.75
Max aggregate size	37.5mm	25mm	12.5mm	4.75mm
Free water(kg/m^3)	152.72	165.822	180.069	199.14
Cement content(kg/m^3)	272.13	272.13	326.229	354.09
Course aggregate (kg/m^3)	1201.58	1132.916	926.931	532.337
fine aggregate (kg/m^3)	770.47	774.443	858.069	1129.35
Water cement ratio	0.61	0.61	0.61	0.61



Figure 3.2: Compressive and tensile strength testing machine for concrete.



Figure 3.3: Tensile strength testing machine for steel and concrete

Table 3.3; compressive and tensile strength for BNAS 37.5 mm

Specimen with nominal agg. Size 37.5mm	14th date value of strength		Test date value of strength	
	Compressive strength(Mpa)	Tensile strength(Mpa)	Compressive strength(Mpa)	Tensile strength(Mpa)
Specimen 1	18.34	1.92	22.31	2.05
Specimen 2	19.67	2.01	24.65	2.45
Specimen 3	16.58	1.87	23.03	1.97
average	18.19	1.93	23.33	2.05

Table 3.4; compressive and tensile sterength for BNAS 25mm

Specimen with nominal agg. Size 25mm	14th date value of strength		Test date value of strength	
	Compressive strength(Mpa)	Tensile strength(Mpa)	Compressive strength(Mpa)	Tensile strength(Mpa)
Specimen 1	20.12	1.63	21.54	1.81
Specimen 2	17.45	1.75	22.23	2.21
Specimen 3	18.35	1.97	24.32	1.85
average	18.64	1.78	22.69	1.95

Table 3.5; compressive and tensile sterength for BNAS 12.5mm

Specimen with nominal agg. Size 12.5mm	14th date value of strength		Test date value of strength	
	Compressive strength(Mpa)	Tensile strength(Mpa)	Compressive strength(Mpa)	Tensile strength(Mpa)
Specimen 1	19.12	1.67	22.04	1.52
Specimen 2	20.36	1.69	20.26	1.91
Specimen 3	19.24	1.52	21.87	2.09
Average	19..57	1.62	21.39	1.84

Table 3.6; compressive and tensile sterength for BNAS 4.75mm

Specimen with nominal agg. Size 4.75mm	14^{th} date value of strength		Test date value of strength	
	Compressive strength(Mpa)	Tensile strength(Mpa)	Compressive strength(Mpa)	Tensile strength(Mpa)
Specimen 1	20.18	1.52	21.12	1.67
Specimen 2	18.67	1.43	23.26	2.08
Specimen 3	19.69	1.31	24.35	1.78
Average	19.51	1.42	22.91	1.84

3.1.3.2 Reinforcing steel properties

The reinforcements of each test specimens were from the same company which is from brothers steel industry the tensile strength for all types of reinforcements were checked at the construction material testing laboratory of Addis Ababa institute of technology (AAiT). Diameter 6 plain bar was used for lateral reinforcement which was bended 135^0 to attain its maximum anchorage. Diameter 16 deformed bar was used for top reinforcement as compression bar. And diameter 20 deformed bar was used for bottom tension reinforcement. Their property based on test result is provided on a tabular form.



Figure 3.4: tensile strength testing machine for steel

Table 3.7: properties of reinforcing steel for the experiment

Specimen No	diameter		Yield load(KN)	Failure load(KN)	Elongation (%)	Mass/length (kg/m)	Strain (ε)(mm)
	D1(mm)	D2(mm)					
6-1	6.17	6.17	5.80	10.20	40	0.223	0.400
6-2	6.06	6.06	6.20	11.40	53.3	0.231	0.533
6-3	6.22	6.22	7.20	11.00	53.3	0.225	0.533
16-1	15.47	17.58	95.10	125.80	25	1.581	0.250
16-2	16.21	17.19	104.50	124.10	18.75	1.536	0.187
16-3	15.45	17.25	96.30	119.10	25	1.551	0.250
20-1	19.47	21.23	151.32	201.80	28	2.469	0.280
20-2	19.49	21.41	150.00	201.41	28	2.455	0.280
20-3	19.65	21.27	153.00	206.20	25	2.478	0.250

Specimen production

This section describes in brief the details of the specimen construction process. The test specimens were produced at the Construction Materials Laboratory. The reinforcement cages were bended and tide with prepared stirrup by using double black wire as per the design. Figure 3.5 shows the steel cage after assembly.



Figure 3.5: Reinforcement cage and formwork of the specimen

The formworks were made from sawn zigba and metal plate stiffened at the mid span to avoid bulging of concrete during casting. After the form work was fixed, form releasing agent was applied on the inner surface of the forms for easy stripping of formwork. Once the formwork was ready, the steel cages were placed inside, maintaining a concrete clear cover of 25 mm on all sides as shown in Figure 3.6.



Figure 3.6: Reinforcement cage of specimens inside the formwork

The concrete were casted on the space provide just in front of the construction material laboratory. Sufficient vibration were applied to the concrete to avoid segregation as well as separation. To check the compressive strength, slump test were carried just before casting as shown on Figure 3.7. After casting sufficient watering were introduced for all specimens.



Figure 3.7: slump test of the concrete during casting.

3.1.4 Testing procedure

Testing procedure of the experimental specimens accommodates a series of events which are

- Applying a white paint on the hardened concrete surface and drawing grid lines on the painted surface.
- Fixing the support on the manually loading machine.
- Placing the specimens on the roller support which was arranged as the span length of the specimens.
- Fixing the loading apparatus (load cell) and the deflection measuring instrument (the transducer) to read the load increments and center line deflection of the specimen respectively. Which are apparently recorded on the data logger.
- Loading the specimens manually by using a hydraulic jack and simultaneously load increment and center line deflection were recorded by the data logger.
- During load increment the instant cracking load and crack pattern were written and drawn on the painted surface.

