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Experimental Study on the Effect of Un-Bonded Longitudinal Steel Reinforcement on Flexural Responses of RC Beams

A Thesis in Structural Engineering

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A THESIS SUBMITTED TO THE SCHOOL OF GRADUATE STUDIES OF ADDIS ABABA UNIVERSITY IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE IN CIVIL ENGINEERING

The undersigned have examined the thesis **'Experimental Investigation on the Effect of Un-Bonded Longitudinal Steel Reinforcement on Flexural Responses of RC Beams'** presented by **FEKADU DEBELA**, a candidate for the degree of **Master of Science,** and hereby certify that it is deserving of acceptance.

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UNDERTAKING

I certify that research work titled "**Experimental Investigation on the Effect of Un-Bonded Longitudinal Steel Reinforcement on Flexural Responses of Reinforced Concrete Beams**" is my work. The work hasn't been presented elsewhere for evaluation. Where materials have been taken from other sources, it has been properly referred.

Signature of Student

Fekadu Debela

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who have work jointly with me.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF ACRONYMS

ACI:	American concrete institute
ASTM:	American standard test method
(B_{11}-B_{14}):	Beams with 16mm bar Diameter having varied Unbounded Length
(B_{21}- B_{24}):	Beams with 8mm bar Diameter having varied Unbounded Length
C-25_1:	Compressive Strength Test of 25Mpa Concrete Sample Number One
C-25_2:	Compressive Strength Test of 25Mpa Concrete Sample Number Two
C-25_3:	Compressive Strength Test of 25Mpa Concrete Sample Number Three
ds:	Displacement at Service Load
dsR:	Displacement at Service Load For Reference Beam
EBCS:	Ethiopian Building Code of Standard
F_{cr}:	First cracking load
FcrR:	First cracking load of the reference beam
Fu:	Ultimate load
FuR:	Ultimate load of reference
kg/m^3:	kilogram per meter cube
kg:	kilogram
kN:	Kilo newton
kN.m:	kilo newton meter
L_{eff}:	Total Effective length of the beam
L_{un}:	Unbonded length of reinforcement
mm:	millimeter
m:	meter
mm^2:	Square millimeter
Mpa:	Mega Pascal
SPT1:	Splitting Test of Sample One
SPT2:	Splitting Test of Sample Two
SPT3:	Splitting Test of Sample Three

ABSTRACT

This paper is devoted to the combined effect of unbounded length and steel bar diameter on the flexural responses of reinforced concrete beams. A test series of eight simply supported beams having varied un-bounded lengths that bond loss starts at the middle and extends toward the supports had been performed in this experiment. All beams have a total span of 1220mm and are loaded at the middle with one concentrated load.

One of the main assumptions in the deigning of the reinforced concrete member is the efficient concrete-steel bond mechanism. But, loss of bond due to factors, such as errors during repairing, construction errors, corrosion, rough and dry formwork, formation of voids, etc. that may reduce the carrying capacity of the reinforced concrete beams and flexural behavior in general. Therefore, bond-slip has much attention from academic areas and industries. An experimental program is conducted to examine the effect of variable un-bonded length of longitudinal reinforcement for bottom reinforcement with different bar diameters on the flexural capacity of reinforced concrete beams, with shear reinforcement, subjected to transverse point load. The bond loss was provided with plastic tubes covering the longitudinal reinforcement by allowing bonded length over supports which prevent the formation of bond-slip at support.

The effect of un-bonded length with different bar diameters on the response, cracking load, ultimate load capacity, ultimate mid deflection, and deflection at service load of beams is discussed in this paper. The cracking load and cracking pattern, ultimate load capacity, and ultimate deflection are highly affected by increasing bond loss length for 16mm bar diameter. But, the ultimate load-carrying capacity of beams with a bar diameter of 8mm for the same length of bond loss was 7.31% even for fully unbounded. This may be attributed to the good bond strength of small bar diameter (8mm) than that of large bar diameter (16mm), in addition to the bond around the end supports and the presence of the anchorage.

Keywords: Beams, tensile strength, bond strength, concrete, flexural strength, reinforced concrete, Deflection, and ultimate capacity

CHAPTER 1 INTRODUCTION

1.1 Background of the study

Reinforced concrete is the concrete in which steel is embedded in such a manner that the two materials act together in resisting forces. Reinforced concrete (RC) may also be called reinforced cement concrete (RCC) is made up of different materials in which concrete is relatively low in tensile strength and good in compression strength and the ductility is provided by the provision of reinforcement having higher tensile strength or ductility, which in turn provide a bond strength between concrete and the reinforcement. Steel provides added strength by taking up the tension, which the concrete allows to resist/withstand compression. Reinforcement for reinforced concrete to develop the strength of a section in tension depends on the homogeneity of these materials to act together as one material in resisting the externally applied load. The reinforcing material, such as a reinforcing steel bar, has to experience uniform strain or deformation as the associated concrete to prevent the discontinuity or disintegration of the two materials under an externally applied load.

The modulus of elasticity, ductility, and yield or rupture strength of the reinforcing materials must also be considerably greater than the concrete to increase the capacity of the steel-reinforced concrete section. As a result, materials like brass, aluminum, rubber, or bamboo are not appropriate for developing the bond or adhesion required between the reinforcement materials and the neighboring concrete. Steel corrosion means loss of bond of reinforced bars which results in cracking, reduction of bond strength, reduction of steel cross-section, and loss of serviceability life of a concrete structure (O'Flaherty et al., 2008).

Also for the structures, which are under seismic loads, concrete-steel bond and flexural capacity have a greater role. The common causes of damage and collapse of RC structures susceptible to the seismic areas are:- Stirrups/hoops confinement, Ductility, Poor bond, Anchorage and lap-splices, Inadequate shear capacity and failure, Inadequate flexural capacity and failure, Inadequate shear capacity at the joints.

Steel and fiberglass reinforcing materials have the main components required: yield strength, ductility, and bond strength. Bond strength is a combined effect of more parameters, such as the mutual adhesion between the concrete and steel reinforcement affinity and the pressure of the hardened concrete against the steel reinforcement or wire due to the drying shrinkage of the concrete part. Adding upon this, friction interlock between the steel bar surface deformations or projections and the concrete resulted from the micro-movements of the tensioned steel bar results in increased resistance to slippage. The total effect of this is called a bond.

In general, bond strength is monitored by the following main causes:

- Diameter, shape, and spacing of reinforcement as they affect crack development
- Concrete cover.
- Contacting surface,
- The surface texture of reinforcing materials,
- chemical adhesion that develops at the steel-concrete interface,
- Gripping effect; resulting from the drying shrinkage of the surrounding concrete and the shear interlock between the bar movements and the concrete around the steel.
- Frictional resistance; to sliding and interlock as the reinforcing material is subjected to tensile stress
- Concrete's quality effect and strength; in tension and compression
- Mechanical interlock; of the ends of steel bars through development length, splicing, hooks, and crossbars.

However, the research carried on the above subject so far has been rather limited to the effect of un-bonded on shear failure and does not include the effect of bar diameter with length and location of bond loss on flexural behavior. Published research has not presented the advantages stemming from the elimination of concrete-reinforcing steel bonds with sufficient importance to attract the interest of the construction industry.

Attention is often focused on examining the load resistance of RC beams weakened by the removal of concrete around the longitudinal reinforcement while under structural repairs (**Cairns, 1995; Cairns and Zhao, 1993**). Though, for cases in which the concrete-steel bond strength develops, there has been considerable progress over the last three decades. In the authors' opinion, this advance has been mainly due to work concerned with the development of the compressive force path (CFP) method. However, even such solutions have recently been found to lead to a significant improvement of structural behavior when combined with the elimination of concrete - longitudinal reinforcement bond strength within the critical regions. The study on bond strength between corrodes steel rebar and concrete (Kotsovos et al., 2015). The study states that bond deterioration was influenced by the corrosion level, the loading history, and the degree of confining reinforcement (Valente, 2012).

Thus, it is foreseen that the un-bonded length of longitudinal reinforcement affects flexural behavior. This bond loss may be raised by:-

- Construction errors,
- Honeycombed (structure of linked cavities), void formed between steel and concrete due to improper vibration
- Concrete resulting from wrong compaction, and
- The use of dry and rough formwork could remove the concrete cover; affect the shape, location, length, width, and propagation of cracks as well as the capacity to carry a load of the section.
- Corrosion of reinforcing materials, etc.

1.2 Statement of the problem

Nowadays, research on reinforced concrete members increases from day to day all over aspects of the property of reinforced concrete. Flexural strength is one type of special property of reinforced concrete beams. The strength and effectiveness of the bond between concrete-reinforcing materials determine the integrity of the structure, in which, this property is affected by many factors. One of the main factors is the loss of concrete-reinforcing materials bond. The transferred Force via the bonding interface results in tensile strength and proves the overall strength of the concrete members and structures. When externally applied load resists the strength of the bond, a slip of bond will happen, and relative movement between the reinforcing materials and the concrete occurs.

Studied the bond between reinforcement and concrete, the influence of steel corrosion, the report has shown that the increase in diameter of bars results in a reduction of the bond strength which was highly significant for specimens without corrosion, followed by a specimen that has moderate corrosion. However, for a large degree of bar corrosion, the effect of bar diameter on bond strength was small. But the study didn't consider the effect of bond loss location on the bond strength (Mohammad Sonebi, Davidson, and Cleland 2011).

Some researches indicated that the ultimate capacity of beams is not reduced even more percent of bond length is lost. The authors concluded that a medium reduction in the load-carrying capacity was seen even in beams with significant bond loss (Mousa & Mousa, 2019).

This observation may be applied to the provision of small areas of a bond of the crossing stirrups that balance for the avoided bond and produces high bond forces. Again here, what if at all contacting area of crossing stirrups is not bonded?

Furthermore, corrosion of the reinforcing steel is a serious factor for the deterioration of bond between the concrete and steel interface which may cause a reduction in the load-carrying capacity of steel bars due to a decrease in the cross-sectional area of steel, (Almusallam et al., 1996).

Generally, Bond failure can result in a catastrophic failure of the structure, which causes a serious problem such as fatalities, injuries, and loss of the structure as a whole.

Not all these researches considered the combined effect of bar diameter and varied length bond loss along the flexural reinforcement starting from the middle toward supports and loading system of concentrated downward transverse load that may affect the concrete - reinforcing materials bond and which influence the flexural response of reinforced concrete beams. Bond failure at the steel-concrete interface does not enable the tensile force to be developed in the steel reinforcing bars, thus reduce the resistance of the structures

So, this research focuses on a basic understanding of this problem and discussed the combined effects of un-bonded longitudinal reinforcement and bar diameter on the reduction of ultimate load capacity and other flexural responses of reinforced concrete beams.

1.3 The objective of the Study

1.3.1 General objectives

To investigate the combined effect of bar diameter and variable un-bonded length and of tensile reinforcement on the flexural response of reinforced concrete beams

1.3.2 Specific objectives

- Determining of Physical properties of materials for concrete (aggregates, fine, and steel reinforcements)
- Determining of compressive and flexural strength of concrete
- Determining the effect of different bar diameter on the flexural response of un-bonded RC beam
- Determining the effect of un-bonded length on the flexural response of RC beam
- Assessing and Comparing the results of the beams

1.4 Significance of the Study

Corrosion of steel reinforcing bars is the main durability problem that leads to the loss of bond between concrete and reinforcing materials. A limited experimental instigation on the effect of unbonded longitudinal bars and bar diameter at the same time starting the bond loss from the middle toward the supports. Moreover, conducted experiments do not consider the important variables that affect the crack propagation and patterns, deflection, and flexural ultimate loads, such as the location of bond loss, bar diameter alongside varied length from the mid-span of the beam towards the support. As these variables have a greater effect on the bond strength and affect the flexural behavior of reinforced concrete beams, this needs to be investigated. The research plays a greater role in providing better knowledge on the impact of bond loss during the designing and construction of reinforcing concrete structures.

1.5 Scope of the research

- The material to be used is granite crushed aggregate, washed sand, stirrups, longitudinal steel reinforcement, and Ordinary Portland cement.
- Throughout the tests, the failure will be only in flexure.
- The effect of the variable un-bonded length of bottom reinforcement on the flexural strength of reinforced concrete beams will be studied.
- The work will be limited to the flexural behavior of the un-bonded length of bottom reinforcement reinforced concrete.

1.6 Research Questions

- What is the effect of unbounded length along the steel bar on the flexural response of the RC beam?
- Is the combined use of different bar diameters and varied un-bonded lengths related to bond strength?
- What effect does using bar diameter and un-bonded length have on the steel bar-concrete flexural strength?
- What is the relation with the reference beam for different bar diameters and varied un-bonded lengths?

