

ADDIS ABABA UNIVERSITY
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FLEXURAL PERFORMANCE OF CONCRETE MEMBERS
REINFORCED WITH MECHANICALLY SPLICED BAR

BY:

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SEPTEMBER 2020

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**FLEXURE PERFORMANCE OF CONCRETE MEMBERS REINFORCED
MECHANICALLY SPliced BARS**

**A Thesis Submitted to the School of Graduates Studies in Partial Fulfillment
of the Requirements for the Degree of Master of Science in
Civil Engineering (Structures)**

BY:

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SEPTEMBER 2020

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SPLICED BARS

STATEMENT OF DECLARATION

I hereby declare that this thesis is my original work; it has not been submitted for any Degree/Diploma in any university. Different online documents and hard copies used in the preparation of this thesis have been duly acknowledged.

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ACKNOWLEDGEMENTS

Primarily I would like to express my deepest gratitude to my study sponsorship, Ethiopian Road Authority (ERA) and Addis Ababa University institute of technology (AAiT). I would also like to thanks my advisor Dr.-Eng. Girma ZeraYohannes for his professional and technical advice; and AAiT supervision staff at commercial bank of Ethiopia, new head quarter design build project such as Eyoel Bezabh, Dr.Girmay Kahssay, Dr.Adil Zakarya, Mohammed Siraj and Tesfaye Molla for their valuable assistance and advice on specimen fabrication process and technical advice during this study.

Finally, I would like to thanks my friends Nuhamin Getachew and Nejib Abdurezak their cooperation in doing this research in terms of logistics and paper work and also Laboratory class representatives Demessew Melaku and Fikru Bedada for their cooperation.

Shemsedin Leja

September 2020

ABSTRACT

Flexural performance test was conducted on five reinforced concrete beams. The beams were reinforced with steel bars which were connected using couplers. All the connections were at mid-span. This investigation incorporates two types of mechanical connection detailing. Three of the beams were reinforced with all the bars connected by couplers, one beam was reinforced with 50% of the reinforcing steel bars connected by coupler and at constant moment region of the mid span while the remaining beam was a control member with no mechanical splicers.

All the beams were subjected to cyclic loading. The range of stress was the predominant factor to determine the flexural behavior of mechanically connected bars at the joint within the concrete beams specimens. Beams reinforced with 100% mechanically connected bars developed similar flexural performance and failure modes. Five reinforced concrete beams were cast for this research purpose. The beams were loaded with cyclic loading and all specimens had almost the same general behavior at respective maximum loading. Furthermore, it was observed that spliced longitudinal re-bars caused the yield load to be reduced by 13.5% and yielding displacement increased by 10.33% with different maximum capacity. In addition to these the ductility of specimens decreased by 9.5% compared to B1. As per the observations of this study parallel threaded coupler connected bars satisfying the manufacturer criteria's can be considered as a continuous bar with increased strain, deflection and cracking properties at the joint.

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LIST OF ABBREVIATIONS

AAIT	Addis Ababa Institute of Technology
ACI	American Concrete Institute
BS	British Standards
BBA	British Board of Agreement
CARES	Certification Authority for Reinforcing Steels
CBE	Commercial Bank of Ethiopia
IEC	International Electro technical Commission
IS	Indian Standards
ISO	International Organization for Standardization
JSCE	Japan Society of Civil Engineers
MS	Mechanical Splicers
PPC	Portland Pozzolana Cement
RC	Reinforced Concrete
UK	United Kingdom
CIB	Coupler incorporated beam

1. INTRODUCTION

1.1. STATEMENT OF THE PROBLEM

This study aims to assess the flexural performance of reinforced concrete beams with mechanical coupler spliced re-bars and to examine the behavior and seismic performance of reinforced concrete structures with mechanically spliced bars.

1.2. BACKGROUND OF THE STUDY

The design of reinforced concrete structures is based on several basic assumptions. Among these fundamental concepts is that reinforcement will be provided to resist any tension that possibly exists in the member. Moreover, it is assumed that the reinforcement should be anchored properly and be continuous in the regions of tensile stress. But the length of the reinforcing bar is limited by fabricating, transporting and storage capacities as well as simplicity of construction, all of which make splicing necessary. The standard length of steel bars cannot ensure the integrity of reinforcement throughout any sizeable members or flat slabs and one way slabs. Additionally, when the span length of the member is less than 12 meters, splicing off unnecessary length is required. Thus, in most situations where splices of reinforcement are involved, the splice choice may be lap splice or mechanical splice (coupler connection or welding).

Lap splice is straightforward concept with the length of lap assumed to be directly related to the concrete grade. However, there may be significant problems in fixing the location of splices, as it may not be applicable to tension zone of the member. For example, in dome structures, beams are sometimes in tension throughout the span. Furthermore, additional cover or confinement required to control the lap length so as to reach the capacity needed in the length provided which also depends on concrete compressive strength. Nonetheless, even after meeting all of the code requirements, there are further challenges upon construction.

The splicing of large diameter bars can lead to required lengths easily over a meter in length, a requirement that is not easily met in many situations. Therefore the contractor/designer may look

at other alternatives of splice technology, such as welding and coupler connectors. The welding option needs special tests and is very technical. It is required to produce a ductile weld.

Moreover, the cost and practicality are also factors of consideration. This leads clients and contractors that are wary of the risks of these options to consider coupler connection technology as a superior alternative as it connects steel bars end to end. This method ensures the reinforcement behave in a manner similar to continuous length bars. In order to ensure the performance of coupler connected steel bar, several tests and researches have been done by scholars of different levels from post graduate to undergraduate. Nowadays, countries codes incorporate this splicing system as one way of connecting two separate bars end to end; but requirements like staggering length, coupler positions and yield strength of the joints are excessively limited. In order to investigate the limitation of those requirements, this study has focused on the flexural performance of thread coupler connected steel bar reinforced members.

1.3. OBJECTIVE OF THE RESEARCH

1.3.1. GENERAL OBJECTIVE OF THE RESEARCH

The objective of this study is primarily to investigate the influence of mechanically connected reinforcement bars on flexural performance of concrete members. The flexural behavior of coupler connected steel bar reinforced concrete beams would then be investigated.

1.3.2. SPECIFIC OBJECTIVE OF THE RESEARCH

- To identify the major problems of locally produced bar connected by parallel thread coupler reinforced beams, if any.
- To provide comparative analysis of flexural behavior of coupler incorporated steel bar reinforced concrete beams and equivalent non-spliced steel reinforced concrete beam for cyclic loading (asses the seismic feasibility).
- Evaluating test results comparing with other data obtained from literature.
- To introduce the topic for future researches and areas that may need detail investigations.

1.4. LIMITATION OF THE STUDY

There are different types of coupler (Mechanical connectors) available in the construction industry. But this study focuses only on parallel threaded coupler connected bar reinforced beams, as parallel threaded coupler was the modified widely used mechanical connectors on high rise buildings and stadiums. This experimental study was also carried out only on simply supported beams, thus it requires detailed investigation for other support cases and frame structures on local materials.

1.5. SIGNIFICANCE OF THE STUDY

This study aimed to discuss the flexural performance of the coupler connected bar reinforced members, the effectiveness of the system on seismic zones by doing experimental investigation, and introduce the advantage or disadvantages of using parallel threaded coupler technology to local construction industry. It will also serve as an introductory research for readers and professionals who wish to study the seismic performance of the system on frame structures.

1.6. RESEARCH METHODOLOGY

In order to achieve the aforementioned objectives, experimental analysis was conducted to study the failure mode, crack pattern, crack width and load to displacement response of the coupler incorporated steel bar reinforced concrete members. Flexural test for members was performed. The difference in the behavior of reinforced concrete members with conventional reinforcing bars and coupler incorporated bars were investigated through an experimental study.

Experimental study included load testing of five reinforced concrete beam specimens which had mechanically coupled reinforcement while one of the specimens was reinforced with conventional reinforcing bars as a control specimen. The reinforcing bar yield strength was measured and used in the analysis.

The compressive strength of concrete designed was C-30. The maximum moment due to live loading of 200KN through spreader beam was determined as 45.5KNm and the maximum shear on the span due to live load was found to be 101.4KN. The reinforced concrete beams were designed and produced using C-30 class concrete and locally available deformed steel bars. The

actual strength of concrete and steel has been used in the analysis of the experimental result. For laboratory testing convenience and simplicity purpose, the span length of the beam was limited to 1500mm, with width of 200mm and a depth of 280mm. The beams were designed for flexure and shear. The flexural reinforcement area was 402.12mm^2 was used to investigate the flexural behavior, considering that the beams are required to fail by flexure. Web reinforcement is provided as two leg $\phi 8@100\text{mm}$ spacing for constant shear area of the span on both sides in order to avoid shear failure before flexural failure.