3.1.5 Test set up and loading apparatus and instrumentation

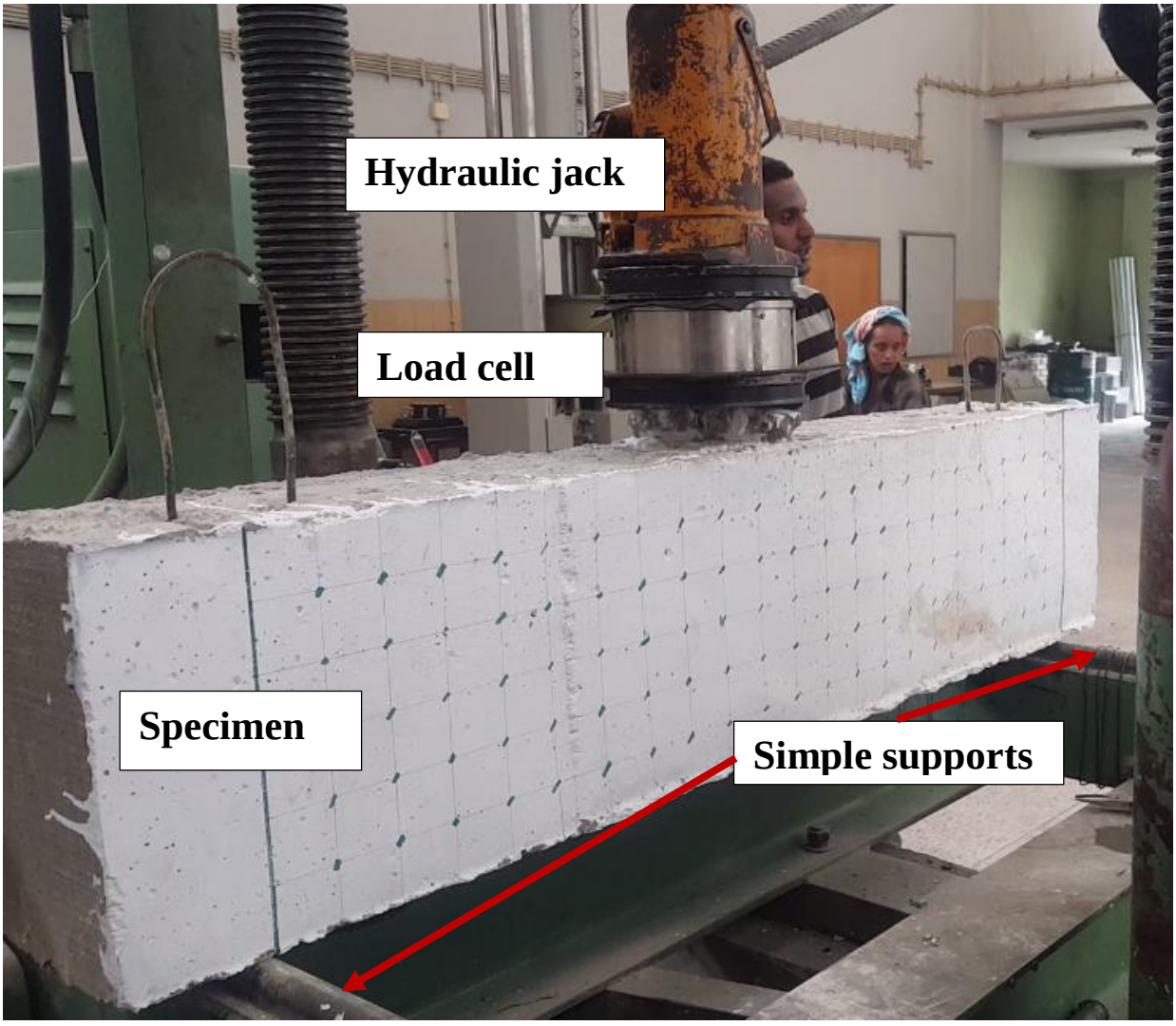


Figure 3.8: Test set up and loading on the specimen.

3.1.5.1 Loading and load measuring instruments.



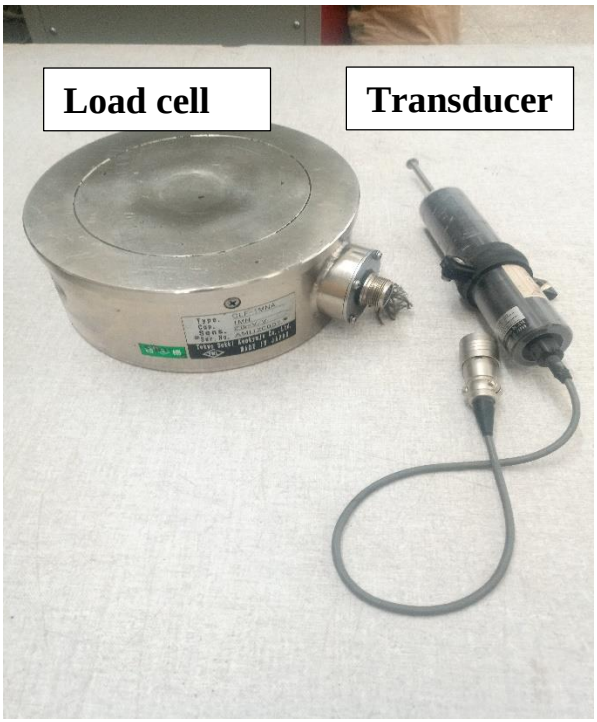




Figure 3.9: Loading and load measuring instruments

CHAPTER FOUR

4 EXPERIMENTAL RESULT AND DISCUSSION

4.1 Observations

During the experimental work a large due care has been taken to control the controlled variables and to set free the uncontrolled variables. In the testing procedure precise result form of failure had occurred in all four groups of specimens (i.e. diagonal shear compression failure). Two specimens was tasted for each groups of aggregates. The following figures shows the failure of each aggregate sizes and load deflection curve of each groups of aggregates.



Figure 4.1: shear failure of BNAS 37.5mm



Figure 4.2: shear failure of BNAS 25mm

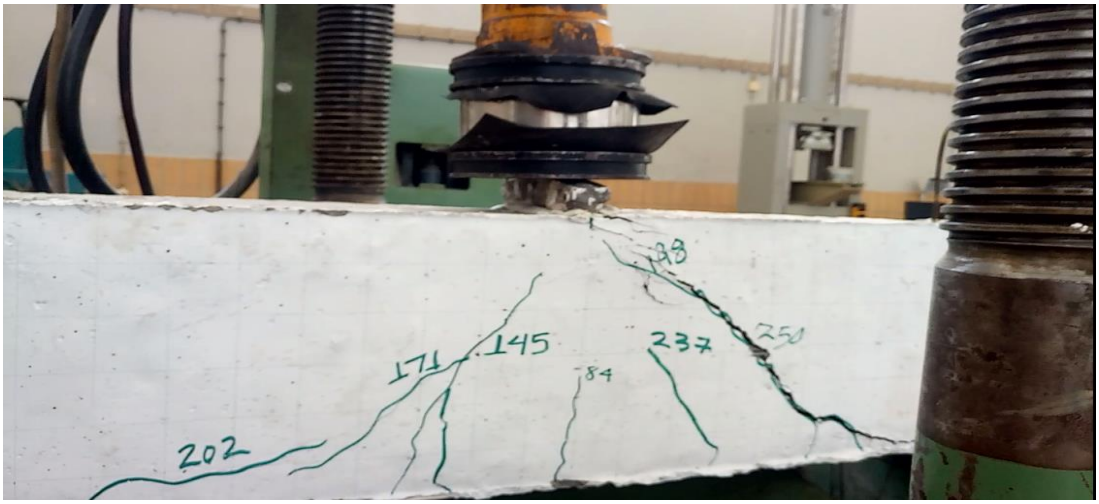


Figure 4.3: shear failure of BNAS 12.5



Figure 4.4: shear failure of BNAS 4.75

The figures above would clearly express the loading and the setup on the test specimens in which the center line deflection and the load increments are recorded carefully by using the data loger and the result of each category is plotted by using excel. And finally their change is compared with one another as a result their final difference including post peak result is plotted by using excel.