1.7 Thesis Layout

The thesis consists of the following chapters

Chapter 1 gives a brief introduction to the assessment and causes of bond loss between concrete and steel reinforcing bars. The objectives of the proposed research work are identified in this chapter.

Chapter 2 reviews detailed literature on the empirical and theoretical review. It continues to discuss the flexural strength and research gaps briefly.

Chapter 3 explains the methodology and physical properties of materials in detail.

Chapter 4 discusses the details of experimental studies performed on the beams that were under a center-point loading system. And discussed results and discussions for all beams with varied un-bonded lengths and different bar diameters, including mode of failure and the ultimate flexural capacity of the affected beams relative to the controlled beams.

Chapter 5 This chapter discussed the conclusion and Recommendations for the experiment program.

CHAPTER 2 LITERATURE REVIEW

2.1 Introduction

2.1.1 Reinforced concrete

Reinforced concrete is composed of various materials. To behave as one material system, reinforcement and concrete must be well bonded together. Concrete is the one, which exerts the bond force onto the reinforcement. And reinforcement carries the transferred tension force. In most cases today, the reinforcements are deformed bars because they provide better mechanical bond strength. Concrete containing a Chain of reinforcing materials to make a composite material that is strong in tension as well as in compression. Smaller volumes of material are used to make beams, bridge spans, etc. compared with unreinforced concrete, brick, or masonry. Reinforcement is designed to carry the tensile forces that are transferred by the bond between the interfaces of the two materials. If this bond is not adequate, the reinforcing bar will just slip within the concrete and there will not be well compacted around the reinforcement during the construction.

2.2 Theoretical review

The advancement of reinforced concrete by French engineers in the middle of the 19 century was one of the major advances in the history of construction Most of today's concrete construction relies on the composite *interaction* of *concrete and steel*, which is aided by the near equivalence of their thermal expansion characteristics. In general, reinforced concrete has proved to be a highly successful material in terms of structural performance. However, there have been numerous examples of durability problems due to the corrosion of reinforcement in concrete structures, mostly due to poor quality concrete, poor design, and workmanship, inadequate cover to reinforcement, chlorides in the concrete, or combinations of these. Which has led to various forms of corrosion-induced damage such as cracking and spalling, resulting in reductions in structural capacity.

The major factors that influence the bond strength were defined by, **(Ozoden and Akpinar, 2006)** as:

- Diameter and shape (rip pattern) of the reinforcing bars.
- Development length, spacing, hooks, and crossbars.

- Adhesion between the steel reinforcement and concrete.
- Position of reinforcing bar during casting.
- Admixture and enhancing materials for concrete.
- Level of compression and tension strength of concrete.
- Concrete cover and bar spacing.

When steel reinforcement corrodes, corrosion products generate tensile stresses along with the concrete. Concrete is very strong in compression but weak in tension, the tensile strength being only about 10 percent of the compressive strength.

Therefore, flexural cracks are readily created and propagated because the formation of corrosion products along the bar surface may affect the failure mode and ultimate capacity of flexural members due to two causes:

- Due to a decrease in the rate of bar confinement caused by an opening of flexural cracks along with the reinforcement and,
- Because of significant changes at the steel-concrete interface caused by changes in the surface phenomena of the reinforcing steel (i.e. roughness to smooth which lead also reduction of bar cross-section.)

The changes in the surface conditions as a result of corrosion are characterized initially by variation in the roughness of the surface, then, by the development of less firm adherent interstitial layers of corrosion products between concrete and bar and, eventually, by local damage in terms of heavy pitting and degradation in the profile of the bar ribs.

2.2.1 Flexural Strength of Reinforced Concrete beams

In reinforced concrete beam design, both the flexural strength and ductility need to be taken into consideration. Strength refers to the maximum load that the structures will carry without failure or collapse. Stress, strain, displacement, and similar quantities are important as a tool for determining the structural capacity. The basic concept of flexural strengthening is to improve the strength and stiffness of concrete flexural members by adding steel rebar to the concrete tensile surface. Conventionally, steel rebar has been highly provided as a building material. For example, section enlargement using steel-reinforced concrete, steel plate bonding, and external steel post-tensioning technique have been widely applied. So, the bond strength between reinforcing materials and concrete is necessarily considered to insure the flexural capacity of reinforced concrete beams

2.2.2 Bar diameter

Stress or load in reinforced concrete is transferred through the steel-concrete bond. As result, the interfacial concrete-steel bond in splice is an important factor to be discussed. Since the term ''bond'' is the resistance against the slip between concrete and steel, and integrity between the two materials must be proved.

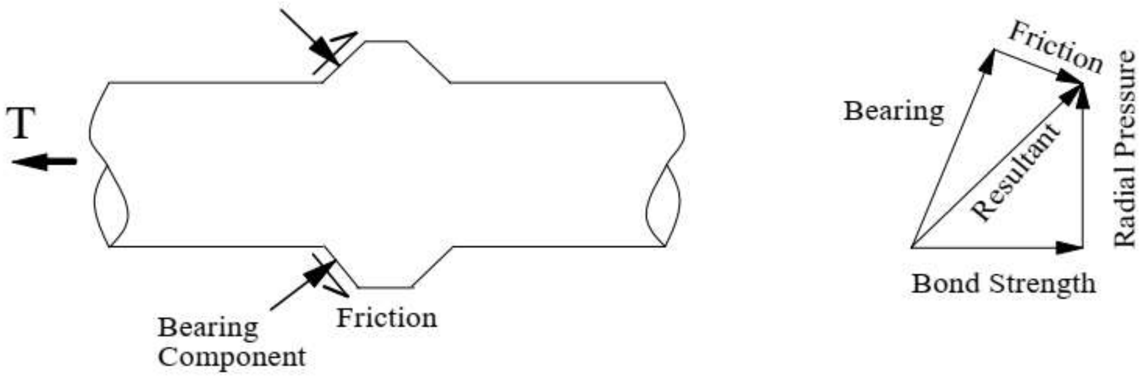


Figure 2.1. Free body diagram of steel section embedded in concrete

It is suggested that splice behavior is affected by splitting cracks developing along the bars and by the flexural cracks which mainly formed at the splice ends. These two types of cracks are governed by the bond between reinforcing steel and concrete. Which, flexural cracks, in particular, are related to the maximum slip of steel, which depends on the local micro-crushing of the porous concrete layer in front of the rip. While splitting cracks, are caused by rib-wedging action and are governed by the bond strength and stiffness. Ribbed bars develop higher bursting forces than the smoothed bars since they develop higher bond strength and not because they generate higher splitting forces. The bond force at failure is a function of the bar area as well as being a function of concrete cover, bar spacing, and embedment length **(Darwin et al. 1996).** Bar diameter plays a great role in the effect of concrete cover on the bond strength.

2.2.3 Bond -Slip in Reinforced Concrete

With the advancement of large-scale and highly complex civil infrastructures, research on the reliability and safety of these infrastructures has attracted much attention. Concrete structures with different reinforcement, such as steel bars, composite material tendons, and recently steel plates, are commonly used in civil infrastructures. The phenomena and strength of the bond between the reinforcement and the concrete determine the integrity and the service life of the structure.

2.2.4 Bond and Bond Stress Development

In reinforced concrete structures, bond stress is the name associated with the shear stress at the steel bar-concrete interface which modifies the steel stress along the length of the bar, by transferring load between the bar and the surrounding concrete **(Moreillon, 2013).**
Bond stress is calculated as the nominal shear force per unit surface area of the reinforcing bar. The load that is applied externally is very rarely applied directly to the reinforcing steel, which receives its share of the load through the surrounding concrete. As a result, an efficient reinforced concrete must have a positive interaction between the bar and concrete to achieve force transfer between the two materials

When the steel bars are embedded in concrete, the concrete after setting maintains to the surface of the bars and thus resists any force that tends to pull or push this bar. For the most efficient reinforcing action, steel and concrete must deform together, i.e. that there be a sufficiently strong ‘’bond’’ between the two materials to ensure that no relative movement of the steel and the surrounding concrete formation. The degree of strength of this adhesion force is called bond stress. It is the longitudinal shear stress acting on the surface between the steel and concrete. The term bond is used to describe how to slip between the steel and concrete is prevented. The bond is provided by anchoring the bars properly and extending the steel beyond the point of maximum shear.
Stress in a bond is primarily the result of the interlock of shear between the reinforcing materials and the enveloping concrete caused by the various factors previously formed. It can be specified as a local shearing stress per unit area of the bar surface. This direct stress is transferred from the concrete to the bar interface to change the tensile stress in the reinforcing bar along its length. Having a bond-stress slip relationship made it possible to make the total bond force could be calculated as the sum of local bond forces. The maximum force in bond occurs at a peak load and when the force in the bond decreases, the load-carrying capacity is reduced. Local areas with bonds play a role efficiently to create anchorage for the flexural bars. It was also found that the loss of bond was compensated by the increased provision of stirrups, and that loss of bond does not lead to more brittle types of failure. The bond action between the steel and the concrete can be idealized as the shear force at the circumferential surface of reinforcement, (Wang, 1961)**.**

2.2.5 Bond mechanisms and Bond Failure

The mechanism of the bond is the interaction between steel reinforcement and the surrounding concrete which allows forces to be transferred from the reinforcement to the surrounding concrete. The bond of reinforcing steel bars mainly depends on mechanical interlocking, adhesion, and friction **(Xing et al. 2015).**

For plain bars the effect of adhesion is small and friction forces don't form until adhesion has failed and relative movement between reinforcing bars and concrete occurs. The mechanical interlock of the rips of the bars embedded in the concrete governs the bond stress-deformation behavior for deformed bars.

2.2.5.1 Adhesion

Adhesion is a chemical bond that formed at the concrete-steel-reinforcing interface. Adhesion at concrete-steel interface depends on the concrete composition, steel bars surface roughness, and the reinforced concrete specimen age. The resisting mechanism for the small loads is the chemical adhesion. But, as the loads increase the chemical adhesion along the bar surface is lost quickly.

It is recommended that the bond strength due to adhesion is between 0.48 to 1.03MPa (ACI Committee 408, 2003)**.**

2.2.5.2 Friction

Friction has a similar manner to the mechanical interlock. The frictional bond is highly based on the surface characteristics of the reinforcing steel bar, aggregate size, and shape (Momayez et al, 2005). Following the destruction of chemical adhesion, friction-slip occurs before the full bearing capacity at steel bar rips is formed, (Wassouf,n.d.2015)**.**

And ACI Committee 408 suggested that friction can give up to 35% of the ultimate strength controlled by splitting of the concrete cover.

2.2.5.3 Mechanical interlock

The surface profile of a steel bar dictates the amount of mechanical bond that can be generated between steel bar and concrete. The mechanical interlocking of the deformed bar is enhanced by the geometry of the rips along the steel bar. Bearing against the lugs is considered to be the most significant transfer mechanism at higher load levels, for deformed steel bars. This is because the force transfer mechanism is due to the mechanical interlocking between the rips and the concrete. Shear starts to occur in concrete between rips as the interlocking forces induce large bearing stresses around the rips, and slip forms when the ultimate bond strength is reached. The slip of the deformed bar may forms in two manners, either through pushing the concrete away from the bar rips (i.e. Wedging action) or through crushing of concrete by the rips (Máca et al., 2016; Martin & Martin, 2006; Zhang et al., 2016).

2.2.6 Crack in reinforced concrete

Cracking in concrete is nearly impossible to avoid. Cracking in concrete is unavoidable due to its low tensile strength and its capability of being extended is low (extensibility). However, the size and location of cracks can be limited and controlled by appropriate reinforcement, control joint, curing methodology, and concrete mix design. Reinforced concrete structures having low stresses in steel under service loads undergo very limitedly cracking, except for the cracks that occur due to shrinkage of concrete and temperature changes. However, in cases where service loads cause high steel stresses, particularly when using high-strength steel, some visible cracking may be expected under service loads. If these cracks are, too wide they destroy the aesthetics of the structure. They also may result in steel reinforcement being exposed to the environment causing corrosion of steel.

To minimize these adverse effects, the reinforced concrete structure design must ensure that the crack widths under normal service conditions are maintained within acceptable limits. In addition to the aesthetic concerns and possible corrosion of reinforcement, the cracking of a reinforced concrete member will cause a significant reduction in the bending stiffness. Therefore, in assessing the deflection of reinforced concrete elements, so incorporating the effect of cracking in the calculation is necessary.

Ultimate failure results in the collapse that can be by crushing of concrete which formed when compressive stresses are beyond its strength, by yielding or failure of the rebar when bending or shear stresses exceed the reinforcement strength, or by the concrete–reinforcement bond.