The beam is also designed for deflection limits according to EBCS-2, provided that the minimum depth required for deflection specified by the code is maintained. Also minimum depth requirement for beams and one-way slabs according to ACI code has been checked and satisfied. In terms of span length, EBCS-2 provides minimum effective depth, d , to be used to control deflection as follows:-

$$\text{➤ } d = (0.4 + f_y/400) * l_e / \beta_a, \text{ where } \beta_a = 20 \text{ for simply supported members [9]}$$

Therefore, the assumed depth of specimens was sufficient for both codes' deflection control requirements. The beams had a constant moment length between two loading points of 433mm. The ends of the reinforcing bars were 90° hooked in order to improve the anchorage to concrete, and the shear reinforcing stirrups were anchored at 135° with open loop and closed loop staggering for simplicity while placing concrete.

The beam specimens were cast at commercial bank of Ethiopia new head quarter compound using ready mix concrete that had a target compressive strength of 30Mpa. The concrete compressive strength at the day of test is given in the appendix (Table 1D). Beam specimens were instrumented with displacement transducers prior to load testing. Moreover, before placing the specimens in the testing machine, the bearing surfaces of the supporting and loading rollers were wiped clean, and all loose sand or other material were removed from the surfaces of the specimens that would make contact with the rollers. The specimens were then placed in the machine in such a manner that the load is applied to the uppermost surface as cast in the formwork. The axes of the specimens were carefully aligned with the axis of the loading device. The load would then be increased cyclically until the specimen fails, and the maximum load

applied to the specimen during the test would be recorded. The appearance of the fractured faces of concrete and any unusual features in the type of failure were also recorded.

A lower limit of 0.25kN was selected to represent an applied dead load due to spreader beam; the upper limit was selected to develop the desired strain and therefore stress levels in the 2Ø16 reinforcing bar and across the splice. Crack widths were measured along pure flexure region by using millimeter ruler and deflections were measured using displacement transducers. After the yielding of the beam, the test was continued by applying load controlled by displacement proportional to control beam displacement until failure. All of the data was recorded on each step by data decoder and detail investigation of the results are presented on chapter four.

1.7. DOCUMENTATION

This study contains experimental investigation carried out on mechanically spliced steel reinforced concrete beams to determine the effect of couplers on flexural performance of the beams. Chapter 1 contains background of the study, objective of the study and a brief overview of the work done. Chapter 2 presents review of literature on reinforcement coupler systems; typically addressed are general characteristics, categories, benefits and engineering considerations as well as the use of couplers for high rise buildings, related cost benefits and different journals on mechanically coupled bars and reinforced concrete beams with a review on mechanically spliced steel bar reinforced beam specimens. Chapter 3 presents the experimental program in detail including material fabrication and specimen preparation. Chapter 4 presents the result of the experimental program i.e. beam test output with primary results, discussions on the cracking behavior of the beams and failure modes. Chapter five includes conclusions and recommendations to be considered in design and additional investigation to be carried out by future researchers. Finally, references used to carry out this experimental study are cited and acknowledged, while sections which are not included in main body are presented and explained at the end in the appendix.

2. LITERATURE OF REVIEW

While researchers tried to find out each and every documents to study the influence of using mechanical coupler to connect reinforcing bars, some national publications currently evaluating couplers for the splicing of steel bars, which are mainly tested in isolation [4], i.e. without being inserted into a reinforced concrete structure. Even internationally, the subject is quite new and it is only in the last 20 years that mechanical splices have been studied more frequently [4], due to the growing worldwide demand for this type of solution in major infrastructure projects. This section covers the definition and types of mechanical bar splices available in the market as well as advantages of using couplers.

2.1. MECHANICAL BAR SPLICERS

Mechanical splice/joint is the common terminology for the complete assembly, including the coupler or sleeve fitted to provide a splice of two reinforcement bars. They are mechanical devices that connect two or more bars end to end. Mechanical couplers are relatively, a new concept in the current Ethiopian construction industry. Reinforcing steel bars need to be spliced to provide continuity in RC members. Bars can be spliced either through overlapping two adjoining reinforcements, also known as “lap splicing or using mechanical connector known as coupler splice” [16]. Figure 2.1 below shows a sample of lap and mechanical splices.

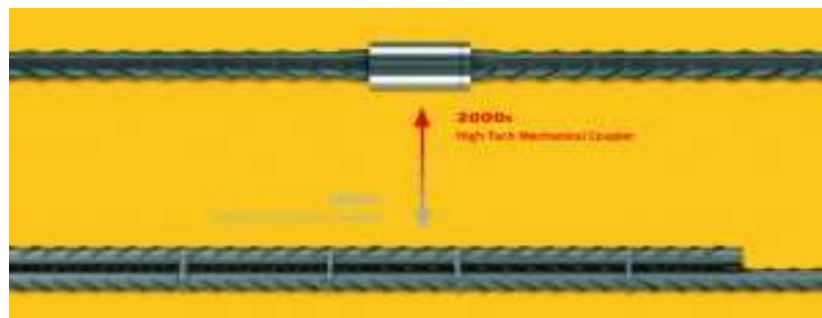


Figure 2-1: Differences between Lap & Mechanical Splicing [16]

Steel bars are generally spliced by lapping, by welding or through mechanical systems using steel couplers. “The lap splice is a solution already consolidated in the world market, being the most used system in reinforced concrete structures. However, there are situations where this type of splice cannot be used, such as in the case of execution errors in the length of pillar starters, or

waits, and especially in cases of structural restoration or reinforcement interventions, or even when it is desired to splice steel bars with diameters greater than the limit established for the lap splicing system. In such cases, designers may employ welded steel bars seams or steel couplers”[4]. The effectiveness of the coupler splicing system must be ensured through proper installation. The manufacturers of these splices assert that when properly installed, mechanical splices offer performance that can be better than that of lapped bars.

2.2. TYPES OF MECHANICAL BAR SPLICES

Different manufacturers produce different types of couplers. “The choice of the mechanical coupler system for the project depends on various factors. Selection of the most suited coupler system should be made after considering all advantages and limitations of the coupler system, the project parameters, and location of coupler joint and the preferences of the structural consultant for the project”[7]. Available couplers in the market are: shear screw, headed bar, grouted sleeve, threaded, and swaged as shown below (Fig. 2.2). Note that couplers with the same anchoring mechanism may be entitled differently by manufacturers.

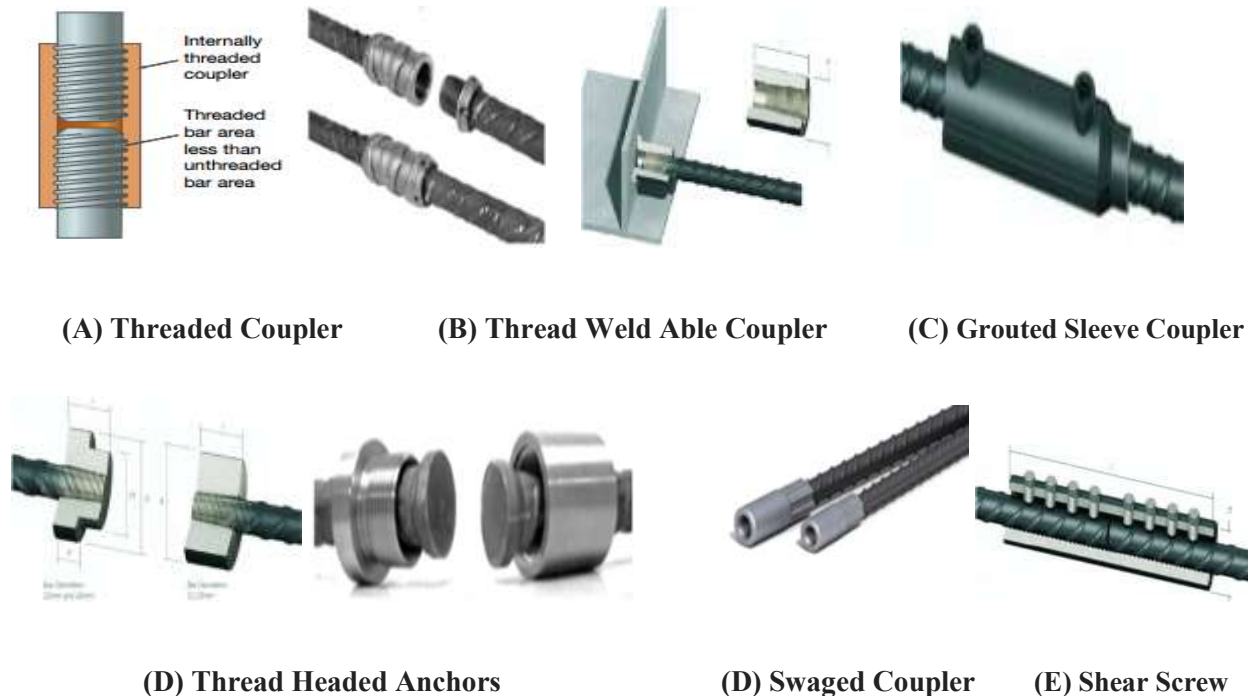


Figure 2-2: Types of Mechanical Bar Splice Couplers [14]

Mechanical couplers can be classified in the following two main categories, threaded and non-threaded couplers.