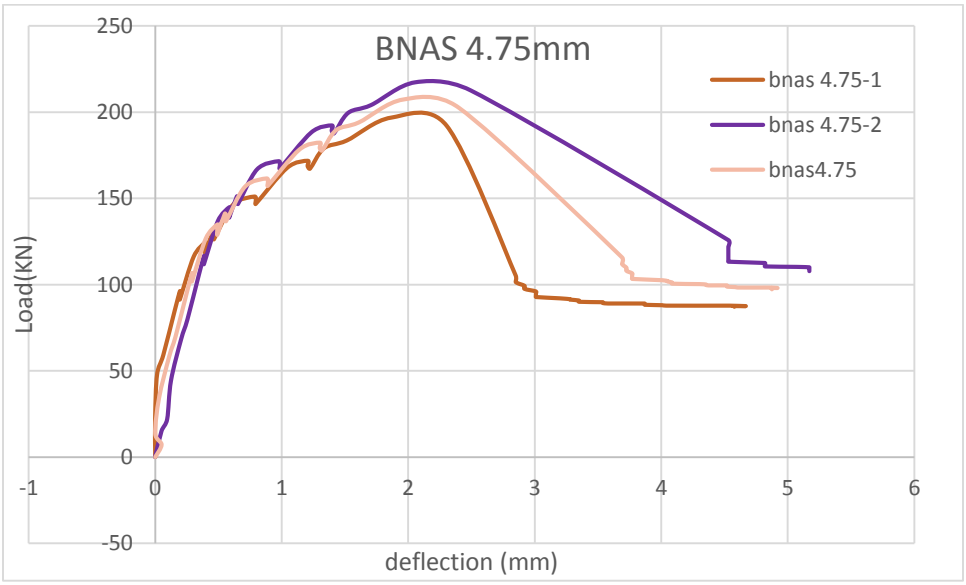


Figure 4.5: load deflection diagram of BNAS 4.75mm

- The designed shear resistance of the specimens according to EBCS-2 1995 were **180kN**
- The actual failure load from the data logger **207.33kN**

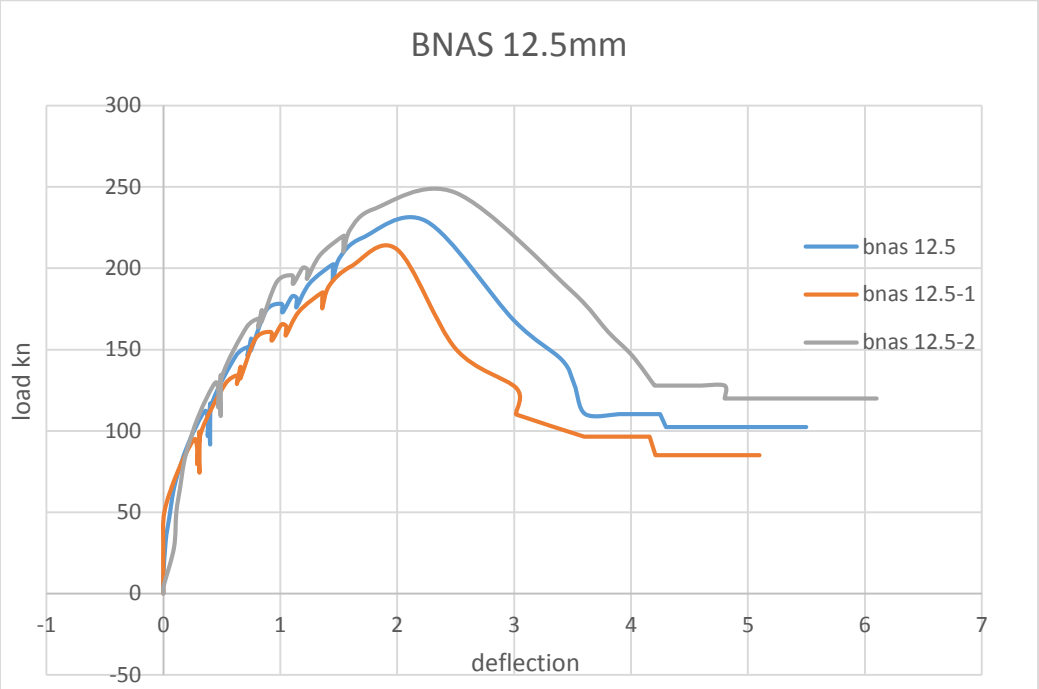


Figure 4.6: load deflection diagram of BNAS 12.5mm

- The designed shear resistance of the specimens according to EBCS-2 1995 were **180kN**
- The actual failure load from the data logger **228.84kN**

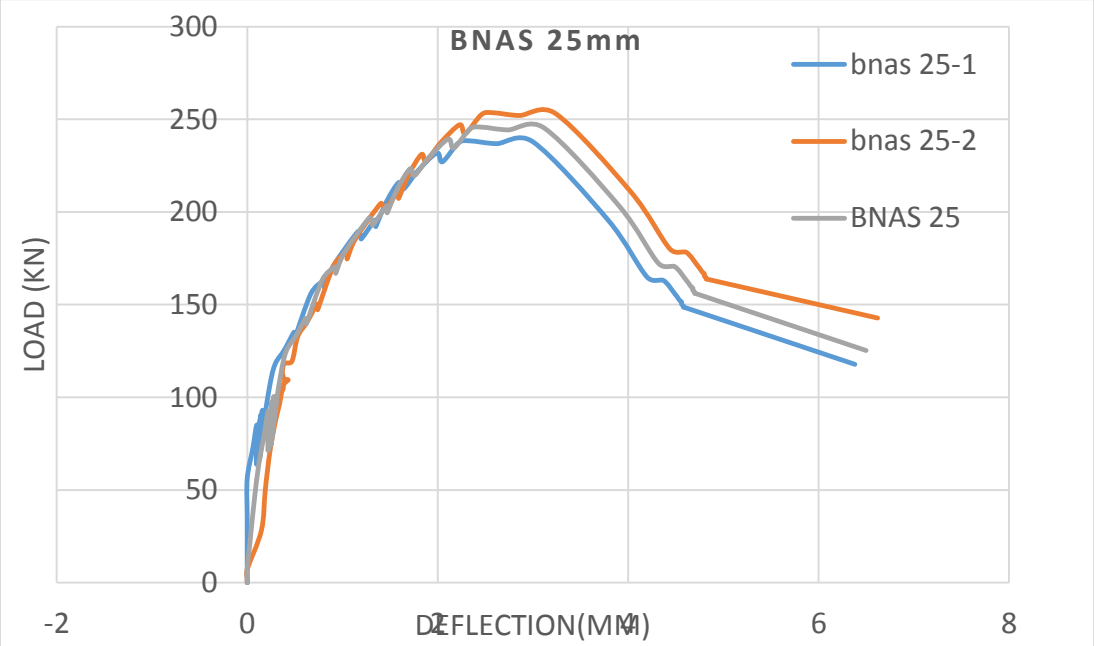


Figure 4.7: load deflection diagram of BNAS 25mm

- The designed shear resistance of the specimens according to EBCS-2 1995 were **180kN**
- The actual failure load from the data logger **245.85kN**

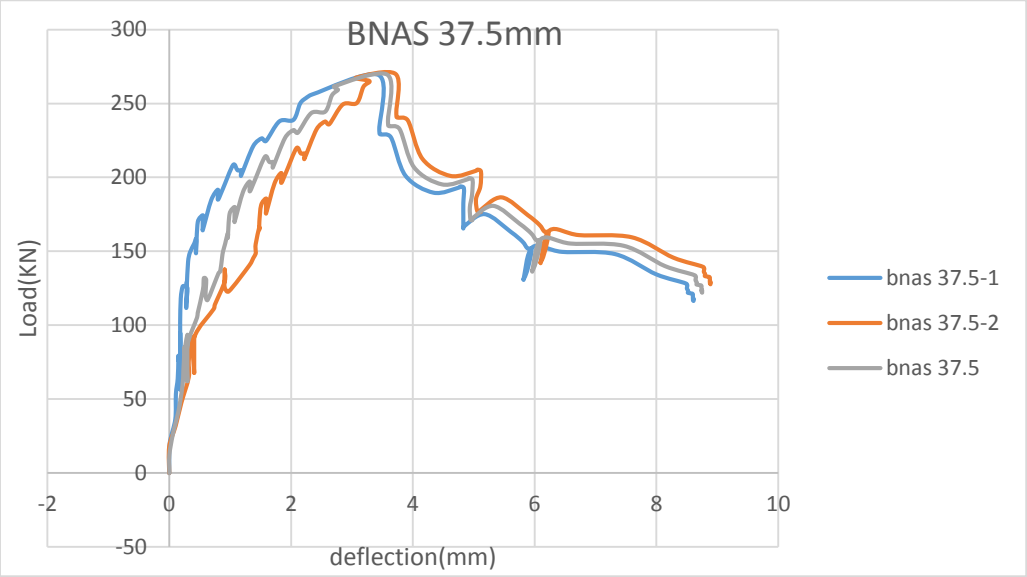


Figure 4.8: load deflection diagram of BNAS 37.5mm

- The designed shear resistance of the specimens according to EBCS-2 1995 were **180kN**
- The actual failure load from the data logger **269.8kN**