2.2.6.1 Flexural Crack

External load results in direct and bending stresses, causing flexural, bond, and diagonal cracks. Immediately after the tensile stress in the concrete exceeds its tensile strength, internal micro-cracks occur. These cracks generate macro cracks propagating to the external fiber zones of the element. The major factors affecting the development and characteristics of the cracks are the percentage of reinforcement, bond characteristics, size of the bar, concrete cover, and the concrete area in tension, (Warren, 1969).

Flexural crack is a big issue in reinforced concrete structures which have been under analysis as a possible result of steel corrosion. Many engineering professions have deliberated to use steel with high strength at increased stress because larger flexural cracks result in larger corrosion damage.

As, flexural crack width is a corrosion factor, the width where the concrete and the bar contacts must be the most significant width

2.2.6.2 Allowable crack widths in reinforced concrete

The maximum width of crack that may be considered not to impair (diminish) the appearance of a structure depends on various factors including the position, length, and surface texture of the crack in addition to the illumination in the surrounding area.

Crack widths in the range of 0.25 mm to 0.38 mm may be acceptable for aesthetic reasons (Rasidi et al., 2020).

The width of the crack that will not damage the steel reinforcement corrosion depends on the surrounding environment of the structure. Table 2.1 shows the maximum allowable crack widths recommended for the protection of reinforcement against corrosion (Rasidi et al., 2013).

These values are taken as the basis for the development of rules prescribed in ACI 318 (1995) for the distribution of tension steel to limit the crack width.

Table 2.1 Maximum permissible crack widths (ACI Committee 224)

Exposure condition	Maximum allowable crack width (mm)
Dry air or protective membrane	0.4
Humid, moist air or soil	0.3
De-icing chemicals	0.2
Seawater and seawater spray; wetting and drying	0.15
Water retaining structures	0.1

2.3 Empirical review

The present work is the first stages of a comprehensive program aimed at establishing whether the use of non-bonded flexural reinforcement can be used for the flexural strength RC beams with transverse reinforcement and simplify its arrangement without violating the recommended code for structural performance. In a number of the specimens test, transverse loading will be along with an axial force that was applied first and then increased to a predefined value where it was maintained constant during subsequent application of the transverse load.

Investigated one hundred and eleven (111) under reinforced concrete beams which under different degrees of reinforcement corrosion to determine their residual flexural capacity. The report has shown that there were marked reductions in flexural strength due to reinforcement corrosion, which was caused primarily by the breakdown of the bond at the steel/concrete interface (Mangat and Elgarf, 1999).

The response of reinforced concrete beams with bond loss flexural reinforcement. Eliminating the concrete-longitudinal reinforcement bond within specific lengths of the shear span leads to an increase in shear performance of which becomes larger as the non-bonded length extends from the point loads towards the support. Eliminating the concrete-longitudinal reinforcement bond within specific lengths of the beam's span may still lead to premature failure that is flexural but occurs before yielding the flexural reinforcement (Kotsovos et al., 2015).

Conducted a wide investigation consisting of tests on eighty-two simply supported partly pre-stressed rectangular beams. Out of this, 41 contained tendons that have un-bonded. The parameters were reinforcement amount, the type of loading, and compressive strength concrete. They reported that beams having no appending (supplemental) reinforcement, failed by developing only one major crack, while those with appending (supplemental) reinforcement developed many cracks before failure. The stress in the unbounded steel bars left in the elastic range till failure. They supposed too conservative equation for prediction of stress in the tendon at an ultimate **(Warwick et al. 2015)**.

Investigated the model and experimental study for residual capacity strength of reinforced concrete beams with non-bonded reinforcement. They observed that, regardless of reinforcement ratio, span-to-depth ratio, and type of loading, all beams had to have their original flexural strength when the loss bond length is less than 50% of the span length (Jnaid, 2016; Jnaid & Aboutaha, 2018).

Flexural Behavior of Partially Bonded CFRP Strengthened Concrete T-Beams. He observed that, among partially bonded beams, the ultimate response (load and deflection) was decreased as the unbonded length was increased. The bond strength at the FRP-concrete interface may decrease as the unbonded length increases because the anchorage (bonded) length at both ends is decreased (Choi, 2008).

High strength concrete provided about 80% higher bond strength with 10 mm diameter of steel reinforcement than that of normal strength concrete, with 25 mm diameter of steel reinforcement, about 30% difference that indicates the influence of reinforcement size and concrete quality on the bond strength (Turk, 2003).

studied the bond capacity and its influence on the flexural strength of RC beam by considering the tensile reinforcement bond length with two-point loads and concluded that increasing the length of bar causes increases flexural strength and the presence of longitudinal rebar resulted in the ultimate momentum being more than crack momentum of the section in parts which have been broken at the point of longitudinal bar cut (Rashidi & Takhtfiroozeh, 2016).

A series of tests were conducted to relate corrosion of reinforcement to bond deterioration by testing beams that were designed to fail in bending. They reported that bond strength increased when corrosion up to a certain level of corrosion, But was very high progressively decreased as corrosion was very high. They also found that low levels of corrosion do not affect the ultimate load in flexure, but at a high level of corrosion, the ultimate load is reduced due to loss in bar diameter (Almusallam et al., 1996).

studied the Effect of bond loss of tension reinforcement on the flexural behavior of reinforced concrete beams and found that the cracking loads, number of cracks in the flexural zone, and the crack width are affected significantly by increasing the area of bond loss. While the reduction in the ultimate load capacity was low even for 73% loss of bond which was because of the presence of bond around stirrups (Magda J Mousa, 2019).

Conceptual Frame Work:

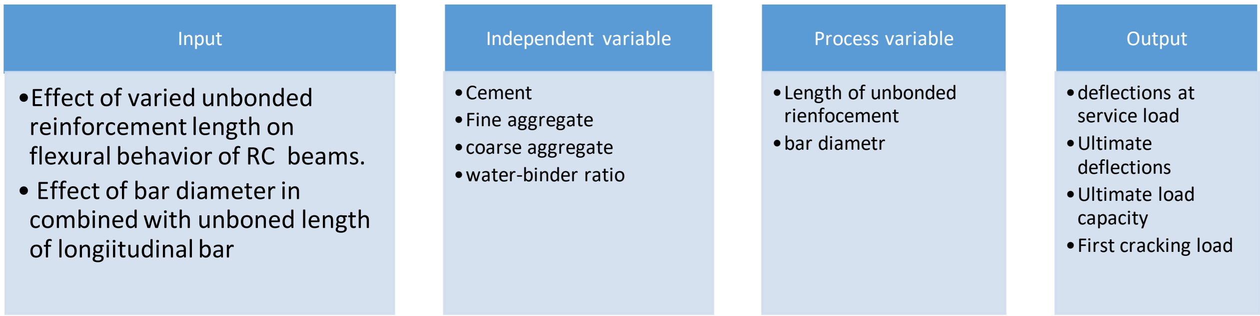


Figure 2.2 Conceptual framework

CHAPTER 3 METHODOLOGY

3.1 Introduction

This portion of the research discusses the methodologies (the way this study has been conducted) of the research. Some studies have been done on the effect of un-bonded reinforcing materials on the flexural response of reinforced concrete beams. Each research had its methodologies. The process and method of conducting this research will be through different kinds of literature and ACI mixing with a mixer. Each process also, literature review and from material characterization to strength testing is conducted accordingly.

The method for experimental investigation is performed by eliminating the reinforcement-concrete bond locally for different lengths from the middle of the beam toward supports. The reinforcement bond was eliminated with plastic pipes covering parts of the steel bars. Studying the effect of concrete-reinforcement un-bonded length for various distances from the middle is performed for different samples depending on the distance from the middle for bond loss and in combination with different bar diameters. Avoiding the bond is introduced artificially with plastic tubes surrounding the longitudinal tension reinforcement including the positions of stirrups crossing the longitudinal reinforcement as well and the loading system is transverse vertical downward load is applied with a point load.

In the preparation of samples and conducting tests the guideline in IS, ISO, ASTM standards were followed. In the section of the paper, the general sample preparations and the experimental setup to finalize the test are described. In this experiment program, eight numbers of reinforced concrete beams in two groups were cast.

In the first group of experiments, four reinforced concrete beams were cast. The main variables were unbounded length and diameter which were used together, i.e. bar diameter of 8mm for longitudinal reinforcement and 6mm bar diameter for shear reinforcements. From these samples of beams, one specimen is a reference or controlled beam. In the second group of experiments, four reinforced concrete beams were cast again. The main variables were unbounded length and diameter which were used together as in the case of the first sets but the bar diameter was changed, i.e. bar diameter of 16mm for longitudinal reinforcement. Again here also, one specimen is a reference or controlled beam with this diameter of the bar.

3.2 Beam Design for Flexure and Shear Based on ES EN 1992-1-1: 2015

This manual calculation was done before mechanical properties of reinforcing steel and compressive strength properties of concrete were conducted. It was intended to know the longitudinal reinforcement requirement for the beam to fail in Flexure manner.

Diameter of shear reinforcement is 6mm. The maximum longitudinal spacing between shear assemblies should not exceed Sl, max where:

$$Sl.max = 0.75 * d(1 + cot\alpha)$$

Where α is the inclination of shear reinforcement to the longitudinal axis of the beam. Overall depth of the beam is 200mm, using Φ16 bars as longitudinal reinforcement, and the concrete cover is 25mm. Thus effective depth, d, is 161mm.

$Sl.max = 0.75 * 161mm(1 + cot90$ $\quad$ $Sl.max = 120.75mm$

Width of the beam is 200mm.

Intended cubic strength of the concrete is 25MPa.Partial safety factors are ignored.

Shear capacity of concrete

The design value of shear resistance $V_{Rd,}$ is given by

$V_{Rd,c} = [C_{Rdc}k\ (100\rho_1 f_{ck})^{(1/3)} + k_1\sigma_{cp}]\ b_w d$ ....(3.1)

With the minimum of

$V_{Rd,c} = (V_{min} + k_1\sigma_{cp})b_w d$ ....(3.2)

where

$V_{min} = 0.035k^{\frac{3}{2}} * \sqrt{fck}$

$$CRdc = \frac{0.18}{\gamma c} = 0.18 \text{ (Partial safety factored } is\ ignored$$

$$k = 1 + \frac{\sqrt{200}}{d} \leq 2\ , 1 + \frac{\sqrt{200}}{161} = 2.114{\sim}2$$

$\rho 1 = \frac{Ast}{bw*d} \leq 0.02$ [However, the area of longitudinal reinforcement is to be determined,

Thus use $\rho 1 = 0.02$]

$\sigma cp1 = \frac{NEd}{Ac} < 0.2fcd(MPa),$

N_{Ed} is axial force in the cross section due to loading or prestressing. There is no axial force for this case.

$K_1 = 0.15$ [Recommended value from the national annex.]

$\sigma cp1 = 0$

Substituting this parameters

$V_{min} = 0.035 * 2^{\frac{3}{2}} * \sqrt{25}$, $V_{min} = 0.495$

$V_{Rd\ min} = V_{min} *bw*d = 0.495*200*161mm = 15.939$ kN

$V_{Rd,c} = [0.18*2(1000.02*25N/mm^2)^{(1/3)} + 0.15*0]\ 200mm*161mm$

$V_{Rd,c} = 42.705$ kN [is the shear resistance of member without shear reinforcement]

Member requiring shear reinforcement

For reinforced concrete members with vertical shear reinforcement, the shear resistance, $V_{Rd,s}$, should be taken to be the lesser, either

$V_{Rd,s} = \frac{Asw}{s} Zfydcot\theta\ or$

$V_{Rd,max} = \frac{\sigma cw\ bw\ z\ v1\ fcd}{cot\theta + tan\theta}$

Maximum value of $cot\theta$ must be used since the angle between longitudinal axis of thebeam and the crack cannot accurately be predicted.

Since U stirrups are used the shear area is twice the area of stirrups.

$$Asw = \frac{\pi * 6^2}{4} = 28.27\ mm^2$$

$V_{Rd,s} = 2(\frac{28.27mm^2}{S120.75mm})0.9 * 161mm * \frac{365N}{mm^2} * 2.5$, $V_{Rd,s} =$ $\mathbf{61.911\ kN}$ is the design value of the shear force which can be sustained by the yieldingshear reinforcemen.

$V_{Rd,max} = \frac{1* 200mm*0.9*161mm*0.6*25N/mm^2}{2.2+\frac{1}{2.5}}$ $V_{Rd,max} = \mathbf{249.83\ kN}$ is the design value of the maximum shear force which can be sustained by theme ber, limited by crushing of the compression strut.