2.2.1. THREADED COUPLERS

Threaded couplers are sub-categorized into Tapered Threaded Couplers, Roll Threaded Couplers and Parallel Threaded couplers. The cross-section and length of the coupler is determined by the grade of rebar for which it is designed. The general coupler systems are described below but one should note that certain modifications are possible in each system as per manufacturer and application requirements.

A. TAPERED THREADED COUPLERS

A mechanical splicing system with tapered threaded couplers is one where the threading carried out on the rebar is at a slight incline. The slope of the threading and the coupler is kept the same to ensure engagement of all threads simultaneously in the coupler joint. Tapered couplers are generally longer in length as compared to parallel couplers and number of threads is generally mentioned as a range. There is a tolerance allowance in tapered couplers for length of threading to be carried out on rebar.

Tapered threaded couplers are the simplest type of threaded couplers where the threads are cut out on the rebar at an angle. These types of couplers are suitable for columns in general and can be used in horizontal applications but it is necessary to ensure tightening of the joint. These couplers require that the rebar be turned for tightening and hence it is difficult to use such couplers in rafts or other applications where it is not possible to turn the rebar and tighten the same.

B. ROLL THREADED COUPLERS

Roll Threaded couplers are a type of parallel threaded couplers where threads are formed by pressing the ends of the rebar using a set of rollers and are then connected by a coupler with matching parallel threads. The rebar end need to be cut perpendicular prior to threading as there is no allowance for additional threads inside the coupler in parallel threaded systems.

Roll Threaded couplers are suitable for both horizontal and vertical application. Due to parallel threads, in columns, more turns are required for tightening the coupler and hence installation

takes more time as compared to tapered system. Roll Threaded couplers provide an option of position threading – where one side of the joint can be threaded to the coupler length and the coupler can be turned to tighten the joint in location instead of rotating the rebar. However suitability of this application is to be determined based on the rebar available, as there will be a reduction in joint strength for this specific application. For vertical applications, full strength joint is achieved.

The machine for Roll Threaded splicing is more portable and such systems can be recommended for projects with coupler requirements in both raft/beams and columns. In such a system, it becomes necessary to ensure proper threading. Gauges are to be provided to ensure tolerance limits for threading on site. There can be situations where the splicing may have a play if the bars provided are undersized. In such case, it may become necessary to consider alternatives.

C. PARALLEL THREAD STANDARD BAR COUPLERS

Parallel Thread Standard Bar couplers offer a full strength connection together with enhanced fatigue resistance. They are suitable for projects of any size, including those requiring a high volume of couplers, such as road and rail bridges. Each end of the rebar to be joined is cut square and enlarged using a cold forge process. A thread is then formed on the enlarged bar end using a thread rolling machine. The thread is such that the cross sectional area of the bar ends are not reduced ensuring the strength of the connection matches or exceeds that of the parent bars. It is the application of the rolled thread that differentiates Parallel Thread Plus from other threaded rebar systems.

Each thread-rolled bar end is proof-loaded to a force equal to the characteristic yield strength of the rebar. It is the combination of these processes that provide the connection with enhanced fatigue resistance. The Type 1 connection utilizes an internally threaded coupler to join two cold-forged and thread-rolled bar ends together. Each bar end is threaded to half the length of the coupler. Type 1 connections are used where the continuation bar can be rotated. Type 1 Bar Coupler is shown in Figure 2.3 below.

2.2.2. NON-THREADED COUPLERS

There are various sub-types of non-threaded couplers which are used mainly for installation of couplers in locations where it is not possible to use threaded couplers or for specialized applications. This category includes but is not limited to crimple/swaged couplers, welded

couplers, bolted couplers, couplers with injected sleeves, friction-weld couplers etc. They are also required to conform to the same standards as the threaded couplers unless specifically approved by the consultant. The Non-Threaded couplers are mainly used as repair aids and are not used extensively in new constructions due to their higher costs, bulkiness and slower installation processes.



Figure 2-3: Parallel Thread Standard Bar Coupler [21]

2.3. CODE RECOMMENDATIONS FOR COUPLER SPLICED BARS

There are a number of different codes around the world for coupler manufacturing and joint threading. Some of the most common and relevant ones are listed below with a brief summary of their requirements.

2.3.1. AMERICAN CONCRETE INSTITUTE (ACI) BUILDING CODE 318, 2008

The ACI 318 -2008 code is the most commonly referenced code for mechanical couplers. It is purely a performance based code and categorizes splicing into two groups – Type 1 and Type 2. A Type 1 joint is required to achieve $1.25F_y$ as a minimum while Type 2 is required to comply with $1.25F_y$ requirement and also meet the specified tensile strength of the rebar. The ACI code accepts steel which has a specified minimum tensile strength. For example, for Fe500, while $F_y=500$ N/mm², the ultimate stress expected from the same is at least 700 N/mm². In view of this, the code can be read as such; For Type 1, the joint is expected to have a tensile strength not less than $1.25F_y=625$ N/mm² while for Type 2, acceptance is at 700N/mm² [25].

ACI Code section 12.14.3.2 requires a full mechanical splice or a full welded splice to develop a tension or compression force, as applicable, of at least 125 percent of the specified yield strength, f_y , of the bar. Splices developing less than full tension are permitted on No.5 and smaller bars if

the splices of adjacent bars are staggered by at least 24in. In this sense, for No.5 and smaller bars, even if full tension requirement is developed, mechanical splice can't be used without staggering. And ACI Code section 12.15.6 requires that splice in tension-tie members be made with full mechanical splices or welded splices and that splices of adjacent bars be staggered by at least 30in. Consequently, this study tries to verify the performance of unstaggered full insertion (full tension developing) bar mechanical splices in different cases [25].

2.3.2. INDIAN CODE (IS) – SP: 34 (S&T) – 1987 SECTION 4

This code specifies various types of mechanical splices and provides recommendations for usage of splicing systems and strength requirements. Acceptance of the results has been left to the structural consultant. Compliance is usually verified against IS 1786 steel code. Fe500 is considered acceptable for use anywhere in the structure if it meets ultimate strength of 545 N/mm² which is much lesser than the 1.25F_y requirement of the ACI code. In India, the Type 2 condition becomes at least 625N/mm² as well. If the joint could sustain the ultimate design requirement, it could be used freely anywhere in the structure. In this case, if the steel meets 625N/mm² criteria, it is much greater than the acceptance criteria of 545 N/mm² as per IS 1786. They do meet minimum acceptance criteria as per IS 1786 but sometimes do fall short of the ACI requirement of 625N/mm² which makes the applicability of the ACI 318 code is questionable. In such instances, it is usually up to the structural consultant to decide if acceptance criteria should be as per ACI 318 or as per IS 1786. The IS 16172:2014 is a manufacturer's code for production of couplers only and is not meant for site execution. It is recommended that the coupler be designed for Fe550 grade as a standard. The performance of the mechanical splice is to be determined with reference to IS 1786:2008 and IS 456:2000. The drawback of this classification is that even if the coupler is designed for a higher grade of rebar and has sufficient capacity, the cyclic and tensile tests will not reflect the site conditions.

The only tests recommended for acceptance on projects as per this code are static tensile tests which are considered as per IS 1786:2008 code. So for example, while a Fe500 rebar breaking at a stress of 580N/mm² would be rejected as per the ACI code criteria of 625N/mm², the same sample would be considered acceptable as per the IS 1786 requirement of 545N/mm². This will encourage the use of relatively sub-standard rebar for preparation of joints effectively reducing the factor of safety for the joint. It is up to contractors and consultants to ensure that such a practice is not promoted [13].