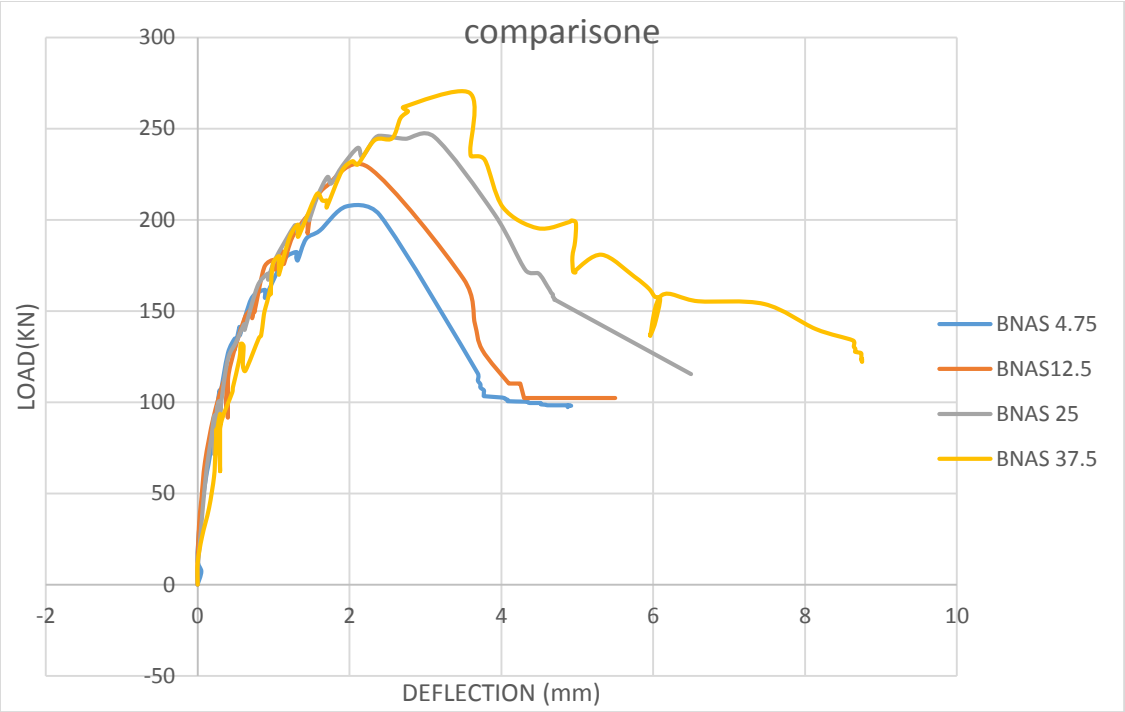


Figure 4.9: comparison of load-deflection diagrams between each aggregate size

- Consider the two extreme graphs
  - pre-peak behavior similarity
  - A visible change in peak load

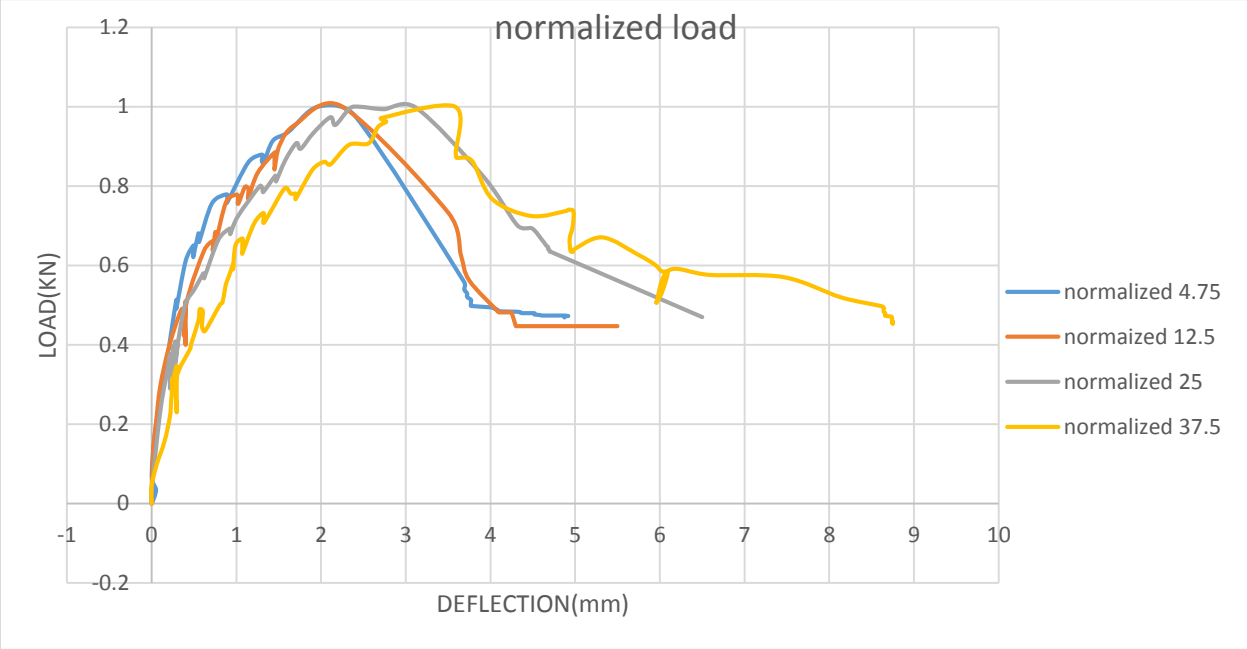


Figure 4.10: Comparison of load-deflection diagrams between each aggregate size with normalized load

- Graph plotted to check the ductility
  - post-peak behavior difference
  - A visible change in post-peak load

4.2 Summary of results

4.2.1 Crack surface roughness.

Roughness of the crack surfaces increases, as it was discussed in the literature review allowing higher shear stresses to be transferred across the cracks. In high strength concrete beams and some light weight concrete beams, the cracks penetrate pieces of the aggregate rather than going around them, resulting in a smoother crack surface. This decrease in the shear transferred by aggregate interlock along the cracks reduces Vc. Roughness of the cracked surface of the specimens used in this experiment is shown in Figures bellow.