$V_{Rd,} = V_{Rd,s} + V_{Rd,C} =$ **103.62kN**

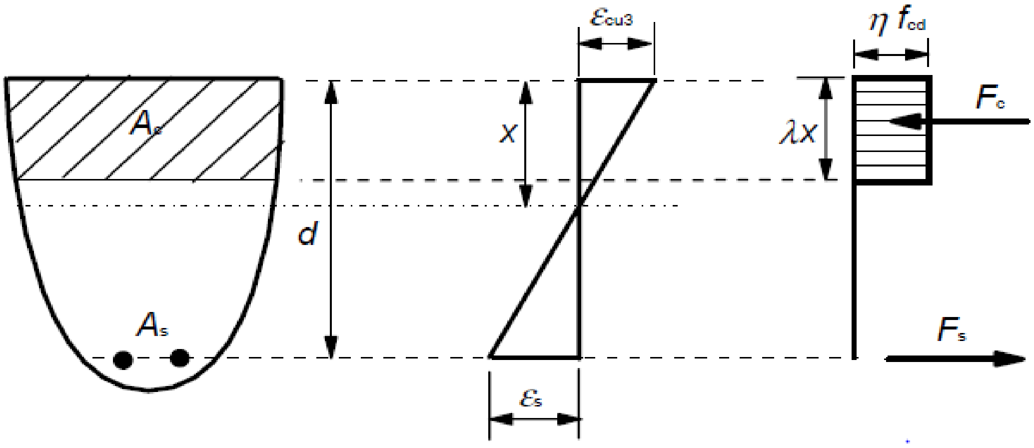
Since tensile reinforcement is provided for the beams to be tested, the failure load is 103.62kN.

Point load at the center to be applied to create this maximum shear is 207.23kN.

The beam is required to fail in flexure during testing and shear capacity of the section must exceed flexural capacity. The beam shall be designed, in flexure, to resist the moment induced due to the point load reduced by 50%, i.e. 103.62kN.

Maximum moment at the mid span for simply supported beam loaded at the mid span

$$Max = \frac{Pl}{4}, Max = \frac{103.62kN * 1ml}{4} = 25.904kN.m$$



Strain in the concrete is 0.0035.

Using a reinforcement with yield strength of 538MPa and knowing that modulus of elasticity of reinforcing steel is 200GPa as ES EN 1992 1-1, strain in the reinforcing steel is:

$$\varepsilon y = 538 \frac{Mpa}{200000Mpa} = 0.0027$$

Determining the neutral axis depth from the strain diagram using similarity of triangle:

x = 94.89mm.

Force in compression zone, F_c = 0.8xbf_{cd}

ES EN 1992 1-1 recommends λ = 0.8 and η =1.

$$Fc = 0.8 * 90.89mm * 200mm * \frac{25N}{mm^2} \quad Fc = 363.56kN$$

Now, determining the force that must be resisted by the reinforcing steel using equilibrium equation:

$M_{max} = T_s(d - 0.4x)$

$$Ts = \frac{\text{Mmax}}{\text{d} - 0.4\text{x}}, Ts = \frac{25.904 * 10^6 \text{N.mm}}{(161\text{mm} - 0.4 * 90.89\text{mm})} = 207.82\ kN$$

$$Ast = \frac{\text{Ts}}{\text{fyd}}, Ast = \frac{\text{T}207.82 * 1000\ \text{N}}{538\ \text{N/mm}^2} = 386.4mm^2 \quad as = \frac{\pi * 16^2}{4}$$

$$= 200.97\ mm^2$$

$$Using\ bar\ ø16mm,\ n = \frac{386.4\ \text{mm}^2}{200.96\text{mm}^2} = 1.92 \approx \#2$$

Use 2Φ16 bars as longitudinal reinforcement.

In a similar way of design for beams with bar diameter of 8mm,

Use 5Φ8 bars as longitudinal reinforcement.

- **Checking for steel to yield first**

$\boldsymbol{\varepsilon sy} < \varepsilon y\ (\varepsilon y = 0.0027)$

$\frac{0.0035}{x} = \frac{\boldsymbol{\varepsilon sy}}{\boldsymbol{d-x}}$ **,** from similarity of triangle

$$\boldsymbol{\varepsilon sy = \frac{(d-x)}{x} * 0.0035} = \frac{\mathbf{(161-94.89)}}{\mathbf{94.89}} * \mathbf{0.0035}$$

$= \mathbf{0.0024} < \varepsilon y(0.0027) \ldots Ok!\ steel\ yeild\ first\quad Or$

$\rho_{cal} < \rho_b$, i.e $\rho b = \frac{\mathbf{(0.8*\varepsilon cu)}}{\mathbf{(\varepsilon cu+\varepsilon yd}} * \frac{\boldsymbol{fcd}}{\boldsymbol{fyd}} = \frac{\mathbf{(0.8*0.0035)}}{\mathbf{0.0035+0.0027}} * \frac{\mathbf{25}}{\mathbf{538}} = \mathbf{0.021}$

$\rho_{cal} = \frac{\boldsymbol{Ast}}{\boldsymbol{b*d}}$**,** $\frac{\mathbf{386.4}\boldsymbol{mm^2}}{\mathbf{200*161}\boldsymbol{mm^2}}$**,** $= \mathbf{0.012} < \rho_b(0.021)$….Ok!

The above calculation is intended to know the number of longitudinal reinforcements required in order to make sure that all the specimens fail in flexure manner. Maximum yield stress values are used for shear reinforcement and longitudinal reinforcement. Also f_{cd} and f_{ck} values are taken as 25MPa.

All eight rectangular reinforced beams have the same dimensions with a width of 200mm, a depth of 200mm, and a total length of 1220mm.

Table 3.1 Detail of beam specimens

ID No:	Reinforcement used (Ømm)	Bond loss length (Lun),m	
B11	16	0.00	Group II
B12	16	0.33	
B13	16	0.67	
B14	16	1.00	
B21	8	0.00	Group I
B22	8	0.33	
B23	8	0.67	
B24	8	1.00	

3.3 Testing procedure

The experimental investigation is conducted with the following phases:

- Material mobilization
- Specimen preparation
- Specimen fabrication
- Instrumentation
- Specimen investigation

3.3.1 Material mobilization

All required materials for the preparation of samples were collected. This includes sand and aggregate from a store of distributors, cement and reinforcement bar from a store of distributors, and Wood sheet for formwork and placed properly.

a) Fine aggregate b) Coarse aggregate

Figure 3.1 fine and coarse aggregate preparation

3.3.2 Specimen preparation

3.3.2.1 Concrete

Concrete is the main construction material made from cement, fine aggregate (sand), coarse aggregate, and water that harden through time.
For this research targeted cylindrical compressive strength of 25 Mpa is required. The mix design is according to ACI mix design is used for normal steel-reinforced concrete and the construction laboratory manual by (Abebe Dinku 2002) was used. The slump of 40mm is recorded.

Materials Physical properties

The research is designed to look into a better understanding of the influence of the variable unbonded length of bottom reinforcement on the flexural strength of reinforced concrete beams. In the study, the materials used are Ordinary Portland cement, fine aggregates, coarse aggregates, Plastic pipes, wood formworks, and water.

Cement

The cement used should conform to the required specifications. From the types of cement available commercially in the market, Ordinary Portland Dangote cement is used for this study. The specific gravity of cement was 3.15 and the grade of 42.5 R.

Fine Aggregate

Locally available washed sand passing through a 4.75mm sieve is used. The material physical properties moisture content, silt content, specific gravity, fineness modulus, and gradation of the fine aggregate was studied according to the recommended standard. Sand is a product of natural artificial disintegration of rocks and minerals, which obtained from deposits of river, Lake Marine…etc., these deposits, however, do not provide pure sand. It may contain dust and clay which are finer than sand. The presence of such materials affects the material concrete which reduces the strength. Therefore it is necessary to check the silt content against the permissible limit to the Ethiopian standards silt content of 6% and the acceptable result should be less than this result.
This was carried out by measuring a jar having a capacity of greater than 100ml and pour sand into a jar and then $\frac{3}{4}$ of fill, a jar with water and shake vigorously, table 3.1 shows the silt content results.

Figure 3.2 Silt content tests

Table 3.2 Determination of silt content

Silt content (ml)	Pure Sand (ml)	%Silt Content = $\frac{silt}{pure\ sand} * \mathbf{100}$
10	260	= $\frac{\mathbf{10}}{\mathbf{260}} * \mathbf{100}$ **= 3.85%**

Fine aggregate grain size distribution or gradation is among those properties that this gave the considerations.

According to Ethiopian standards, coarse aggregate is between 9.5 and 0.15mm in size.

Table 3.3 Gradation of fine aggregate

Sieve size (mm)	Weight of sieve (gm)	Weight of sieve and soil retained (gm)	Weight of soil retained (gm)	Percentage retained (%)	Cumulative Coarser (%)	Cumulative Passing (%)	Standard Limit
9.5	1180.7	1180.7	0.00	0.00	0.00	100	100
4.75	1167.5	1167.9	0.4	0.08	0.08	99.92	95-100
2.36	982.8	1025.8	43	8.51	8.59	91.41	80-100
1.18	900.5	973.9	73.4	14.53	23.12	76.88	50-85
0.6	836.1	951.1	115	22.76	45.88	54.12	25-60
0.3	757.8	950.2	192.4	38.08	83.97	16.03	10 to 30
0.15	783.3	855.6	72.3	14.31	98.28	1.72	2 to 10
Pan	734	742.8	8.7	1.72	100.00		

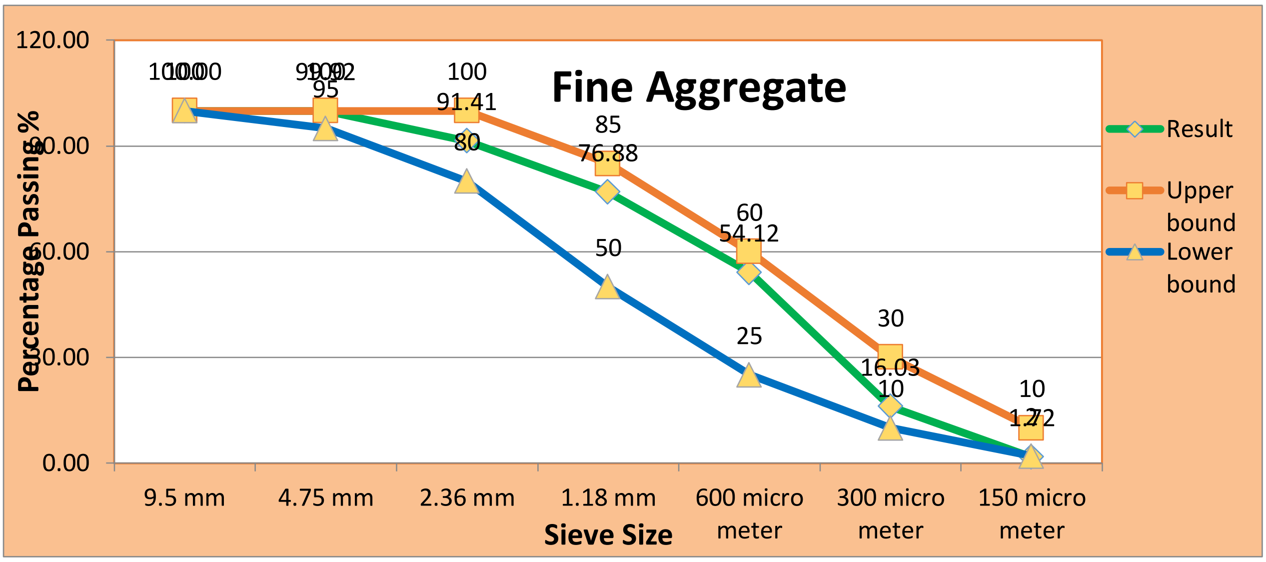


Figure 3.3 Fine aggregate Gradation curve line in the limit

Table 3.4 Physical properties of fine aggregate

S.No	Characteristics	Values obtained
1	Type	River sand
2	Specific gravity	**2.57**
3	Fineness Modulus	**2.60**
4	Bulk density	**1380 kg/m^3**
5	Water absorption	**4.32 %**

Table 3.5 the recommended range fine aggregate

Type of Sand	Fineness modulus
Fine Sand	2.2-2.6
Medium Sand	2.6-2.9
Coarse Sand	2.9-3.2

So, it is medium sand

Coarse Aggregate

Coarse aggregates used for the production of concrete will be crushed aggregate with strong, impermeable, durable & capable of producing a sufficient workable mix to achieve proper strength. The maximum size of the coarse aggregate used is 19mm. Physical properties such as specific gravity, moisture content, water absorption, and gradation are studied.