2.3.3. BRITISH STANDARD (BS) 8110 - PART 1 SECTION 3:1989

While the British code has lower acceptance criteria as compared to the ACI code and is almost similar to IS standard, it specifies tensile testing and Slip Test for the joint for acceptance of the sample. At the beginning of the project, the product certification taken from an authorized body of certification by a company holding a valid third party technical approval (e.g. CARES UK, BBA etc.) and should be processed by fabricators being a member of the CARES UK third party certification scheme or equivalent is presented to the employer. The certification institution controls the company's management system and coupler product regularly. According to the relevant standards of BS8110, for the couplings of Fe460B quality of steel, 497MPa and couplings of Fe500B quality of steel, 540MPa tensile strength is searched [13].

2.4. ADVANTAGES OF USING COUPLERS

The main advantages of using couplers over conventional bar splicing are broadly discussed as follows.

I. STRUCTURAL INTEGRITY

Mechanical connections offer greater structural integrity, strength and toughness during man-made, seismic or other natural events. Couplers enable the reinforcement to behave as a continuous bar in the connecting region. Loading path and structural integrity are improved as a result. In the case of overlap, the load transfer mechanism takes advantage of the bond between the steel and the concrete to transfer the load here. Mechanical splices or couplers transfer load directly from one bar to other without indirect interaction of concrete which enhances the structural integrity and thus overall reliability of the reinforcement system. Spliced bars behave as continuous lengths of reinforcing steel bars by providing full strength in tension and compression and stress reversal applications.

II. Easy to Implement

The threads are made in the steel end where automatic threading machines are used for precise and optimum pitched threads. Due to involvement of machinery, the workmanship is improved and time is reduced.

III. Reduced Congestion

Although the diameter of couplers are a little larger than that of the reinforcement but not as large as when rebars are overlapped or contact (weld) spliced. Therefore, congestion is reduced to almost 50% due to use of couplers; hence less congestion of reinforcement at joints. The amount of reinforcement used in a particular structure is lower when couplers are incorporated. Moreover, bar couplers may be used in precast member connections to accelerate construction. Building codes stipulate a steel ratio for columns of less than 8% and this makes it nearly impossible to achieve a balanced design with lap splicing. Mechanical splices allow the designer to achieve an ideal balance of steel and concrete by eliminating the additional rebar in the lap zone.

IV. Cost Efficient Solution

As per the personal experience and observation of the author of this research on CBE New Head Quarter Building Project, tons of reinforcement is saved because of re-bar couplers; wastage and overlap are entirely avoided and in comparison to that, coupler splicers are much cheaper and economical. For bars larger than 16mm diameter, it costs approximately ¥3 per piece in China. In addition to the aforementioned advantages, it provides the possibility of joining bars of any diameter together.

2.5. RESEARCHES ON MECHANICALLY SPliced BAR

Different codes and standard stipulate that mechanical splices should be staggered in the longitudinal direction. It means that staggering must be ensured in all cases. However, it is demonstrated that no or some splice staggering may be the best solution for the actual site condition (constructability) [26]. Those researchers say that “staggering requirement was first established in JSCE Standard 1982 and has not been modified until now. It maintains an old practice and must be improved for the new condition. The strength of the beams using mechanical splices that could develop over 120% yield strength of rebar (03 thread pitches and 06 thread pitches) does not depend on the staggering length. The staggering length has an effect on improving cracking behavior and ductility of the beams. The full insertion mechanical splices (06 thread pitches insertion) that could develop over 140% yield strength of rebar can be used at the same cross section with almost no difference with the beam without mechanical splice”[26].

The research carried out an experiment on deformed steel bars with nominal strength 345 N/mm² test. The bars had rolled-on deformations with the profile similar to the shape of a threaded coupler so that the bars can be mechanically spliced using the couplers.

To produce some qualities of mechanical splices, “insertion length of the bar into the coupler was controlled”. There were four types of insertion lengths studied (Figure 2.4), including: 2 thread pitches (MS-16), 3 thread pitches (MS-24) and 6 thread pitches (MS-48). The strains were measured over 180 mm length (included the mechanical splice) of the specimens. Observably, “specimen MS-48e has the same behavior with the MS -48 specimens, with the same yield strength, yield strain, ultimate strength and failure mode and has almost the same as the specimen MS-48, the failure mode is also bar-break type but with a lower stiffness. Specimen MS-24 could obtain the yield strength of the bar but with had a lower ultimate strength and stiffness and failed due to slipping out of the bar from the coupler. Specimen MS-16 could not obtain the yield strength of the bar (about 80%), had very low stiffness and also failed due to slipping out of the bar from the coupler” [20].

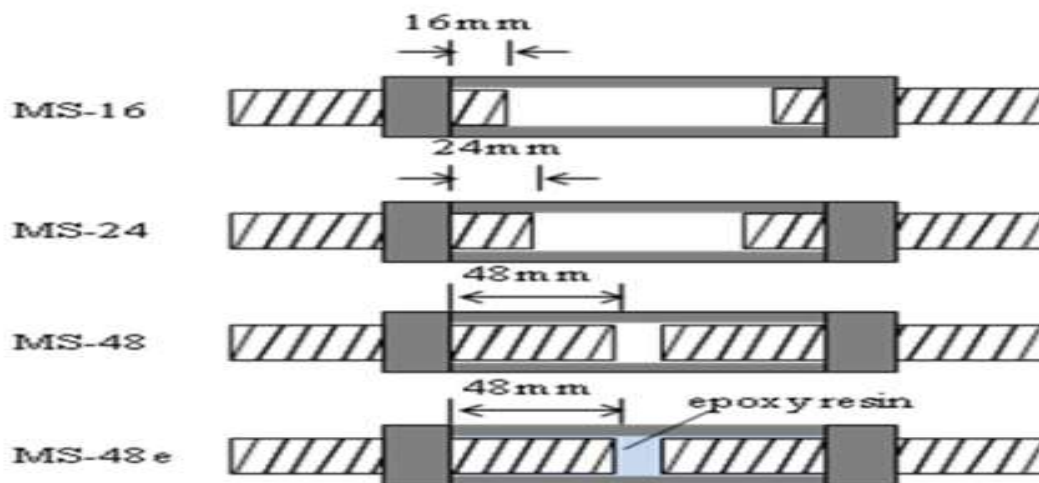


Figure 2-4: Schematic Diagram of Quality Test Specimens by [26]

Table 2.1: Tensile Test Result Spliced Bars by mutsuyoshi

Type	P_y (kN)	P_u (kN)	P_u/p_{y1} %	Failure mode
D19	112	159	142	BB
MS-16	-	90	80.04	SO
MS-24	112	138	123.14	SO
MS-48	112	160	142.86	BB
MS-48e	112	159	142	BB

Where: BB = Bar Break; SO = Slipping Out

All the splices were intentionally assembled improperly except one splice sample, only one of the splices (MS-48) was assembled as required by the manufacturer. The tensile test showed that only the correctly assembled splice exhibited almost same initial stiffness and strength as the reference non-spliced reinforcing bar. Other specimens failed by bar pullout before reaching the ultimate strength [26].

2.6. RC MEMBERS INCORPORATING MECHANICAL BAR SPLICES

The performance of RC beams and columns incorporating mechanical bar splices was investigated in several studies. However, no data was found regarding the flexural performance of RC frames with couplers. According to the researcher [26], the beams using MS-16 mechanical splices (B2, B3, B4) show the lower strength than the beam without mechanical splice (about 65 ~ 75%). The beams using MS-24, MS-48 and MS-48e mechanical splices could achieve peak loads slightly larger than the beam without mechanical splice. This can be explained by the strength of the mechanical splices used in the RC beams. Even the mechanical coupler splices are staggered they were located in the constant moment area, as shown below figure 2.6, the applied loads on the continuous steel bar and mechanical splices in this area were the same. Consequently, the beams strength corresponded to the strength of mechanical splices used. Finally they conclude that staggering length of the mechanical splices bar does not affect the strength of the beam but it will have a better cracking behavior and ductility if the staggering

length is increased, Full strength mechanical splices (48mm of insertion length and injected epoxy) that could develop 140% yield strength of rebar can be arranged at the same cross section without significant changes from the beam without mechanical splices [26].

3. EXPERIMENTAL PROGRAM

3.1. INTRODUCTION

In this study, a C-30 concrete was selected for the specimens. The mixture included cement, coarse aggregate, sand and tap water. The coarse aggregate was well-graded gravel and the water was clean tap municipal water. The mix design was prepared referring to the specification for mix proportion design of ordinary concrete for target strength of 30MPa strength. Experiments were conducted at Addis Ababa institute of technology (AAiT) materials laboratory. Cubes (150 mm) were cast at the same time with the beams, and all were cured outdoors together. The compressive strength was tested with beam loading tests, which were 14 days after casting. The average 14 day strength was 30.5MPa.