Figure 4.11: crack surface of BNAS 4.75mm



Figure 4.12: crack surface of BNAS 12.5mm



Figure 4.13: crack surface of BNAS 12.5mm



Figure 4.14: crack surface of BNAS 12.5mm

4.2.2 Effects of aggregate size

In the case of homogeneous material (in fact concrete by itself is not homogeneous to some extent) shear cracks develop due to principal tensile stresses that exceed the tensile strength of the concrete. But in the case of reinforced concrete the case is quite different because there is a possible dalliance of crack initiation due to the reinforcement. But in this case the percentage of the reinforcement in each specimens were kept constant so that the crack initiation load would possibly the same. So that the crack incitation depends on the tensile strength of concrete. In fact the compressive strength of each specimens were kept similar but there is a clear increase in tensile strength as the aggregate size increase. Based on this the lab results indicates;

- ✓ Specimens with larger aggregate size were absorbing more loads before failure.
- ✓ For specimens with smaller aggregate size there were sudden wide crack opening during loading.
- ✓ After failure it was noticed that the cracks were following the path of the aggregate rather than crossing them.
- ✓ Specimens with larger aggregate size were able to sustain more loads after the peak points.
- ✓ Prior to testing of the specimens there were a little increase in tensile strength of the concrete, during splitting test of concrete.

CHAPTER FIVE

5 CONCLUSION AND RECOMMENDATION

5.1 Conclusion

Shear failure of short beams is one of the representative type for the demonstration of a typical diagonal shear compression failure. Based on this it was explored the participants of shear failures, so that we can handle the problem of shear failure without causing any damage for the occupants. So that the following conclusions were forwarded.

- As the aggregate size increases there is a possible increase in shear strength of the beam.
- Importantly the crack surface roughness increases as the aggregate size increases in NSC
- The pre-peak behavior for all aggregates used in this experiment were almost similar therefore the pre-peak behavior remain an affected by changing the aggregate size.
- Size effect of the beam such as shear span to depth ratio, beam cross-section has an inverse relation with shear strength though they have web reinforcement.
- Most codes including Ethiopian building code doesn't give explicit value for the aggregate size, in fact it has an impact on shear strength but lump it with concrete compressive strength.

5.2 Recommendation

1. The building codes should constitute the aggregate size effect for shear strength of beams in contrast now a days smaller aggregate sizes are used in order to solve reinforcement congestion, which would possibly reduce the aggregate interlock.
2. A possible and an allowable maximum aggregate size should be used in order to enhance the shear strength of reinforced concrete beams with web reinforcement.
3. Other impacts like aggregate type and aggregate shape effects on shear strength of reinforced concrete beams with web reinforcement should be checked as further study in the future.
4. The interaction of aggregates size and web reinforcement should be checked in order to find distinguished relations of the two parameters.
5. A further study on tensile strength of concrete and aggregate size should carried on larger sets of specimens.

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Appendix

Mix-design procedure of concrete

A complete proportioning of constitutes of concrete for the intended compressive strength of concrete were held before the commencement of experimental work. It constitutes;

1. Determining properties of the constitute components of concrete (i.e. cement, course aggregate, fine aggregate and water.
2. Proportioning of constitutes of concrete by mass.

Properly proportioned concrete is expressed in the following qualities;

a. Acceptable workability of the freshly mixed concrete
b. Durability, strength and uniform appearance of the hardened concrete and
c. Economy

Trail Mix- Design Using ACI Methods for Conventional Concrete (C25 with a

Maximum size of aggregate=37.5mm).

Materials properties:-

Concrete with compressive strength of 25 Mpa (C-25) is required to be produced.

- from materials quality test we have the following result based on construction material laboratory test by Abebe Dinku.

a. Cement type I; specific gravity=3.15
b. Coarse aggregates';
   - Bulk specific gravity=2.822
   - Absorption capacity=1.41%
   - Moisture content= 1.62%
   - Compacted unit weight=1716.54KN/m^3
c. Fine aggregate
   - Bulk specific gravity=2.514
   - Absorption capacity=0.956%
   - Moisture content=2.35%
   - Fines modules=2.9

- Based on this the mix design procedures are as follows.

1. Choice of the slump

Table 1 Recommended slumps for various types of construction (ACI-2111_91)

Types of construction	Maximum slump(mm)	Minimum slump(mm)
Reinforced foundation walls and footings	75	25
Plain footings, caissons, and substructure walls	75	25
Beams and reinforced walls	100	25
Building columns	100	25
Pavements and slabs	75	25
Mass concrete	75	25

From table 1, take that a type of construction is beams and reinforced wall.

This implies, maximum slump= 100mm and minimum slump= 25mm.

2. Maximum size of aggregate
   - ✓ Fine aggregate=4.75mm
   - ✓ Coarse aggregate=37.5mm
3. Estimation of mixing water and air content
   Assume non air entrained concrete
   Table 2 Approximate Mixing Water and Air Content Requirements for Different Slumps and Nominal Sizes of Aggregates (ACI-2111_91)

NON-AIR-ENTRAINED CONCRETE								
Slump (mm)	**9.5mm**	**12.5mm**	**19mm**	**25mm**	**37.5mm**	**50mm**	**75mm**	**150mm**
25 to 50	207	199	190	179	166	154	130	113
75 to 100	228	216	205	193	181	169	145	124
150 to 175	243	228	216	202	190	178	160	-
More than 175	-	-	-	-	-	-	-	-
Approximate amount of entrapped air in non-air-entrained concrete (%)								
Slump (mm)	**9.5mm**	**12.5mm**	**19mm**	**25mm**	**37.5mm**	**50mm**	**75mm**	**150mm**
All	3.0	2.5	2	1.5	1	0.5	0.3	0.2

   From table 2 of ACI standards, for non-air entrained and slump of 25-50, the water in $1m^3$ of concrete is 166Kg.

4. Water to cement selection table 3

Table 3 Relationship between water-cement or water-cementations materials ratio and

Compressive strength of concrete (ACI-2111_91)

Compressive strength at 28 days (MPa)	Water-cement ratio by weight (Non-air-entrained concrete)
40	0.42
35	0.47
30	0.54
25	0.61
20	0.69
15	0.79

From table 3, for C -25 and non-air entrained concrete water to cement ratio is 0.61.

5. Cement content calculation
   For slump of 25-50
   Water content=166Kg/m³
   Water/cement=0.61

$$cementcontent = \frac{watercontent}{\frac{w}{c}}$$

$$= \frac{166}{0.61}$$

$$= \mathbf{272.13 kg/m^3}$$

6. Estimation of coarse aggregates content (table 4)

Table 4 Volume of oven-dry-rodded coarse aggregate per unit volume of concrete for different fineness moduli of fine aggregate.(ACI -2111_91)

Nominal maximum size of aggregate (mm)	2.4	2.6	2.8	3.0
9.5	0.50	0.48	0.46	0.44
12.5	0.59	0.57	0.55	0.53
19	0.66	0.64	0.62	0.60
25	0.71	0.69	0.67	0.65
37.5	0.75	0.73	0.71	0.69
50	0.78	0.76	0.74	0.72
75	0.82	0.80	0.78	0.76
150	0.87	0.85	0.83	0.81

From table 4 of ACI, for maximum size of aggregate= 37.5mm and fines modules for sand = 2.9, the volume of course aggregate per unit volume of concrete is

$$= 0.7$$

$$required dry mass of C.A = 0.7 * 1716.54$$

$$= \mathbf{1201.58 kg/m3}$$

7. Estimation of fine aggregates content

At the end of Step 6, all ingredients of the concrete have been estimated except the fine aggregate. Its quantity is determined by making difference. For this process either of the two procedures may be employed:

- the weight method
- the absolute volume method

If the weight of the concrete per unit volume is assumed or can be estimated from experience, the required weight of fine aggregate is simply the difference between the weight of fresh concrete and the total weight of the other ingredients. Therefore, I n this paper the weight method preferred.

Fine aggregate content (F.A)=unit weight of concrete-(C.A+cement+water)

First estimate the unit weight of fresh concretes from table 5.