Similar to the fine aggregate the gradation for coarse aggregate is done with the sieve size between 19mm and 4.75mm.

Table 3.6 Gradation of coarse aggregate

Sieve size (mm)	Weight of sieve (gm)	Weight of sieve and retained (gm)	Weight of soil retained (gm)	Percentage retained (%)	Cumulative Coarser (%)	Cumulative Passing (%)	Standard Limit
19	1204	1204	0	0.00	0.00	100.00	100
12.5	1176	1410	234	5.08	5.08	94.92	90-100
9.5	1157.4	3512.9	2355.5	51.10	56.18	43.82	40-85
4.75	1264.2	3284.2	2020	43.82	100.00	0.00	0-10

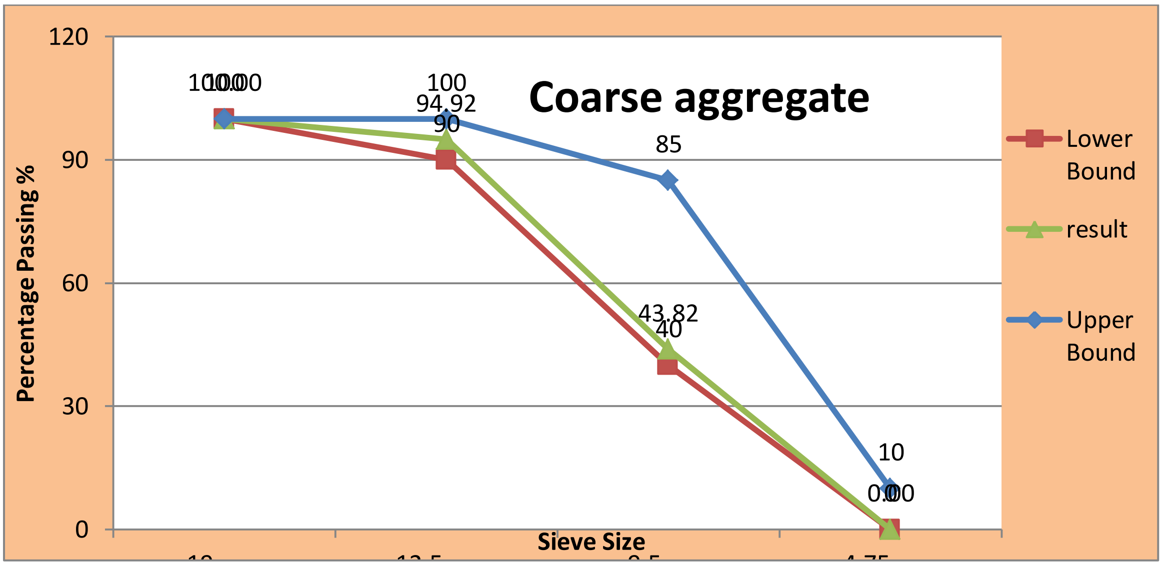


Figure 3.4 Coarse aggregate Gradation curve line in the limit

Table 3.7 Physical properties of coarse aggregate

S.No	Characteristics	Value obtained
1	Specific gravity	2.93
2	Fineness modulus	4.3
3	Bulk density	1578 kg/m^3
4	Water absorption	0.92 %

Water

The quality of water for the mixing of concrete affects the resulting hardened concrete. Impurities in water may interface with the setting of cement adversely affect the durability and strength of concrete. Fresh and clean water that is free from organic matter, silt, oil, and acid material as per standard used for casting and curing the specimen.

Table 3.8 Quantity of materials

Quantity of materials for C-25				
Material	**Water**	**Cement**	**Fine aggregate**	**Coarse aggregate**
Quantity (kg/m^3)	191.68	341.67	920.89	1022.67
Mix ratio	0.56	1.00	2.70	2.99

To have a consistency of concrete strength in each specimen, 350kg concrete mixer has been used as shown in figure 3.5

The concrete mix was kept identical for all flexural test samples and the concrete was allowed to sit for 28 days to gain the 28 days concrete compressive strength and an average concrete cube compressive strength of 32.62Mpa is recorded.

Figure 3.5 concrete mixing and cylinder, cube, and flexure taken

During the mixing process, mixing was divided into three times, one for filling of two beams, the second, and the third was for three beams, and for all beams, the design mix is the same. From this, three cubes (150mmx150mmx150mm), three cylinders with (150mm diameter and 300mm height), and three flexures (500mm length, 100mm width, and 100mm depth) were taken for testing of compressive strength, tensile splitting, and flexural strength of concrete respectively. Figure 3.7 shows the tensile splitting, compressive, and flexural tests.

Table 3.9 compressive strength Test

Naming of specimen	Cube designation	Mass (kg)	Load (kN)	Compressive strength(Mpa)	Load Mean Strength (kN)	Cube Mean strength (Mpa)	Cylinder Mean strength (Mpa)
NSC-C25	**C-25$_1$**	7932	661.4	30.39	720.88	32.62	26.10
	C-25$_2$	8210.2	772.2	34.52			
	C-25$_3$	7818.5	729.05	32.96			

a) b) c)

Figure 3.6 compressive strength tests: a) weighing b) Testing and c) crushed concrete after the test

Splitting strength of concrete is given by:

$ft = \frac{2P}{\pi DL}$, Where, L= Length of Cylinder, D =diameter, P=compressive load at failure

Table 3.10 Splitting strength Test

Naming of specimen	Designation	Mass (kg)	Load (kN)	tensile strength(Mpa)	Load Mean Strength (kN)	Tensile Mean strength(Mpa)
NSC-C25	**SPT$_1$**	12443	170.3	2.41	156.95	2.22
	SPT$_2$	12421	143.6	2.03		
	SPT$_3$	12422	156.95	2.22		

Figure 3.7 Tensile splitting tests:

Table 3.11 flexural strength Test

0Naming of specimen	Designation	Mass (kg)	Load (kN)	Flexural strength(Mpa)	Load Mean (kN)	Flexural mean strength(Mpa)
NSC _C25	**FS$_1$**	11642	7.8	3.48	7.23	3.40
	FS2	11800	6.8	3.08		
	FS$_3$	11550	7.1	3.64		

Figure 3.8 flexural strength tests

3.3.2.2 Reinforcement

In this experimental program, the deformed bar of 8 mm and 16mm diameter were used as longitudinal reinforcement. With the bar diameter of 8mm, there were four beams including one reference beam and with 16mm there were also four beams including one reference beam. A universal tensile test for these steel bars was conducted by using the universal tensile strength test machine UTM. Table 3.8 shows the mechanical properties of reinforcements.

Table 3.12 Mechanical properties of reinforcements

Diameter (mm)		Area (mm^2)	Yield load (kN)	Yield stress (Mpa)	Failure load (kN)	Failure stress (Mpa	Elongation (%)	Average yield stress (Mpa	Average failure stress (Mpa
6	27.14	9.9	364.78	11.2	412.68	23.9	366.03	425.44	
	26.96	9.8	363.50	11.4	422.85	23.8			
	26.77	9.9	369.82	11.8	440.79	23.8			
8	58.33	25.8	442.31	33.2	569.18	23.5	447.57	568.45	
	58.60	26.4	450.51	32.8	559.73	23.6			
	58.46	26.3	449.88	33.7	576.46	23.6			
16	218.93	118.2	539.90	138.8	633.99	22.1	537.85	632.94	
	219.45	116.7	531.78	138.7	632.03	22.2			
	219.98	119.2	541.87	139.2	632.78	22.9			

3.2.4 Specimen Fabrication

For this study eight number of reinforced concrete beams with the width of 200mm, depth 200mm, and total length of 1220mm. All beams were tested in a laboratory using a downward point loading system as simply supported beams. So, for these specified dimensions of beams and loading system, the specimen fabrication was as follow:

3.2.4.1 Formwork

In this experimental program, the whole formwork is plywood, as shown in figure 3.10. The reason why plywood is taken is that during concrete filling it wouldn't burst easily and it is smooth as well which prevents the concrete cover from damage. One full plywood was used for the bottom cover and the other plywood was divided according to the dimension of the beams for side covering.

After it was erected well, it kept on a level position to maintain the straightness of the beam member.

Figure 3.9 Plywood formwork preparations

3.2.4.2 Reinforcement

The reinforcement preparation and detail were according to the euro code for flexure recommendation. Since the diameter of the bar was one parameter of the study, reinforcement for the bottom of the beam was varying. A total number of eight beam specimens were prepared and out of this four beams were cast with 8mm diameter of the bar for main or longitudinal bars with a varied length of bond loss artificially, along those longitudinal bars while the rest four beams were cast with a bar diameter of 16mm having the same bond loss length as that of the first beam group. The reason why the specimen varied in terms of bar diameter and bond loss is to assess the effect of bond loss with different bar diameters on flexural behaviors (deflection, flexural load-carrying capacity, first cracking load, and failure mode) of reinforced concrete beams and compare the results of all cases in terms of all behaviors. Generally, it is to understand and to give recommendations to which extent of bond loss is more affect the performance of the beam.

Figure 3.10 Specimen reinforcement preparations

3.2.4.3 Avoiding the roughness of the reinforcement

As explained earlier in the introduction part, to make the longitudinal reinforcement is unbounded with the neighboring concrete. The roughness of the reinforcement was covered with plastic tubes of greater diameter than the diameter of the bar. Figure 3.12 shows how the steel bars were unbounded.

Figure 3.11 Preparation of unbounded reinforcement specimen

The concrete was cast outside the laboratory and a vibrator was used to consolidate the concrete. The surface of the concrete susceptible to air was covered and moist cured wet cotton and other materials to prevent the concrete from high evaporation and to absorb water as shown in figure 3.13.

Figure 3.12 Beams casted outside laboratory

3.4 Test setup

The span length of all beams was similar and the setup for all beams was the same appearance. All the beams were instrumented to record the applied load and displacement. The type of support was roller support and a point load system was applied at the middle-span using a Hydraulic jack with a load cell measuring the load applied by the hydraulic jack. These all instrumentations are connected to the data logger and the results were easily obtained.

Figure 3.14 Laboratory test setup

Figure 3.15 Data Logger

CHAPTER 4 RESULTS AND DISCUSSIONS

4.1 Experimental Results

4.1.1 Experimental results for specimens with a bar diameter of 8mm

Figure 4.1 Flexural failure of specimen B21

Figure 4.2 Flexural failure of specimen B22

Figure 4.3 Flexural failure of specimen B23

Figure 4.4 Flexural failure of specimen B24

4.1.2 Experimental results for specimens with a bar diameter of 16mm

Figure 4.5 Flexural failure of specimen B11

Figure 4.6 Flexural failure of specimen B12

Figure 4.7 Flexural failure of specimen B14

4.1.3 Ultimate load vs. First cracking load

4.1.3.1 First Cracking Load (F_{cr})

The first cracking load of all tested beam specimens is given in Table 4.1. It is observed from Table 4.1 that the cracking load of all these tested beam specimens was influenced by both bar diameter accompanied by the unbounded length of the tensile reinforcement. For the specimens of beams with a bar diameter of 8mm and varied bonded length, the first cracking load was highly reduced than the specimens of beams with a 16mm bar diameter.

The first cracking load of the beam specimens (B_{21}) was 38% more than that of beam specimens (B_{11}). This is due to an increase in the bar diameter that generates bond loss and as a result propagation of cracks will be formed. The reduction in terms of cracking load was higher in beam specimens with a bar diameter of 8mm than that of beam specimens of 16mm. The beam specimens with a bar diameter of 16mm, was reduced by 18.36% for around 66.66% (B_{13}) bond loss, while 55.34% reduction for beam specimen (B_{23}) was a huge reduction.