3.2. MATERIALS AND SPECIMEN FABRICATION

3.2.1. CONCRETE MATERIALS

Water: Water from municipality was used to prepare concrete mix and cure the test specimens.

Cement: For the purposes of this study, Portland pozzolana cement (PPC) from Mugger Cement Factory was used for the concrete mix.

Aggregates: Fine aggregate (sand) and coarse aggregates (gravel) were used in this test specimen preparation. The nominal maximum size of coarse aggregate was as large as possible within the limits specified but in no case greater than one-fourth of the minimum thickness of the member, provided that the concrete can be placed without difficulty so as to surround all reinforcement thoroughly and fill the corners of the form.

3.2.2. MECHANICAL COUPLERS

Mechanical splice/joint is the common terminology for the complete assembly including the coupler or sleeve fitted to provide a splice of two reinforcement bars. There are various types of mechanical splicing methods that have been developed and implemented in reinforced concrete structure projects to transfer tension forces from one bar to the other spliced bar and to ensure the continuity of the reinforcement but this study was restricted to threaded couplers (couplers with

parallel threads) where the mechanical connectors with threaded portion had a cross sectional area equal or greater than the nominal size of the bar. It is the most common type of coupler and incidentally, the type of coupler used in CBE new head quarter design and build project. It can develop yield strength of 625MPa as shown in Figure 3.1



Figure 3-1: Type-1 Parallel Threaded Mechanical Splicers (Couplers)

Table 3-1: Physical Properties of the $\phi 16$ Threaded Coupler

Bar Size (mm)	Diameter Of Coupler (mm)	Internal Diameter Of Coupler (mm)	Length Of Coupler (mm)	Internal Thread (No.)
16	24	14.2	40	14

3.2.3. REINFORCING STEEL BARS

Locally manufactured and commonly used deformed bars were used to evaluate the local rebar's performance by using parallel threaded connectors. Three types of diameters were adopted for beam specimen reinforcement, and the tensile strength of the bars were determined by testing three identical specimens from each diameter using tensile strength

testing machine. Threads were made on the reinforcement at end of the bar for coupling and properly anchored at the other end of the beam (Figure 3.2).



Figure 3-2: Reinforcing Deformed Steel Bar Specimens

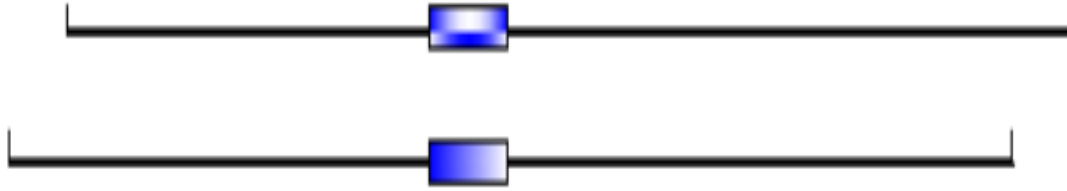
3.3. PREPARATION OF TEST SPECIMENS

In the preparation of test specimens, ISO was used as a guideline along with other considerations such as AAiT laboratory restrictions. For example, sections of the specimen were decided depending on the laboratory conditions in AAiT and manual cyclic load applying frames available.

3.3.1. MECHANICAL COUPLED (SPLICED) DEFORMED STEEL BARS

To understand the flexural effect of the member with coupled steel reinforcement, two types of coupler detailing arrangements were used. The layout of the beams included a reference beam with no splice (B1), three beams (CIB-2) with correctly installed threaded coupler (100% spliced at the constant moment region of the span) and one beam (CIB-1), 50% of the bar spliced at constant moment region of the span. Figure 3.3 shows a schematic diagram of rebar detailing for each specimen and mechanically coupled reinforcing bars. The splice specimens have identical couplers and $\varnothing 16$ identical bars. The coupling system has a threaded internal anchorage to develop the required bond strength, yield strength of the bar and to transfer of axial tension or compression from one bar to the other. The couplers used in this study were commonly available in the market with specified tensile strength of greater than 625 Mpa and 3% relative elongation. The profile made it easy to embed bars by torque and ensure the quality of coupling splice. In

fabrication of the splices, the bars were carefully inserted in to the coupler from their two ends to make the two bars line up accurately.



(a) Specimen With Bars 100% Connected By Coupler At Constant Moment Region (CIB-2)



(b) Specimen with 50% of the bars connected by coupler at constant moment zone (CIB-1)

Figure 3-3: Schematic Diagram of Mechanically Coupled Bars

Table 3-2: Details and Specification of Tested Beams

No.	Specimen	Details
1	B-1	Beam as Reference (without splice)
2	CIB-1	Specimen with bar clustered at mid span 50%
3	CIB-2	Specimen with 100% of the bars clustered at point of maximum constant moment region (mid span).

3.3.2. DETAILED RE-BAR SPECIMENS FOR BEAM

Stirrups of the reinforcing steel bar were hooked at 135 degrees at the end of the beams' diagonal bars to enhance the bond strength between the bar and concrete. The reinforcing area required for the beam was calculated from two points lading of 300KN and provided with 2ø16 bars as shown above on the figure. 2ø12 bars were used for compression-zone reinforcement to ensure the beams would not be damaged during handling. Closed loop stirrups were used in areas where shear force is pronounced i.e. areas between the support and the point of load application. A total of 12 steel stirrups were used per specimen with the arrangement as shown in the Figure 3.4 and 3.5 below. Reinforcing steel bars were hooked at 90 degrees at the end of the beams to enhance

the bond strength between the bar and concrete. Mechanical splices were located in the 500 mm uniform bending moment region which was obtained by applying two symmetric concentrated loads 280 mm away from mid span. No stirrup was used in this region in order not to disturb the crack patterns.



Figure 3-4: Photography of Coupler Connected Reinforcement Specimen



Figure 3-5: Reinforcement Arrangement in the Specimen Beam

3.4. FLEXURAL BEAM TEST PROGRAM

To examine the flexural performance of the reinforced concrete beams, a total of five reinforced concrete beams were cast. For this test, each specimen was 280 mm deep, 200 mm wide and 1500 mm long.



(a) Fresh Concrete Pouring



(b) Compacting in the Mold



(C) RC Beams

Figure 3-6: Photograph of Flexural Beam Test Specimens

3.5. INSTRUMENTAL SETUP

Initially, the two supports were adjusted with equal distance from center of loading cell i.e. the supports were fixed at 650mm from center. Afterwards, the beam was placed on the testing machine with respect to the side in which the reinforcement laid correctly. The instrumental setup is shown in Figure 3.7.



Figure 3-7: Photograph of Instrumental Setup

3.6. FLEXURAL BEAM TEST SETUP

With two lower pin supports positioned on the lower part of the middle frame cross beam, the upper roller steel rectangular solid section was made to transfer the load from spreader beam to RC beam. After that, upper roller was positioned 433mm apart by placing them in equal distance from the center. The load was applied acting through a spreader beam; the spreader beam rested on steel rollers and the steel rollers were placed on the RC beam. The setup is shown in figure 3.8 below. Test procedure listed on IS was used to perform flexural test.

The beams were restrained against out-of-plane deformation and were pin supported at both the right and left sides of the beam. The beams were tested under two points bending test. Displacement transducers were used to measure the deflections of beams at mid span.

Load was applied using a 300kN hydraulic actuator, to examine the performance of mechanical splices in the RC beams; the beams were investigated cyclically by three stages, 10 cycles were applied for each of the three load amplitudes: 100, 150 and 200kN respectively and finally to P_{max} to get the failure load where the design load was 200kN.



(a) Photograph of Flexural Beam Setup



(b) Displacement Measuring Transducer

Figure 3-8: Photograph of Flexural Beam Test Setup

4. EXPERIMENTAL PROGRAM TEST RESULTS

4.1 INTRODUCTION

This chapter presents the experimental program test results of flexural beam tests. First test was conducted to get the strength of each material and then taking those properties and material strength, the flexural test program was conducted. The first experimental test was on a non-spliced beam (control unit). Remaining five beam tests were on beams with two types of splice connections Type 1 (CIB-1) and Type 2 (CIB-2). These tests were conducted to investigate behavior of coupler splice connections at constant bending moment region.

To assess the flexural performance of the mechanically spliced steel bar reinforced concrete members, the spliced bars were embedded in concrete beam, and then tested under cyclic load to point of failure. The behavior of the splice in laboratory was assessed and compared to the behavior of a straight un-spliced steel bar reinforced beams. Finally, the results are presented to demonstrate the performance of mechanically spliced steel bar reinforced concrete beams.