Table 5 First estimate of concrete weight (kg/m3)

Nominal maximum size of aggregate (mm)	Non-air-entrained concrete
9.5	2280
12.5	2310
19	2345
25	2380
37.5	2410
50	2445
75	2490
150	2530

From the table 5 the u nit weight of fresh concrete corresponding to max. Aggregate size of 37.5mm and non- air entrained is **2410kg/m^3**.

Fine aggregate content (F.A)=2410-(1201.58+272.13+166)=**770.42kg/m^3**

8. moisture adjustment

Absorbed water does not become part of the mixing water and it must be removed from the mixing water, if moisture content is greater than absorption capacity. But, if absorption capacity is greater than moisture content of aggregate, we need to add water up to its moisture capacity. Therefore, in this case since the moisture content of the aggregates are greater than their absorption capacity, water should be deducted from the mixing water.

Removed water from C.A=1.62+1.41=**0.21%**

Removed water from F.A=2.35-0.956=**1.394%**

$$total\ water\ required = 166 - \left[1201.58 * \left(\frac{0.21}{100}\right) + 770.42 * \left(\frac{1.394}{100}\right)\right]$$

=152.728kg/m^3

The estimated ingredient of concrete for nominal aggregate size of 37.5mm in one meter cube are summarized as follows.

Ingredients	Weight per meter cube(kg/m^3)
Coarse aggregate	1201.58
Fine aggregate	770.42
Cement	272.13
Water	152.728
Concrete unit weight.	**2396.858**

9. Trial batch

Six cubes which are 150x150x150mm^3 and six cylinders which have a diameter of 150mm by a depth of 300mm are required for compressive and tensile strength test respectively. Having total volume of 0.1263m^3

Specimen beams concrete volume= 2*1.3*.3*.2=0.195

Sub-total=0.195+0.1263=0.3213m^3

Considering 5% west=0.016 m^3

Total volume=0.3213+0.01606=0.3374 m^3

Ingredients	Weight in kg
Coarse aggregate	1201.58*.3374=405.413
Fine aggregate	770.47*.3374=259.956
Cement	272.13*.3374=91.817
Water	152.72*.3374=51.680

Trail Mix- Design Using ACI Methods for Conventional Concrete (C25 with a Maximum size of aggregate=25mm).

Material properties:-

Concrete with compressive strength of 25 Mpa (C-25) is required to be produced.

- from materials quality test we have the following result based on construction material laboratory test by Abebe Dinku.

a. Cement type 1; specific gravity=3.15
b. Coarse aggregates';
    a. Bulk specific gravity=2.822
    b. Absorption capacity=1.41%
    c. Moisture content= 1.62%
    d. Compacted unit weight=1716.54KN/m^3
c. Fine aggregate
    - Bulk specific gravity=2.514
    - Absorption capacity=0.956%
    - Moisture content=2.35%
    - Fines modules=2.9

- Based on this the mix design procedures are as follows.
    1. Choice of the slump

Table 1 Recommended slumps for various types of construction (ACI-2111_91)

Types of construction	Maximum slump(mm)	Minimum slump(mm)
Reinforced foundation walls and footings	75	25
Plain footings, caissons, and substructure walls	75	25
Beams and reinforced walls	100	25
Building columns	100	25
Pavements and slabs	75	25
Mass concrete	75	25

From table 1, take that a type of construction is beams and reinforced wall.

This implies, maximum slump= 100mm and minimum slump= 25mm.

2. Maximum size of aggregate
   - ✓ Fine aggregate=4.75mm
   - ✓ Coarse aggregate=25mm
3. Estimation of mixing water and air content

Assume non air entrained concrete

Table 2 Approximate Mixing Water and Air Content Requirements for Different Slumps and Nominal Sizes of Aggregates (ACI-2111_91)

NON-AIR-ENTRAINED CONCRETE								
Slump (mm)	**9.5mm**	**12.5mm**	**19mm**	**25mm**	**37.5mm**	**50mm**	**75mm**	**150mm**
25 to 50	207	199	190	179	166	154	130	113
75 to 100	228	216	205	193	181	169	145	124
150 to 175	243	228	216	202	190	178	160	-
More than 175	-	-	-	-	-	-	-	-
Approximate amount of entrapped air in non-air-entrained concrete (%)								
Slump (mm)	**9.5mm**	**12.5mm**	**19mm**	**25mm**	**37.5mm**	**50mm**	**75mm**	**150mm**
All	3.0	2.5	2	1.5	1	0.5	0.3	0.2

From table 2 of ACI standards, for non-air entrained and slump of 25-50, the water in $1m^3$ of concrete is 179Kg.

4. Water to cement selection table 3

Table 3 Relationship between water-cement or water-cementations materials ratio and Compressive strength of concrete (ACI-2111_91)

Compressive strength at 28 days (MPa)	Water-cement ratio by weight (Non-air-entrained concrete)
40	0.42
35	0.47
30	0.54
25	0.61
20	0.69
15	0.79

From table 3, for C -25 and non-air entrained concrete water to cement ratio is 0.61.

5. Cement content calculation

For slump of 25-50

Water content=179Kg/m³

Water/cement=0.61

$$cementcontent = \frac{watercontent}{\frac{w}{c}}$$

$$= \frac{179}{0.61}$$

=**293.443kg/m³**

6. Estimation of coarse aggregates content (table 4)

Table 4 Volume of oven-dry-rodded coarse aggregate per unit volume of concrete for different fineness moduli of fine aggregate.(ACI -2111_91)

Nominal maximum size of aggregate (mm)	2.4	2.6	2.8	3.0
9.5	0.50	0.48	0.46	0.44
12.5	0.59	0.57	0.55	0.53
19	0.66	0.64	0.62	0.60
25	0.71	0.69	0.67	0.65
37.5	0.75	0.73	0.71	0.69
50	0.78	0.76	0.74	0.72
75	0.82	0.80	0.78	0.76
150	0.87	0.85	0.83	0.81

From table 4 of ACI, for maximum size of aggregate= 25mm and fines modules for sand = 2.9, the volume of course aggregate per unit volume of concrete is

$$= 0.66$$

$$requireddrymassofC.A = 0.66 * 1716.54$$

$$= \mathbf{1132.916}\boldsymbol{kg/m3}$$

7. Estimation of fine aggregates content

At the end of Step 6, all ingredients of the concrete have been estimated except the fine aggregate. Its quantity is determined by making difference. For this process either of the two procedures may be employed:

- the weight method
- the absolute volume method

If the weight of the concrete per unit volume is assumed or can be estimated from experience, the required weight of fine aggregate is simply the difference between the weight of fresh concrete and the total weight of the other ingredients. Therefore, I n this paper the weight method preferred.

Fine aggregate content (F.A)=unit weight of concrete-(C.A+cement+water)

First estimate the unit weight of fresh concretes from table 5.

Table 5 First estimate of concrete weight (kg/m3)

Nominal maximum size of aggregate (mm)	Non-air-entrained concrete
9.5	2280
12.5	2310
19	2345
25	2380
37.5	2410
50	2445
75	2490
150	2530

From the table 5 the u nit weight of fresh concrete corresponding to max. Aggregate size of 25mm and non- air entrained is **2380kg/m^3**.

Fine aggregate content (F.A)=2380-(1132.916+293.443+179)**=774.64kg/m^3**

8. moisture adjustment

Absorbed water does not become part of the mixing water and it must be removed from the mixing water, if moisture content is greater than absorption capacity. But, if absorption capacity is greater than moisture content of aggregate, we need to add water up to its moisture capacity. Therefore, in this case since the moisture content of the aggregates are greater than their absorption capacity, water should be deducted from the mixing water.