Table: 4.1 Comparison of ultimate load (F_u) and the First cracking load (F_{cr})

Beam Specimen	**B11**	**B12**	**B13**	**B14**	**B21**	**B22**	**B23**	**B24**
$L_{eff.}$ (m)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
L_{un} (m)	0.00	0.33	0.67	1.00	0.00	0.33	0.67	1.00
$L_{un}/L_{eff.}$	0.00	0.33	0.67	1.00	0.00	0.33	0.67	1.00
First Cracking Load (F_{cr}). kN	24.51	21.5	20.01	17.51	39.52	22.26	17.65	7.25
F_{cr}/F_{crR}	1.00	0.88	0.82	0.71	1.00	0.56	0.45	0.18
Ultimate Load (F_u). kN	152.81	148.56	134.55	124.80	113.05	111.54	106.84	104.79
F_u/F_{uR} (%)	1.00	0.97	0.88	0.82	1.00	0.99	0.95	0.93
%age (Fcr/F_{crR})	100.00	88	82	71	100.00	56	45	18
%age(F_u/F_{uR})	100.00	97	88	82	100.00	99	95	93
%age difference(F_u)	0.00	2.78	12.95	18.33	0.00	1.34	5.49	7.31
%age difference(F_{cr})	0.00	12	18	29	0.00	44	55	82

Fig.4.8 shows the relation between the cracking load of the beams to the reference cracking load (F_{cr}/F_{crR}) and the beam's ultimate load with the controlled beam ultimate load (F_u/F_{uR}). Regarding the effect of bar diameter, it can be observed by comparing the results of beams

with the beam B11 which was fully bonded with a bar diameter of 16mm, 12% reduction in cracking load for 33.33% (B_{12}) of available bond avoided, while 18% and 29% reduction for beams having 66.67% (B_{13}) bond loss and fully unbonded (B_{14}). But for the bar diameter of 8mm wth the same varied length of bond loss, 44%,55%, and 82% reduction of cracking load for 33.33% (B_{22}), 66.67%(B_{23}), and fully unbonded beam (B24) respectively relative to controlled beam (B21). As we can observe from the results, the cracking load of the beam with the larger diameter, but equal steel ratio bar and the same varied length of bond loss is less than that of beams with smaller bar diameter in general. But the reduction of cracking a load of beam specimens with a bar diameter of 8mm is more than that of beam specimens with a bar diameter of 16mm as the length of bond loss increased. The cracking load of beam Specimen (B_{22}) was reduced by 44% and bam specimen (B_{23}) was reduced by 55%, while The cracking load of beam specimen(B_{12}) and (B_{13}) was reduced by 12%and 18% respectively, for the same varied length, but different bar diameter. This clearly showed that the bond loss and bar diameter affect the crack condition of reinforced concrete beams. Generally, as it is seen from the result the cracking load for the smaller diameter without a bond loss was greater than that of the larger diameter, But as the bond loss is introduced and increased in length without bond, the cracking load reduction is higher than that of the larger diameter of the bar.

4.1.3.2 Ultimate Flexural Load (F_u)

The ultimate flexural load of all tested beam specimens is given in Table 4.1. It is seen that the ultimate flexural load of the tested beam with 16mm bar diameter accompanied with varied unbonded length ($B_{11,}$ B12, $B_{13,}$ and B_{14}) along tensile reinforcement was 26.02%,24.92%,20.59%, and 16.03% more than that of the beam specimens cast with 8mm bar diameter ($B_{21,}$ B22, $B_{23,}$ and B_{24}) for similar varied bond length and equal steel ratio respectively. This observation showed that the ultimate flexural load of all tested beam specimens was highly influenced by both bar diameter even if there is bond loss and bonded length along with tensile reinforcement. This is because as, bar diameter increases, bond strength reduces while load carrying capacity increases. And as the bond between steel and concrete decreases or is avoided the area of steel will reduce and as a result the tensile strength of steel will be lost, then the steel yields which is followed by crushing of concrete.

The relation between the ultimate load and the unbonded length is linearly proportional while the relation between cracking load and unbonded length is polynomial as shown in figure 4.9 below.

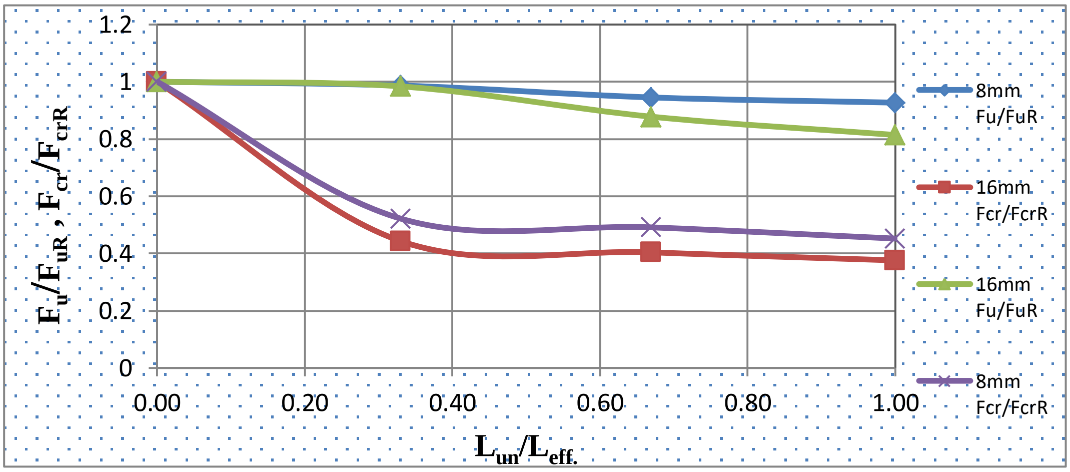


Figure 4.8 comparison of cracking load and ultimate load as a function of bonded length for both bar diameter

4.2 Cracking pattern and mode of failure

The cracking patterns of controlled beams (B_{21} and B_{12}) was combined flexural cracks, that the flexural cracks were accompanied by the crushing of concrete as shown in figure 41 and 4.5 which was not the desired failure pattern. This flexural shear cracking happens via three main stages. Initially, the crack starts to form at the bottom of the beam as a result of flexural tensile stress. Next, the crack bends in a diagonal direction as it goes towards the upper end of the beam along with continuation to propagate in length and widen in width. And, finally, as the diagonal tensile stress becomes beyond the tensile strength of the concrete, the concrete fails by crushing. The flexural failure begins as a flexural crack and ends with the failure of concrete in shear. The location of the flexural shear failure is at both the maximum shear force and maximum moment. While for the rest of the beams of the cracking pattern is a ductile flexural pattern (B_{22}-B24 and B_{12} to B_{14}) But the cracking form in the beams with a bar diameter of 8mm were more ductile flexural failure than that of beams with a bar diameter of 16mm as expected as ductility is higher for smaller bar diameter than the larger diameter.

As we have seen from the figures above, for both types of beams (beams with a bar diameter of 8mm and 16mm), as the bond length decreased, the crack width increased. And as the length of unbounded length increased, while the number of cracks decreased. For the beams with higher bond loss; for over 65% of bond loss for both types of beams, the flexural failure is a brittle flexural failure as the beam suddenly fails without warning sign in flexure. This is because of the location of bond loss of the beams that the artificially provided unbounded length was starting from the middle (maximum bending moment) toward the supports which is different from what is experimentally studied before and this location of bond loss increases its effect on flexural behavior as the deflection starts from mid-point where the maximum bending moment has occurred.

To sum up, it is observed that the cracking pattern of all tested beams was affected by both diameter and varied bond loss length as shown in figures 4.1- 4.7 above

4.3 Variation of the ultimate flexural load as a function of bond loss length

The most obvious observation from the experimental investigation is the reduction of flexural load-carrying capacity as the unbonded length between concrete-steel reinforcement increases, as shown in fig.4.10. The reduction in flexural load-carrying capacity was surprisingly much lower in the case of using a bar diameter of 8mm than beam specimen with 16mm bar diameter,i.e in the case of beams with a diameter of a bar of 16mm the reduction of flexural load-carrying capacity was about 11.95% for 66.67% of the available bond length is avoided and 18.6% reduction for full bond loss with a bar diameter of 16mm, while the reduction in load-carrying capacity for bar diameter of 8mm for the same unbonded length was very small even for the fully removed bond, was only about 7.31%.

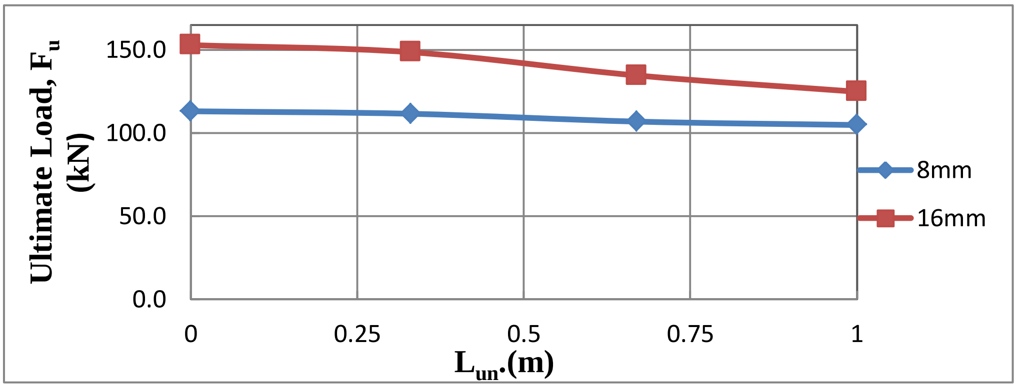


Figure 4.9 Flexural Load carrying capacity as a function of loss of bond

4.4 Load-Deflection and Moment-Deflection Relation

Table 4.2 Variation of deflection at approximated service load related to a controlled beam

Beam	Fs (kN)	ds (mm)	ds/dsR	Variation	Relation with respective controlled beams
B11	76.4	1.44	1.00	1.00	_
B12		2.69	1.87	0.87	1.87 times
B13		3.19	2.22	1.22	2.22 times
B14		3.31	2.30	1.3	2.30 times
B21	56.55	0.75	1.00	1.00	_
B22		1.53	2.04	1.04	2.04 times
B23		2.62	3.49	2.49	3.49 times
B24		3.40	4.53	3.53	4.53 times

Note: 'Fs' is service Load approximately taken as 50% of ultimate load of respective controlled beam

4.2.1 Load-deflection and moment-deflection curve for the beam with 8mm bar diameter (group-1)

Figure 4.10a shows the load-deflection diagram for the beam with a bar diameter of 8mm including a controlled beam. From the figure, it is seen that the loss of bond of 33.33% (B_{22}) changes its behavior after the cracking point following the higher reduction of stiffness (slope of the load-deflection curve) related to the controlled beam (B_{21}). Beams with the bond loss of 66.67% and 100% (B_{23} & B_{24}) behaved almost linear relation of load-deflection with a high reduction of stiffness relative to the controlled beam with full bonded. At a service load of approximately 56.55kN, the deflection increased as the available bond avoiding was increased and observed that 2.04 times that of the controlled beam for over 33% bond loss and 3.49 times and 4.53 times increments of deflection for losing of the bond over 66% and fully unbounded respectively as shown in table 4.2. The ultimate deflection of these beams (B_{22} to B_{24}) was slightly reduced to 90.7%, 83.0%, and 73.3% relative to the controlled beam respectively. This is due to the reduction of the beam ductility as the length of bond loss increases. The results showed that there was much variation of stiffness (initial load-deflection curves) and variation in deflection for all tested beams with a bar diameter of 8mm for this varied length of bond loss. The higher ductility was associated with the beam with a small length of bond loss and controlled beam because of the existence of a bond of steel between stirrups and concrete.

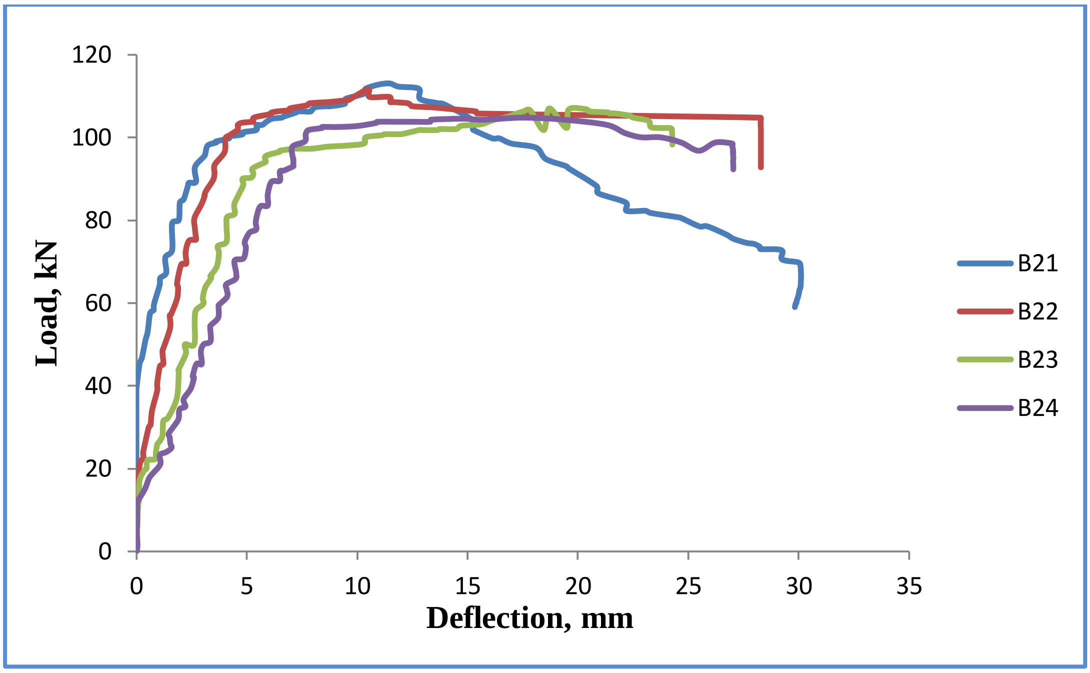


Figure 4.10a. Load-Deflection graph for 8mm diameter bar

4.2.2 Load-deflection and moment-deflection curve for the beam with 16mm bar diameter (Group-2)

Figure 4.11a shows the load-deflection diagram for the beam with a bar diameter of 16mm including the controlled beam. From the figure, it is observed that all beams do not change the behavior starting from the beginning up to the cracking point. But after the cracking point higher reduction of stiffness (slope of the load-deflection curve) relative to the controlled beam (B_{11}). Beams with the bond loss of 33.33% (1/3) higher reduction of stiffness (slope of the load-deflection curve) compared to the controlled beam (B_{11}). While the beams with (66.67% (*2/3*)) (B_{13}) and 100% (B_{14}) bond loss showed more reduction of stiffness (slope of the load-deflection curve) relative to the controlled beam (B_{11}). This is because flexural load carrying capacity decreased as the bond loss between concrete and steel bar increased due to the diameter of bar increases in addition to artificially provided bond loss, then, cracks will occur and this leads to a reduction in resistance to external load reduces, which results reduce load carrying capacity of the beams. In addition to this, the ductility of the beam will also be lost as well.