4.2 OBSERVATIONS OF PRIMARY RESULTS

The analysis of observations and obtained results of experimental works are main stage of a research to get essential information. Observation results including load-displacement and load-strain curves, general behavior and crack pattern, ultimate strength and failure modes are presented in this section. Load - displacement curves of the samples are depicted in Figure 4.1. The maximum capacities of the samples were 259.85, 215.59, 249.85, 243.35, and 223.26KN, for the B-1, CIB-1, CIB-2-1, CIB-2-2 and CIB-2-3 sample beams respectively. Cracking pattern of the specimens is shown in Figure 4.3. Initial cracking of the B-1, CIB-1, CIB-2-1, CIB-2-2 and CIB-2-3 samples were recorded at forces of 82.78, 75.03, 72.28, 68.03 and 62.02KN, respectively.

4.3. RESULT AND DISCUSSION OF EXPERIMENTAL PROGRAM

The specimens subjected to cyclic loading and for each loading scheme, the loading procedure and residual deformations were recorded. Using 300KN servo hydraulic actuator. As the frame can not apply load in cyclic way, the way to obtain the required results was manual way of cyclic loading. The results of the experimental investigation of each specimens are presented below.

4.3.1. LOAD-DISPLACEMENT CURVES

Load-deformation response of the specimens and typical curves are presented in the figures below. One can identify the three stages from the figures; the yielding stage, the no underside brittle failure stage and the brittle failure stages. The load–displacement curves of the specimens have been briefly discussed. With regard to the stiffness, at earlier stages (before flexural cracking), the load–displacement curves of all specimens are close to each other for the first 100KN loading cycles, regardless of the using mechanical splices. This is because the behavior of un-cracked beams is determined by the gross moment of inertia of the concrete cross section. Upon cracking, since the flexural stiffness of specimens is mainly dependent on the reinforcement, the specimens using mechanical splices experience distinct degradation of flexural stiffness whereas the stiffness of the specimen without mechanical splice reduces slightly. It can be observed from figure 4.3 that as the load increases the rate of stiffness degradation after the initial flexural crack of the specimen with mechanical splices increases and is higher than that of specimen B1. Although the mechanical splices used in each specimen had different load-strain relationship as shown in the test results, the beams using them exhibit quite the same stiffness. It means that the location of mechanical splices inside the concrete is improved due to the restraining effect of the concrete on the mechanical splices. Another reason is that the mechanical splice occupies only a very small part compared to the whole length of the beam so its effect to the beam stiffness is not significant.

1. SPECIMEN B1

Specimen B1 (Figure 4.1) had a highest recorded applied load of the beam tests at the last cyclic load. At this cycle, the beam is loaded to sustain a load greater than its design load. Additionally, this beam performed better than others – having higher yield stiffness than the other beam specimens. The peak load was near 259.85KN, although this specimen was loaded to a deflection closer to -16.51 mm.

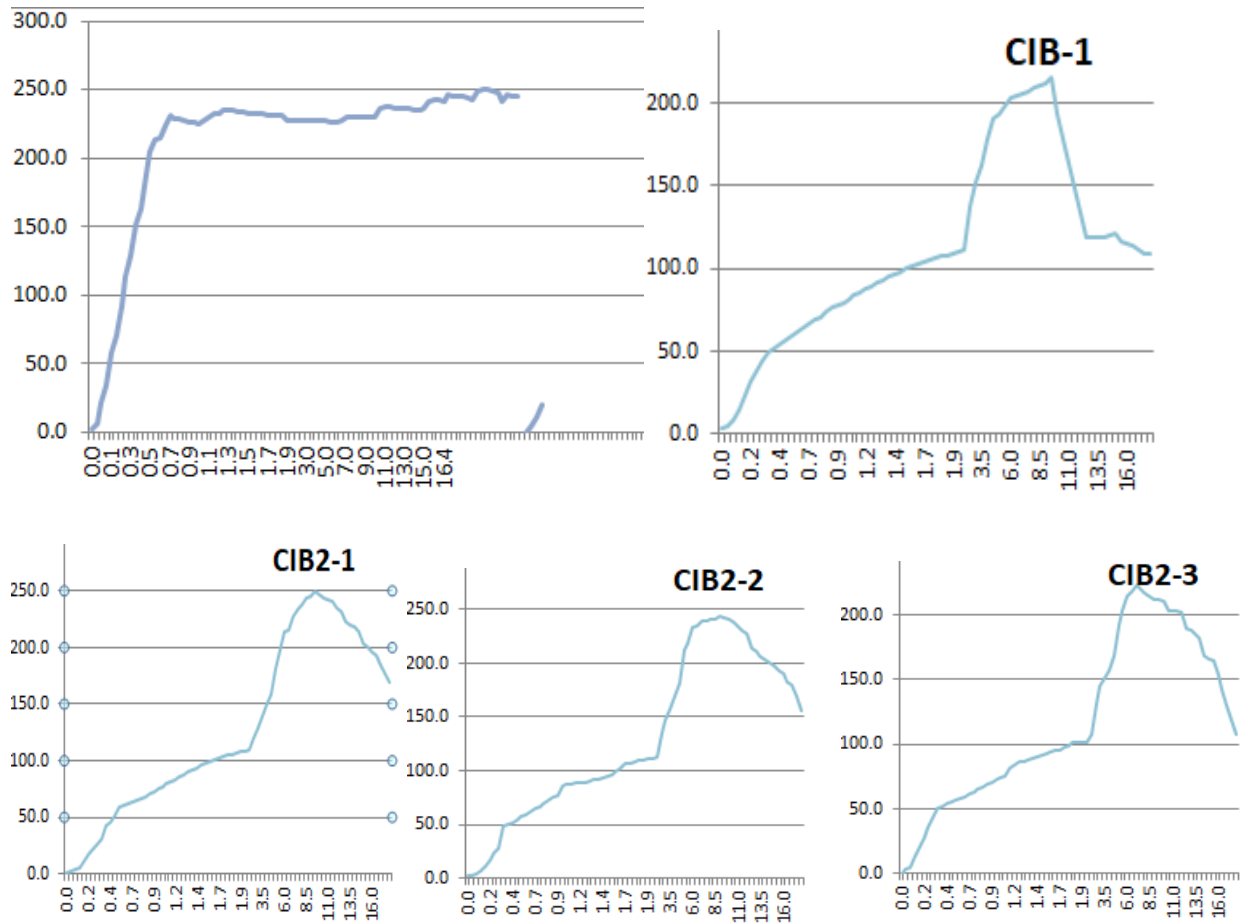


Figure 4-1: Displacement Curves at Mid-Span under Cyclic Loading

2. SPECIMENS CIB-1-1

During testing of the CIB-1 specimen, the peak load was 215.59KN with corresponding deflection of -9.65mm. After this peak load, the beam increasingly deflected without any increase in the load. Deflection was increasing even when the load intensity was decreasing at a deflection of -16.755mm. After this point, for sake of safety the load cell and hydraulic jacks were removed.

Specimen CIB-1 performed less than that of B1 and the stiffness degradation behavior of specimens is evident of this fact. At the end of the test at displacements exceeding 9.65mm, this series had a lower stiffness than the control specimen and a peak load of 215.59KN. The apparent degradation of behavior of the CIB-1 specimen as compared to the control specimen may be attributed to the expected response of the mechanical splice as compared to the continuous bar. In this mechanical splice, relative slip of bars was noticed in addition to steel bar strain, which contributed to the measured elongation across the splice.

3. Specimen CIB-2-1, CIB2-2 and CIB-2-3

Specimen CIB-2-1 and CIB-2-2 performed similarly with B1 in stiffness performing comparable to specimen B1 with increased deflection. A little degradation in the behavior of specimens is evident at the end of the test at displacements exceeding 12mm, and this series had approximately the same stiffness with that of the control specimen and a peak load of 249.85 for CIB-2-1, 243.35 for CIB-2-2 and 223.26kN for CIB-2-3 respectively. The load-displacement curves are shown in Figure 4.2.