Removed water from C.A=1.62+1.41=**0.21%**

Removed water from F.A=2.35-0.956=**1.394%**

$$total\ water\ required = 179 - \left[1132.916 * \left(\frac{0.21}{100}\right) + 774.64 * \left(\frac{1.394}{100}\right)\right]$$

=165.822kg/m^3

The estimated ingredient of concrete for nominal aggregate size of 25mm in one meter cube are summarized as follows.

Ingredients	Weight per meter cube(kg/m^3)
Coarse aggregate	1132.916
Fine aggregate	774.64
Cement	293.443
Water	165.822
Concrete unit weight.	**2366.821**

9. Trial batch

Six cubes which are 150x150x150mm^3 and six cylinders which have a diameter of 150mm by a depth of 300mm are required for compressive and tensile strength test respectively. Having total volume of 0.1263m^3

Specimen beams concrete volume= 2*1.3*.3*.25=0.195

Sub-total=0.195+0.1263=0.3213m^3

Considering 5% west=0.016 m^3

Total volume=0.3213+0.01606=0.3374 m^3

Ingredients	Weight in kg
Coarse aggregate	1132.916*.3374=382.246
Fine aggregate	774.443*.3374=261.297
Cement	272.13*.3374=91.81
Water	165.822*.3374=55.948

Trail Mix- Design Using ACI Methods for Conventional Concrete (C25 with a Maximum size of aggregate=12.5mm).

Material properties:-

Concrete with compressive strength of 25 Mpa (C-25) is required to be produced.

- from materials quality test we have the following result based on construction material laboratory test by Abebe Dinku.

d. Cement type 1; specific gravity=3.15
e. Coarse aggregates';
    a. Bulk specific gravity=2.822
    b. Absorption capacity=1.41%
    c. Moisture content= 1.62%
    d. Compacted unit weight=1716.54KN/m^3
f. Fine aggregate
    - Bulk specific gravity=2.514
    - Absorption capacity=0.956%
    - Moisture content=2.35%
    - Fines modules=2.9

- Based on this the mix design procedures are as follows.

1. choice of the slump

Table 1 Recommended slumps for various types of construction (ACI-2111_91)

Types of construction	**Maximum slump(mm)**	**Minimum slump(mm)**
Reinforced foundation walls and footings	75	25
Plain footings, caissons, and substructure walls	75	25
Beams and reinforced walls	100	25
Building columns	100	25
Pavements and slabs	75	25
Mass concrete	75	25

From table 1, take that a type of construction is beams and reinforced wall.

This implies, maximum slump= 100mm and minimum slump= 25mm.

2. Maximum size of aggregate
    - ✓ Fine aggregate=4.75mm
    - ✓ Coarse aggregate=12.5mm

3. Estimation of mixing water and air content

Assume non air entrained concrete

Table 2 Approximate Mixing Water and Air Content Requirements for Different Slumps and Nominal Sizes of Aggregates (ACI-2111_91)

NON-AIR-ENTRAINED CONCRETE								
Slump (mm)	**9.5mm**	**12.5mm**	**19mm**	**25mm**	**37.5mm**	**50mm**	**75mm**	**150mm**
25 to 50	207	199	190	179	166	154	130	113
75 to 100	228	216	205	193	181	169	145	124
150 to 175	243	228	216	202	190	178	160	-
More than 175	-	-	-	-	-	-	-	-
Approximate amount of entrapped air in non-air-entrained concrete (%)								
Slump (mm)	**9.5mm**	**12.5mm**	**19mm**	**25mm**	**37.5mm**	**50mm**	**75mm**	**150mm**
All	3.0	2.5	2	1.5	1	0.5	0.3	0.2

From table 2 of ACI standards, for non-air entrained and slump of 25-50, the water in $1m^3$ of concrete is 199Kg.

4. Water to cement selection table 3

Table 3 Relationship between water-cement or water-cementations materials ratio and Compressive strength of concrete (ACI-2111_91)

Compressive strength at 28 days (MPa)	**Water-cement ratio by weight (Non-air-entrained concrete)**
40	0.42
35	0.47
30	0.54
25	0.61
20	0.69
15	0.79

From table 3, for C -25 and non-air entrained concrete water to cement ratio is 0.61.

5. Cement content calculation

For slump of 25-50

Water content=199Kg/m³

Water/cement=0.61

$$cementcontent = \frac{watercontent}{\frac{w}{c}}$$

$$= \frac{199}{0.61}$$

$$\underline{=\mathbf{326.229kg/m^3}}$$

6. Estimation of coarse aggregates content (table 4)

Table 4 Volume of oven-dry-rodded coarse aggregate per unit volume of concrete for different fineness moduli of fine aggregate.(ACI -2111_91)

Nominal maximum size of aggregate (mm)	**2.4**	**2.6**	**2.8**	**3.0**
9.5	0.50	0.48	0.46	0.44
12.5	0.59	0.57	0.55	0.53
19	0.66	0.64	0.62	0.60
25	0.71	0.69	0.67	0.65
37.5	0.75	0.73	0.71	0.69
50	0.78	0.76	0.74	0.72
75	0.82	0.80	0.78	0.76
150	0.87	0.85	0.83	0.81

From table 4 of ACI, for maximum size of aggregate= 25mm and fines modules for sand = 2.9, the volume of course aggregate per unit volume of concrete is

$$= 0.54$$

$$required dry mass of C.A = 0.54 * 1716.54$$

$$= \mathbf{926.931 kg/m3}$$

7. Estimation of fine aggregates content

At the end of Step 6, all ingredients of the concrete have been estimated except the fine

Aggregate. Its quantity is determined by making difference. For this process either of the two procedures may be employed:

- ❖ the weight method
- ❖ the absolute volume method

If the weight of the concrete per unit volume is assumed or can be estimated from experience, the required weight of fine aggregate is simply the difference between the weight of fresh concrete and the total weight of the other ingredients. Therefore, I n this paper the weight method preferred.

Fine aggregate content (F.A)=unit weight of concrete-(C.A+cement+water)

First estimate the unit weight of fresh concretes from table 5.

Table 5 First estimate of concrete weight (kg/m3)

Nominal maximum size of aggregate (mm)	**Non-air-entrained concrete**
9.5	2280
12.5	2310
19	2345
25	2380
37.5	2410
50	2445
75	2490
150	2530

From the table 5 the u nit weight of fresh concrete corresponding to max. Aggregate size of 25mm and non- air entrained is **2310kg/m^3**.

Fine aggregate content (F.A)=2310-(926.931+326+199)=**858.069kg/m^3**

8. moisture adjustment

Absorbed water does not become part of the mixing water and it must be removed from the mixing water, if moisture content is greater than absorption capacity. But, if absorption capacity is greater than moisture content of aggregate, we need to add water up to its moisture capacity. Therefore, in this case since the moisture content of the aggregates are greater than their absorption capacity, water should be deducted from the mixing water.

Removed water from C.A=1.62+1.41=**0.21%**

Removed water from F.A=2.35-0.956=**1.394%**

$$total\ water\ required = 199 - \left[926.931 * \left(\frac{0.21}{100}\right) + 858.069 * \left(\frac{1.394}{100}\right)\right]$$

=180.406kg/m^3

The estimated ingredient of concrete for nominal aggregate size of 12.5mm in one meter cube are summarized as follows.