At a service load of approximately 76.4kN of beams, the deflection increased as the available bond between concrete and steel, avoided was increased and observed that increment by 1.87 times relative to the controlled beam for over 33% (B_{12}) bond loss and 2.22 times and 2.3 times increments of deflection for loosing of the bond over 66% (B_{13}) and fully unbounded beam (B_{14}) respectively as shown in table 4.2. The ultimate deflection of beam specimens (B_{12} to B_{14}) was 88.4%, 77.8%, and 71.5% of the controlled beam respectively. This is due to the reduction of the beam ductility as the length of bond loss increases. The results showed that much reduction of stiffness (initial load-deflection curves) for beams (B_{13} & B_{14}) and variation in deflection for all tested beams (B_{12} to B_{14}) with a bar diameter of 16mm. Generally, ductility and stiffness (load-deflection curves) reduced as the length of bond loss increased along with the tensile reinforcement.

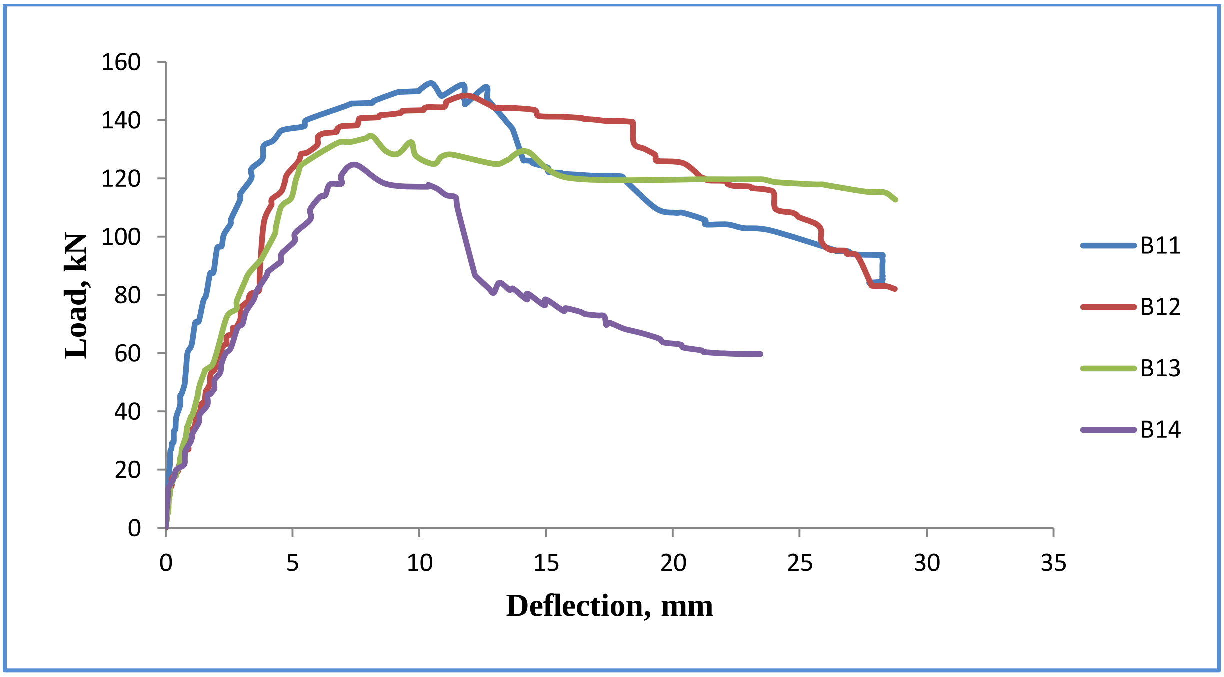


Figure 4.11a. Load-Deflection graph for 16mm diameter bar

4.2.3 Effect of bond loss location and length on flexural behavior of Reinforced concrete beams

The location and the length of bond loss in tension reinforcement were highly affected the flexural response of RC beams as observed from the result. As shown in figure 4.4 to figure 4.7, as the bond loss length increases from mid-span towards supports number of cracks is reduced and the width of cracks became wider and wider. Beam specimen (B_{12}) was with many cracks with one wider width of crack which occurred at the mid of the beam. For the beam specimen (B_{14}) the propagation of cracks was decreased in number but wider in width of cracks than beam specimen (B_{12} and B_{13}). This indicates that as the bar diameter combined with bond loss length increases the cracks pattern and the failure situation was affected highly which was a brittle flexural failure as the bone loss increases.

The number of cracks for beam specimens (B_{21}, B_{22}, $B_{23,}$ and B_{24}) with a bar diameter of 8mm is greater than that of the beam with a bar diameter of 16mm for the same bond loss length. This is due to the good bond strength formation of smaller bar diameter and more number of mini cracks that indicate the ductility of the member rather than a wider and small number of cracks as it has happened in beam specimens of bar diameter of 16mm, which shows the more stiffness but less ductility i.e., brittle flexural failure.

The reduction in the number of cracks was also affected by the location of bond loss for both bar diameters as compared to previously conducted experiments with the bond loss starts around supports toward the mid of the beams(*Magda I.Mousa,2016*).

There was also a rapid reduction in load-carrying capacity of a beam with a bar diameter of 16mm than beam specimen with a bar diameter of 8mm for similar bond loss conditions. This is due to the location of bond loss in addition to the bar diameter, as compared with the experimental investigation of (Magda I. Mousa, 2016) in which the reduction of the load-carrying capacity of beam specimens for different bond loss lengths is small, it is 12.5% reduction of load-carrying capacity even for a beam with 73% of bond loss, but for beam specimen (B_{13}) the reduction of flexural load carrying capacity was 11.95% even for the bond loss of 66.66%. This is due to the reduction of tensile strength at the middle where the load is applied and no bond strength at the crossing point of transverse reinforcement and longitudinal reinforcement which initiates the relative movement rather withstand the applied load.

4.2.4 Comparison of the result for the two bar diameters

For the beams with bar diameters of 8mm and 16mm having the same variable length of bond loss for steel bar reinforcement.

1. The load-carrying capacity of the beam (B_{13}) with a bar diameter of 16mm with the same variable bond loss length was reduced by 11.95% for bond loss of over 65%, while for the beam(B_{24}) with a bar diameter of 8mm the reduction in load-carrying capacity was 7.31% even for fully unbounded length. This is attributed to the smaller

bar diameter that as bar diameter decreases, the bond strength increase. The result has shown that as the diameter of the steel reinforcement reduces the bond strength and ductility increase, whilst the stiffness decreases. This is due to a decrease in the rate of bar confinement caused by the opening of flexural cracks along with the reinforcement and due to significant change at the steel-concrete interface (i.e. roughness to smooth that leads to reduction of bar cross-section).

2. The reduction in the first cracking load of the beams with 8mm bar diameter was greater than that of the beams with 16mm bar diameter for the same length of bond loss, which was recorded as 43.7% for beam (B_{22}) with a bar diameter of 8mm having the bond loss length of 65%, while for the beam (B_{12}) with 16mm of bar diameter was observed as 12.3% reduction for the same length of bond loss (33.33%). The reduction of load at cracking was observed as 55.3% and 83.6% for beam specimens B_{23} and B_{24} respectively, while for beam specimens B_{13} and B_{14} the load at cracking was decreased by 24.5% and 51.0% respectively.
3. The ultimate deflection of the beams (B_{22} to B_{24}) with a bar diameter of 8mm was reduced by 9.32%, 16.99%and 26.74% respectively compared to the controlled beam, while reduction of ultimate deflection for the beams with the bar diameter of 16mm for the same length of bond loss was recorded as 11.64%, 22.23%and 28.53% which was higher than that of beams with 8mm bar diameter. The larger deflection was observed for beam specimens with a bar diameter of 8mm for the same bond loss length with beam specimens prepared with a bar diameter of 16mm. This is attributed to the smaller bar diameter that as the diameter of the bar decreased the bond between concrete and steel bar reinforcement will increase.
4. As the results from both groups of beam specimens shown, the beam specimens with a bar diameter of 16mm showed higher stiffness due to larger bar diameter but less ductility and small deflection due to low bond strength and occurrence of flexural cracks which reduces ductility. But for bar diameter of 8mm beam specimens the reverse is true, that larger deflection and before fracture which shown ductile properties.

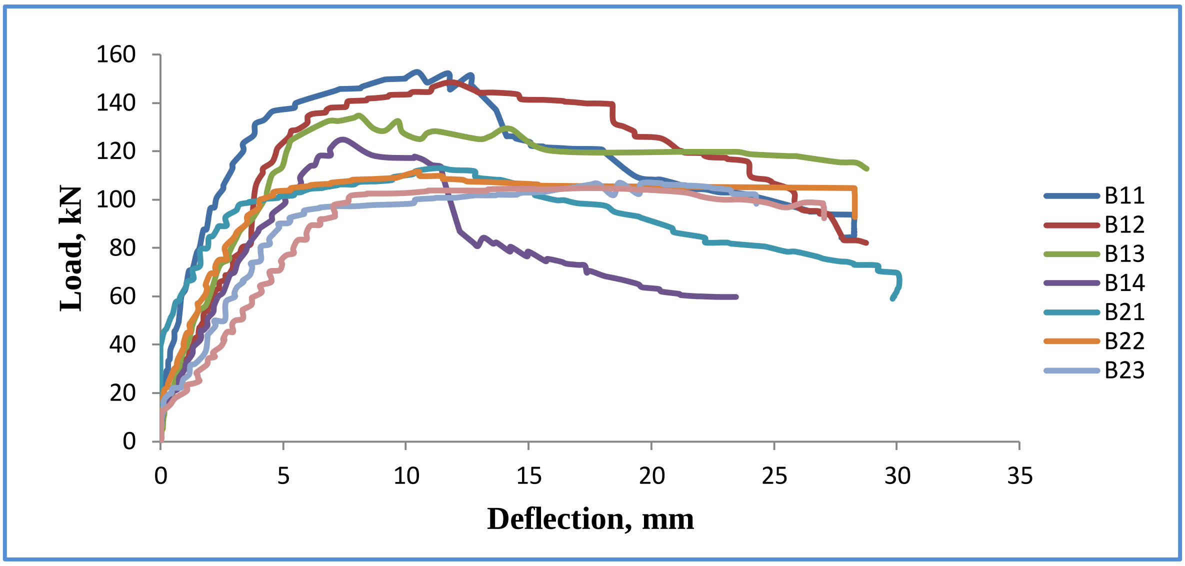


Figure 4.12. Load-displacement graph for all tested beams in one

CHAPTER 5 CONCLUSIONS AND RECCOMENDATIONS

5.1 CONCLUSIONS

1. The cracking load was significantly reduced with about 43.67% for the beam having 33.33% (B_{22}) unbounded length of tension reinforcement with 8mm bar diameter. The reduction of first cracking loads for beams (B_{23} &B_{24}) increased to 55.34% and 81.70% the bond loss between the concrete and the steel reinforcing bar was increased to 66.67% and fully unbounded respectively.