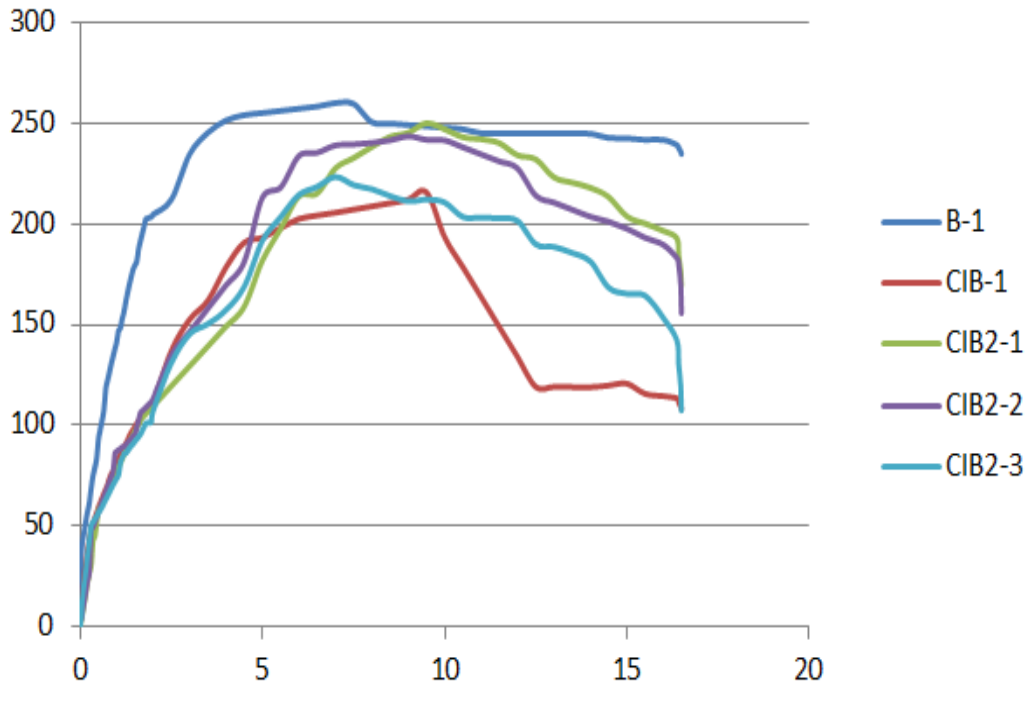


Figure 4-2: Representative Combined Load-Displacement Curves of the Specimens at Mid-Span

4.3.2. FAILURE MODE OF THE SPECIMENS

The failure modes of the beams are shown in Figure 4.3. The beam without mechanical splicers (control beam) failed after significant cyclic loading, with wide flexural crack at bottom face and the crack developed above the neutral axis with compression zone crushing between the two point loading (at constant moment zone). As it is noticed on all beams, after reaching peak load and flexural failure ends, the beams' top surface concrete failed by compression as shown on photographs below.



Figure 4-3: Mode of Failure of Control Beam Specimen

For the CIB-1 beam using 50% mechanical splices, the failure modes are a little different as the cracks suddenly widen due to slipping of the bar within the coupler. Here in this case, slipping occurred. Similarly, when the deflection is continuously increased compression failure on top surface ensued as shown on the picture.



Figure 4-4: Mode of Failure for CIB-1

FLEXURAL PERFORMANCE OF CONCRETE MEMBERS REINFORCED WITH MECHANICALLY SPLICED BARS

The beams using 50% Mechanical splices were of lesser quality and failed with a breaking sound but when the failure inside the concrete is checked by chiseling concrete around the coupler, it is noticed that the failure mode clearly shows a greater strain/slip from the mechanical splices with lesser peak load of 215.59kN. For the beams using 100% of mechanically spliced bars, the first breaking sound was heard after the peak load (249.85KN) was reached and with further loading, crushing of the beam. Brittle failure occurred with the sound of the second bar breaking nearly followed by the sound of the first bar breaking. The compression crushing of concrete occurred due to bar yielding on all beams similar to the one shown on the control specimen.



Figure 4-5: Mode of Failure for CIB-2 beams

Similarly for the CIB-2 beams with 100% mechanical splices, the failure modes were completely different from the control beam. Initially, the cracks widened bit and suddenly the bars were breaking near the couplers and within the coupler after reaching peak load of 249.85KN. Also, the crack widths observed in members with these types of mechanical splices were wider than the control beam. The cracks initiated on both sides of the couplers and had straight path between the face of the two couplers; finally, brittle failure occurred.

4.3.3. CRACK BEHAVIOUR OF THE SPECIMENS

Cracking of concrete should not adversely affect the appearance or durability of the structure; the acceptable limits of cracking would vary with the type of structure and environment, where a specific attention is required to limit the designed crack width to a particular value. The “surface width of the cracks should not, in general, exceed 0.3 mm in members where cracking is not harmful and does not have any serious adverse effects up on the preservation of reinforcing steel not up on the durability of the structures.” In this experimental investigation, there was no crack until the cracking moment was reached in the pure bending zone. These first cracks were predominantly vertical and perpendicular to the direction of the maximum stress induced by the bending moment. After the first cycle of loading, M_{cr} creating load, no more new cracks appeared and only the extension of the existing cracks was observed. The experimental crack width (w) measured at the level of longitudinal reinforcement is shown in Table 4.1. As shown in the table, the large crack widths observed were near coupler positions. The reason may be that the strains near the mechanical splices had higher values than other locations and the concrete cover at mechanical splices were smaller than the control specimen. Therefore, the important factors affecting crack width are slip out of the bar from the coupler, increasing strain in that specific location and concrete cover around the joint.



1. Cracking Behavior of B1

(B) Cracking Behavior of CIB-1

Figure 4-6: Cracking Behaviors of B-1 and CIB-1

After initial cracking, no new cracks were initiated until the third cyclic load. At this cycle, two other new cracks developed and at 150kN cyclic loading, the initial crack intensified becoming wider with new fine cracks appearing. As it is shown on the diagram, this specimen is weaker than the other two beams.

For CIB-2-1 specimen, extending and widening of the middle crack was observed after the cyclic load reached 200kN. At this point, other fine cracks were developing while the maximum measured crack was 54.6mm. The beam was still loaded cyclically in order to investigate the final failure mode, and a brittle failure of the beam occurred as a result of bar breaking.



(C) Crack behavior of CIB-2

Figure 4-7: Crack Behavior of the Specimens CIB-2

Table 4-1: Crack Width and Average Crack Spacing

Specimen	Max crack width		Max crack spacing	Average width	Average crack spacing
	At bar	At coupler			
B-1	3.4mm	-	37.8mm	2.54mm	133.45mm
CIB-1	3.5mm	6.7mm	44.6mm	2.55mm	94.1mm
CIB-2-1	3.23mm	8.6mm	59.3mm	2.957mm	125.6mm
CIB-2-2	3.12mm	7.32mm	56.7mm	3.45mm	108.34mm
CIB-2-3	4.4mm	8.2mm	46.7mm	3.15mm	110.2mm

Additionally, there is a general consensus [11]“for cyclic loading conditions, the effects of confinement reinforcement are insignificant.” and for the tests conducted in this study, no transverse confinement was provided at constant moment span in order to not disturb the flexural cracks. And shear critical zones were confined with $\varnothing 10$ stirrups and no diagonal cracks developed. The confined sections of the beams have a better crack resisting performance as observed on the experimental investigation.

4.3.4. ULTIMATE LOAD OF THE SPECIMENS

Failure modes have revealed that the ultimate load of a specimen depends on two factors; ultimate tensile strength of the beam reinforcement and the resistance force of the coupler. Accordingly, the ultimate load for B-1, CIB-1, CIB-2-1, CIB-2-2 and CIB-2-3 was 259.85, 215.59, 249.85, 243.35 and 223.26KN, respectively.

Table 4-2: Loading and Displacement Results of Samples

Name	P_{cr}	Δ_{cr}	P_{yr}	Δ_{yr}	P_{max}	Δ_{max}	P_u	Δ_u
B-1	82.78	0.43	234.59	3.77	259.6	9.925	220.66	5.730
CIB-1	75.03	0.85	204.83	6.788	214.84	14.51	182.614	7.470
CIB-2-1	62.02	0.61	223.84	6.815	249.85	9.765	212.372	6.052
CIB-2-2	68.03	0.73	202.88	5.335	243.84	11.775	208.964	6.390
CIB-2-3	72.28	0.81	193.33	5.545	201.75	9.705	191.888	5.243

The cracking, yielding, maximum (peak) and ultimate loads are denoted by P_{cr} , P_y , P_{max} , and P_u (kN unit) with their corresponding displacements of Δ_{cr} , Δ_y , Δ_{max} and Δ_u (mm), respectively. P_u was defined as $0.85 P_{max}$ and its related displacement is Δ_u , which was read from the curves in Figure 4.5.