Ingredients	**Weight per meter cube(kg/m^3)**
Coarse aggregate	926.931
Fine aggregate	858.069
Cement	326.229
Water	180.406
Concrete unit weight.	**2291.635**

9. Trial batch

Six cubes which are 150x150x150mm^3 and six cylinders which have a diameter of 150mm by a depth of 300mm are required for compressive and tensile strength test respectively. Having total volume of 0.1263m^3

Specimen beams concrete volume= 2*1.3*.3*.25=0.195

Sub-total=0.195+0.1263=0.3213m^3

Considering 5% west=0.016 m^3

Total volume=0.3213+0.01606=0.3374 m^3

Ingredients	Weight in kg
Coarse aggregate	926.931*.3374=312.746
Fine aggregate	858.069*.3374=289.512
Cement	326.229*.3374=110.069
Water	180.406*.3374=60.86

Trail Mix- Design Using ACI Methods for Conventional Concrete (C25 with a Maximum size of aggregate=4.75mm).

Material properties:-

Concrete with compressive strength of 25 Mpa (C-25) is required to be produced.

- from materials quality test we have the following result based on construction material laboratory test by Abebe Dinku.

a. Cement type I; specific gravity=3.15
b. Coarse aggregates';
    i. Bulk specific gravity=2.822
    ii. Absorption capacity=1.41%
    iii. Moisture content= 1.62%
    iv. Compacted unit weight=1716.54KN/m^3
c. Fine aggregate
    - Bulk specific gravity=2.514
    - Absorption capacity=0.956%
    - Moisture content=2.35%
    - Fines modules=2.9

- Based on this the mix design procedures are as follows.

1. Choise of the slump

Table 1 Recommended slumps for various types of construction (ACI-2111_91)

Types of construction	Maximum slump(mm)	Minimum slump(mm)
Reinforced foundation walls and footings	75	25
Plain footings, caissons, and substructure walls	75	25
Beams and reinforced walls	100	25
Building columns	100	25
Pavements and slabs	75	25
Mass concrete	75	25

From table 1, take that a type of construction is beams and reinforced wall.

This implies, maximum slump= 100mm and minimum slump= 25mm.

2. Maximum size of aggregate
   - ✓ Fine aggregate=4.75mm
   - ✓ Coarse aggregate=12.5mm
3. Estimation of mixing water and air content

Assume non air entrained concrete

Table 2 Approximate Mixing Water and Air Content Requirements for Different Slumps and Nominal Sizes of Aggregates (ACI-2111_91)

NON-AIR-ENTRAINED CONCRETE								
Slump (mm)	**9.5mm**	**12.5mm**	**19mm**	**25mm**	**37.5mm**	**50mm**	**75mm**	**150mm**
25 to 50	207	199	190	179	166	154	130	113
75 to 100	228	216	205	193	181	169	145	124
150 to 175	243	228	216	202	190	178	160	-
More than 175	-	-	-	-	-	-	-	-
Approximate amount of entrapped air in non-air-entrained concrete (%)								
Slump (mm)	**9.5mm**	**12.5mm**	**19mm**	**25mm**	**37.5mm**	**50mm**	**75mm**	**150mm**
All	3.0	2.5	2	1.5	1	0.5	0.3	0.2

From table 2 of ACI standards, for non-air entrained and slump of 25-50, the water in $1m^3$ of concrete is 216kg.

4. Water to cement selection table 3

Table 3 Relationship between water-cement or water-cementations materials ratio and

Compressive strength of concrete (ACI-2111_91)

Compressive strength at 28 days (MPa)	**Water-cement ratio by weight (Non-air-entrained concrete)**
40	0.42
35	0.47
30	0.54
25	0.61
20	0.69
15	0.79

From table 3, for C -25 and non-air entrained concrete water to cement ratio is 0.61.

5. Cement content calculation

For slump of 75-100

Water content=$216Kg/m^3$

Water/cement=0.61

$$cementcontent = \frac{watercontent}{\frac{w}{c}}$$

$$= \frac{216}{0.61}$$

$$=\underline{\mathbf{354.09kg/m^3}}$$

6. Estimation of course aggregates content (table 4)

Table 4 Volume of oven-dry-rodded coarse aggregate per unit volume of concrete for different fineness moduli of fine aggregate.(ACI -2111_91)

Nominal maximum size of aggregate (mm)	2.4	2.6	2.8	3.0
9.5	0.50	0.48	0.46	0.44
12.5	0.59	0.57	0.55	0.53
19	0.66	0.64	0.62	0.60
25	0.71	0.69	0.67	0.65
37.5	0.75	0.73	0.71	0.69
50	0.78	0.76	0.74	0.72
75	0.82	0.80	0.78	0.76
150	0.87	0.85	0.83	0.81

From table 4 of ACI, for maximum size of aggregate= 4.75 mm and fines modules for sand = 2.9, the volume of course aggregate per unit volume of concrete is

$$= 0..31$$

$$requireddrymassofC.A = 0.54 * 1716.54$$

$$= \mathbf{532.127}kg/m3$$

7. Estimation of fine aggregates content

At the end of Step 6, all ingredients of the concrete have been estimated except the fine aggregate. Its quantity is determined by making difference. For this process either of the two procedures may be employed:

- the weight method
- the absolute volume method

If the weight of the concrete per unit volume is assumed or can be estimated from experience, the required weight of fine aggregate is simply the difference between the weight of fresh concrete and the total weight of the other ingredients. Therefore, I n this paper the weight method preferred.

Fine aggregate content (F.A)=unit weight of concrete-(C.A+cement+water)

First estimate the unit weight of fresh concretes from table 5.

Table 5 First estimate of concrete weight (kg/m3)

Nominal maximum size of aggregate (mm)	Non-air-entrained concrete
9.5	2280
12.5	2310
19	2345
25	2380
37.5	2410
50	2445
75	2490
150	2530

From the table 5 the unit weight of fresh concrete corresponding to max. Aggregate size of 4.75mm and non- air entrained is **2232kg/m^3**.

Fine aggregate content (F.A)=2232-(532.127+354.09+216)=**1129.783kg/m^3**

8. moisture adjustment

Absorbed water does not become part of the mixing water and it must be removed from the mixing water, if moisture content is greater than absorption capacity. But, if absorption capacity is greater than moisture content of aggregate, we need to add water up to its moisture capacity. Therefore, in this case since the moisture content of the aggregates are greater than their absorption capacity, water should be deducted from the mixing water.

Removed water from C.A=1.62+1.41=**0.21%**

Removed water from F.A=2.35-0.956=**1.394%**

$$total\ water\ required = 216 - \left[532 * \left(\frac{0.21}{100}\right) + 1129.35 * \left(\frac{1.394}{100}\right)\right]$$

=199.14kg/m^3

The estimated ingredient of concrete for nominal aggregate size of 4.75mm in one meter cube are summarized as follows.

Ingredients	Weight per meter cube(kg/m^3)
Coarse aggregate	532
Fine aggregate	1129.35
Cement	354.09
Water	199.14
Concrete unit weight.	**2214.58**

9. Trial batch

Six cubes which are 150x150x150mm^3 and six cylinders which have a diameter of 150mm by a depth of 300mm are required for compressive and tensile strength test respectively. Having total volume of 0.1263m^3

Specimen beams concrete volume= 2*1.3*.3*.25=0.195

Sub-total=0.195+0.1263=0.3213m^3

Considering 5% west=0.016 m^3

Total volume=0.3213+0.01606=0.3374 m^3

Ingredients	Weight in kg
Coarse aggregate	532*.3374=179.496
Fine aggregate	1129.35*.3374=381.04
Cement	354.09*.3374=119.17
Water	199.14*.3374=67.189