2. A moderate reduction of first cracking load for the beams with 16mm bar diameter for about 33.33% bond loss was observed as 12.28% and this reduction increased to 18.40% and 28.60% s the bond loss increased to 66.67% and fully unbounded respectively.

3. A conservative load-carrying capacity of beams designed with 16mm bar diameter was very small for the length of bond loss of about 33.33% (B_{22}) with a value of 3% reduction was recorded. But as the bond loss length of reinforcement increases, it was observed that 12% and 18% reduction for the bond loss length of 66.67% (B_{23}) and fully unbounded beams (B_{24}) respectively.

4. The reduction in load-carrying capacity of beams designed with a bar diameter of 8mm was very small even for a beam fully unbounded beam specimen (B_{24}) was only reduced by 7.31%. This may be attributed to the good bond strength of the smaller bar diameter, in addition to the bond around the end supports and the presence of the anchorage.

5. The ultimate deflection of beams at ultimate load, with unbounded reinforcement, decreased as the length bond loss increased from middle span toward supports compared to controlled beams with fully bonded and showing lower stiffness at the same time for both groups of beams. With increasing the rate of bond loss, the ultimate defection of beams decreased; avoiding bond length about 66.67%, decreased the ultimate deflection by 22.23% and 16.99% for two types of beams designed for

16mm and 8mm bar diameter respectively compared to the respectively controlled beams. All tested beams with unbounded reinforcement reflected lower stiffness and ductility than that of a controlled beam. This is because the loss of bond between reinforcing materials and concrete decreases the cross-sectional area of steel and avoids the integrity of the two materials and allows the relative movement that results in cracks.

6. But, the deflection of beams for unbonded beams is higher than that of the controlled (reference) beam at equal load that indicates lowering of stiffness. As the rate of bond loss increased, deflection increased in general. At an approximated service load (76.4kN) of beam specimens with a bar diameter of 16mm, the deflection raised by 1.87, 2.22, and 2.3 times the deflection of the reference beam for beam specimens B_{12}, B13, and B_{14} respectively. And 2.04, 3.49, and 4.25 times the controlled beam for beam specimens B_{22}, $B_{23,}$ and B_{24} respectively.

7. Generally, in all tested beams as the length of bond loss of reinforcement increased, the number of cracks decreased and increased the width of cracks. And the higher deflection is attributed to the beam specimens with the smaller bar diameter which indicates the beam with a larger bar diameter is affected more by bond loss.

8. The result between beams with a bar diameter of 8mm and 16mm and having the same varied unbounded length, shown that reduction of load-carrying capacity, first cracking load, and ultimate deflection of beams with bar diameter 8mm is less than that of beams with a bar diameter of 16mm. this is due to the contribution of small bar diameter which can create a good bond and ensure ductility than that of larger bar diameter.

9. Generally, the location of bond loss in reinforced concrete beams affects highly the pattern of the cracks and other flexural behaviors depending on the length of unbounded length. It shows that the number of cracks decreased as the length bond loss increased, and then as a result the failure mode changed to brittle flexural failure as the length of bond boss increased.

5.2 RECOMMENDATIONS

1. All tested beams needed to be failed in flexure, but the undesired mode of failure was observed on beam specimens that were a combined mode of failure has occurred even if it were designed for flexural failure. So, this needs further investigations on the causes of this situation.
2. Tests shall be conducted on the short beams without anchorage by using different bar diameters to study the effect of unbonded longitudinal reinforcement on flexural behaviors of RC beams.
3. An experimental investigation shall be conducted on the effect of concrete compressive strength with unbounded longitudinal reinforcement on flexural behaviors of RC beams.
4. The effect of beam size with unbounded longitudinal reinforcement on flexural behaviors of RC beams.
5. Comparisons of computer analysis and experimental investigation on the effect of concrete compressive strength with unbounded longitudinal reinforcement on flexural behaviors of RC beams.

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APPENDIX A

Appendix A: Mix design (ACI Method)

C25 Mix Design

Mix Design procedure

Mix Design with ACI Method

Mix Component Properties:

Cement Type: Dangote OPC (42.5 R)

Strength: 25MPa

Margin: Specific Gravity: 3.15

Concrete grade: C25

Characteristic 8MPa for strength less than 26.25 MPa, Target strength is fc'+8

Target Mean Strength: 33MPa

MIX DESIGN PROCEDURE

Step 1

Choice of the slump:

For Beams and Reinforced Concrete Walls from Table A1.5.3.1, the required slump is 25-100

Step 2

Choice of maximum size aggregate: 19mm

Step 3

Estimation of mixing water (Table A1.5.3.3)

For 75-100mm slump and 19mm nominal maximum size of aggregate mixing water should be 205 kg/m^3

Step 4

Selection of Water Cement ratio

From Table A1.5.3.4 (a): The Water Cement Ratio should be 0.56

Step 5

Calculation of Water Cement content

This is 366.07 kg/m3 (205/0.56)

Step 6

Estimation of coarse aggregate content Table A1.5.3.6

For 2.59 fineness modulus and 19mm nominal aggregate size, the volume of coarse aggregate is 0.641m^3/m^3

Weigh of coarse aggregate per cubic meter of concrete should be 1011.71(Bulk density of coarse aggregate *coarse aggregate content=1578.33*0.641=1011.71kg.

Step 7

Estimation of fine aggregate content (Massa basis).Table A1.5.3.7.1

For 19mm nominal maximum size of coarse aggregate and non-air entrained concrete mass of 1m^3 of concrete is 2345 kg/m^3

Table A.1 Weight of ingredients for 1m^3 of fresh concrete:

Descriptions	Amount	Unit
Water	205	kg
Cement	366.07	kg
Coarse aggregate	1011.71	kg
Total	1569.66	kg

Mass of fine aggregate is calculated by subtracting the total of ingredients in the table from the mass of fresh concrete, the result is 851.94kg.

Step 8

Table A.2 Adjustment for aggregate Moisture

Descriptions	Amount		Unit
Coarse aggregate (wet)	1011.71*1.014	1025.87	kg
Fine aggregate (wet)	851.94*1.0529	897.01	kg

Water Absorption Adjustment

Mixing water from coarse aggregate is 1011.71*(0.014-0.0092) =4.8kg

Mixing water from fine aggregate is 897.01*(0.0529-0.0432) =8.7kg

Total Mixing water from coarse aggregate and fine aggregate is 14kg

The estimated requirement of added water for mixing is (205-14) =191.5kg

So,

Table A.3 The Estimated batch for a cubic meter of concrete is:

Descriptions	**Amount**	**Unit**
Water (to be added)	191..5	kg
Cement	366.07	kg
Coarse aggregate(wet)	1025.87	kg
Fine aggregate (wet)	897.01	kg
Total	2480.45	kg

Step 9

Table A.4 The required percentage of ingredients

Descriptions	**Amount**	**%**	**Unit**
Water (to be added)	191.5/2480.45	7.72	kg
Cement	366.07/2480.45	14.76	kg
Coarse aggregate(wet)	1025.87/2480.45	41.36	kg
Fine aggregate (wet)	897.01/2480.45	36.16	kg
Total	2466.83/2480.45	100	kg

Step 10

Table A.5 The required amount of ingredients for a volume of 0.431 m^3

Descriptions	Amount	Unit
Water (to be added)	82.54	kg
Cement	157.78	kg
Coarse aggregate(wet)	442.15	kg
Fine aggregate (wet)	386.61	kg
Total	1069.08	kg

Appendix B: Load-Deflection Results from the test

Table B.1 Experimental Results Load deflection values for the sample B_{11}

No.	Load, kN	Disp. Mm	Continued…			Continued…		
1	0.00	0.00	**78**	112.80	2.93	**129**	93.54	28.27
2	0.50	0.00	**79**	114.80	2.94	**130**	93.54	28.26
3	1.00	0.00	**80**	120.05	3.37	**131**	93.29	28.27
4	1.25	0.00	**81**	123.30	3.37	**132**	93.29	28.26
5	1.75	0.00	**82**	126.80	3.81	**133**	92.54	28.26
6	1.75	0.01	**83**	131.30	3.86	**134**	92.04	28.27
7	2.00	0.01	**84**	133.05	4.23	**135**	91.54	28.27
8	2.25	0.02	**85**	136.30	4.53	**136**	90.54	28.26
9	2.25	0.03	**86**	137.05	4.78	**137**	89.54	28.26
10	2.50	0.03	**87**	138.06	5.47	**138**	88.79	28.26
11	2.50	0.03	**88**	140.06	5.53	**139**	88.29	28.26
12	2.75	0.03	**89**	145.06	7.12	**140**	87.79	28.27
13	2.75	0.03	**90**	145.81	7.31	**141**	87.79	28.26
14	3.00	0.04	**91**	145.81	7.39	**142**	87.54	28.26
15	3.50	0.04	**92**	146.06	8.15	**143**	87.03	28.26
16	3.50	0.04	**93**	146.31	8.16	**144**	86.78	28.27
17	3.50	0.04	**94**	146.81	8.23	**145**	86.53	28.27
18	4.00	0.04	**95**	149.56	9.09	**146**	86.53	28.26
19	4.25	0.05	**96**	149.81	9.24	**147**	86.53	28.27
20	4.75	0.05	**97**	150.06	9.98	**148**	86.28	28.26
21	5.00	0.05	**98**	150.31	9.98	**149**	86.03	28.26
22	5.50	0.05	**99**	152.81	10.48	**150**	86.03	28.26
23	6.00	0.05	**100**	148.56	10.84	**151**	86.03	28.26
24	7.25	0.05	**101**	148.56	10.93	**152**	86.03	28.26
25	7.50	0.06	**102**	150.31	11.72	**153**	85.78	28.26
26	8.25	0.06	**103**	147.56	11.76	**154**	85.78	28.27
			….Continued…					
27	8.25	0.06	**104**	146.56	11.81	**155**	85.53	28.27

28	8.50	0.06	**105**	145.56	11.81	**156**	85.53	28.26
29	8.50	0.06	**106**	151.56	12.62	**157**	85.53	28.26
30	9.50	0.07	**107**	147.31	12.66	**158**	85.28	28.26
31	9.50	0.08	**108**	146.81	12.73	**159**	85.28	28.27
32	10.00	0.08	**109**	137.81	13.61	**160**	85.28	28.27
33	10.25	0.08	**110**	136.81	13.69	**161**	85.28	28.26
34	11.50	0.08	**111**	126.30	14.10	**162**	85.03	28.26
35	16.51	0.12	**112**	126.30	14.15	**163**	84.78	28.26
36	18.76	0.12	**113**	126.05	14.44	**164**	84.78	28.26
37	21.51	0.16	**114**	125.30	14.48	**165**	84.78	28.25
38	22.76	0.16	**115**	123.80	15.06	**166**	84.78	28.25
39	24.76	0.17	**116**	122.30	15.10	**167**	84.78	28.25
40	26.51	0.18	**117**	122.05	15.59	**168**	84.78	28.25
41	27.26	0.22	**118**	121.80	15.46	**169**	84.53	28.24
42	29.26	0.25	**119**	121.30	16.40	**170**	84.53	28.24
43	29.51	0.31	**120**	121.05	16.80	**171**	84.53	28.24
44	33.26	0.32	**121**	120.80	17.97			
45	34.01	0.38	**122**	119.30	18.13			
46	38.02	0.41	**123**	109.79	19.33			
47	42.02	0.56	**124**	108.29	20.10			
48	45.52	0.57	**125**	108.29	20.39			
49	46.02	0.62	**126**	105.79	21.27			
50	49.27	0.74	**127**	104.29	21.28			
51	50.02	0.75	**128**	104.29	22.15			
52	54.77	0.80	**78**	103.04	22.78			
53	60.27	0.86	**79**	102.29	23.82			
54	63.03	1.02	**80**	95.29	26.45			
55	70.53	1.16	**81**	95.04	26.45			
56	71.03	1.30	**82**	95.04	26.45			
57	78.03	1.48	**83**	95.04	26.92			
58	80.03	1.59	**84**	94.04	27.11			
59	87.54	1.75	**85**	93.79	28.27			
60	87.79	1.88	**86**	93.54	28.27			
61	96.29	2.03	**87**	146.56	11.81			
62	96.79	2.20	**88**	145.56	11.81			
63	100.79	2.29	**89**	151.56	12.62			
64	104.54	2.56	**90**	147.31	12.66			

The load-deflection test results of other beams can be received from the data logger at construction materials testing and laboratory of AAiT with file names of b12_000a, b13_000a, b14_000a, b21_000a, b22_000a, b23_000a, and b24_000a respectively.