4.3.5. ESTIMATION OF DUCTILITY

Ductility is defined as “the ability of a material, component, connection or structure to undergo inelastic deformations with acceptable stiffness and strength reduction. It is also the capability of a structure to undergo deformation after yielding without any significant reduction in yield strength”[10]. Figure 4.5 compares the structural response of brittle and ductile systems. A margin of safety against failure of a splice is required and it is also desirable that a degree of

ductility is available at the splice location in the members. Lack of ductility could result in little warning of possible sudden failure of the connection. Ductility is particularly important when designing couplers for use in structures subject to seismic loading, especially if their intended use is within the plastic hinge zones. Therefore, a reinforced concrete should have good ductility, good energy dissipation, low stiffness degradation and strength degradation during cyclic loading. The results show that the ductility of RC beam reduced when using mechanical splices. Factors such as strength of mechanical splices joint and staggering length of mechanical splices have effects on ductility. Table 4.3 shows the ductility ratio of each beam. The collapse of brittle systems occurs suddenly beyond the maximum resistance, denoted as P_{max} , because of lack of ductility. Curve 1 corresponds to large inelastic deformations representing specimen B1 on figure 4.2, which is typical of ductile systems. Whereas the other response curves are identical up to the maximum resistance P_{max} with a little difference, they should be treated differently under seismic loads.

Table 4-3: Ductility Estimation of Specimens

Beam Specimen	Yield		Failure		Ductility Δ_U/Δ_Y	Deflection at design load
	Load	Displacement	Load	Displacement		
B-1	234.59	3.77	262.34	16.53	4.384	-2.355mm
CIB-1-1	204.83	6.788	218.08	14.51	2.138	-7.505
CIB-2-1	223.84	5.815	252.34	10.995	1.891	-5.345
CIB-2-2	202.88	5.335	245.44	11.775	2.207	-5.795
CIB-2-3	193.33	5.545	203.45	9.765	1.761	-5.545

As it shown above, the control beam is more ductile than the remaining beams. Two of the beam with 100% mechanical splices at the same cross section and the beam with 50% at the same cross section, have almost the same ductility. But the beam with 50% mechanical splices increased in yielding and its failure point was earlier than CIB-2. Also, to increase the ductility of the structure, confinement reinforcement may be needed around the coupler to avoid wide cracks.



B-1

CIB-2

Figure 4. 1: Crack Behavior and Deflection at Design Load

Specimen CIB-1 performed well initially, reaching a peak load of 215.59KN. However, upon reloading and holding at this peak (to record cracking), the specimen never regained its previous capacity. As the load began to decrease, the deflection continued to increase indicating the failure of the specimen. The test was stopped at a deflection of 16.35 mm. Following testing, the splice was recovered and inspected. The splice exhibited clear signs of failure due to joint failure. The bar slipped approximately 6.7 mm through the splice. The slip is clearly visible in Figure 4.4.

In each of the bar splice beams the concrete was unable to properly confine the splice. There was cracking evident on the bottom face of the specimens caused by the excessive strain due to section reduction at the face of the coupler. This also occurs in part because the thickness of the concrete cover around the splice is lesser in size. This cracking demonstrates a particular problem with mechanically spliced re-bars. Such kinds of cracking of the concrete members would cause particular problems in structural elements.

The beams using mechanical splices could achieve peak loads slightly smaller than the beam without mechanical splice. This can be explained by the strength of the mechanical splices used in the RC beams. The CIB-1 mechanical splices could develop about 82.97% and CIB-2 could develop 91.91% of the performance of B1 beam specimen. The yield strength reduction of the specimens occurred due to threading, which reduces the actual diameter of the bar at face of the joint. And as it is shown on table above, the deflection at design load is almost double as compared to control beams for CIB-2 specimens.

5. CONCLUSIONS AND RECOMMENDATIONS

This chapter includes conclusion of the experimental program, recommendations and area of study for future researchers.

5.1. CONCLUSIONS

Flexural performance of reinforced concrete members reinforced with locally manufactured coupler connected steel bars (at mid span of constant moment region) was tested by two point loading and the following conclusion had developed from experimental program test result:-

- Parallel threaded coupler connected bar reinforced concrete beams had reduced stiffness in comparison to conventionally reinforced beam.
- Fully inserted bars into threaded parallel couplers (the connected steel bars) can be considered as a continuous bar, keeping in mind that degradation of deflection capacity and cracking behavior of the beams at the joint. Safety factors for deflection and cracking issues may need to be considered in the design stage of the parallel threaded coupler connected bar reinforced concrete members.
- Past researcher has shown that the stiffness of the coupler splicing region is even higher than that of a continuous reinforcement, due to the increased cross section at coupler splice section. However, in this study it was noticed that the splice joint reduced stiffness behavior due to thread lines at the face of the joint. Breaking failure of the bars was a result of section deduction due to threading of the bar.
- The connection had impact on the displacements, ductility and stiffness if it is used on flexure critical zone
- Brittle failure occurs due to bar breaking at threading lines of the bar.
- Beam specimens reinforced with mechanically connected bar had a good ultimate strength as; but its displacement and ductility were increased by 10.33% and decreased by 9.5% respectively.
- The crack widths of mechanically connected bar reinforced beams are wider than the control beam at the coupler region.

- Finally, the comparison of continuous bar and mechanically spliced bar reinforced concrete beams indicated that the beams could sustain the ultimate design load with increased deflection and crack width.

5.2. RECOMMENDATIONS

As mechanical splicers are a better way to join two separate bars with different diameters as well same diameter bars for continuity purposes, they are a viable option. Ethiopian building construction standards should include splicers as a primary alternative to other old and traditional ways of connecting reinforcement bars.

The design standards, specifications and safety factors on yielding strength of the splicing mechanical coupler depends on each country's seismic load and design guidelines. Different countries have different factor of safety and design considerations. The National design code of Ethiopia should incorporate this technology considering the national seismic load assessments in past years and local steel bar strengths.

5.3 AREA OF FURTHER STUDY

All test investigation in this study focused on $\varnothing 16$ locally local bars and only flexural behavior of the specimens under cyclic loading. Future researchers should investigate the behavior of larger bars which may help to develop a general specification of mechanically spliced reinforcing bars.

With the first cycle of loading, no visible difference was observed (Initial loading of 100KN), but with further cyclic loading, the deflection of the beams was completely different from the control beam. Therefore, future researchers may need further detail investigation on different loading and support condition.

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APPENDICES

APPENDIX A

TERMS AND DEFINITIONS

For the purposes of this document, the terms and definitions given in *ISO 16020* and the following apply. ISO and IEC maintain terminological databases for use in standardization:

Coupler:- Coupling sleeve or threaded coupler for mechanical splicing of reinforcing bars for the purpose of providing transfer of axial tension or compression from one bar to the other, where;

- *Threaded coupler* is a threaded device for joining reinforcing bars with matching threads.

Mechanical splice:- Complete assembly of a coupler, including any additional intervening material or other components providing a splice of two reinforcing bars.

Slip: - Relative displacement between the components of a mechanical splice while being loaded to a defined load level.

Coupler length: - Actual length of the coupler including all load-transferring parts, if more than one, and including lock nuts, if any.

APPENDIX B

DESIGN OF REINFORCED BEAM SECTION

The reinforced concrete beams were designed and produced using C-30 class concrete and locally available steel bars. The actual strengths of concrete and steel have been used in the analysis of the experimental result.

In order to do this experimental investigation, it is required to carry out the analysis to decide the section of the beam. Nominal moment strength, M_n , for any beam section with the corresponding moment strength equal to or exceeds M_u . The primary thing in this section analysis is to start with known moment value of M_u , and design of the beam cross section capable of to resist that moment and shear ($\phi M_n > M_u$ and $\phi V_n > V_u$) and also the section should satisfy all the requirement of national codes. For section analysis and flexural and shear design of this study, the ACI code requirement had used.

MATERIAL PROPERTIES

$f_{cu} = 30\text{Mpa}$, $f_{ck} = 24\text{Mpa}$, $f_{ctd} = 1.165\text{Mpa}$, $f_{yk} = 499.8\text{Mpa}$ for main bars and $f_{yk} = 500\text{Mpa}$ for stirrups. Due to the 300KN loading from the hydraulic jack, a design load of $M_u = 43.33 \text{ KNm}$ and $V_u = 100\text{KN}$.

DESIGNING FOR FLEXURE AND BEAM SECTION

The design problem in this section involves finding b and d/h as a preliminary estimate of the self-weight of the beam, selection of a target steel percentage, and final selection of the section dimension b and h . The weight of rectangular beam will be roughly 10 to 15% of the un-factored load, and the width of the beam in this study is kept to be 0.8 of the depth.

Assume b is approximately $0.8h$; then next step is to select the reasonable starting value for reinforcement ratio, $\rho = \frac{A_s}{bd}$ (1)

To start the design process, the target strain diagram was selected as shown on Figure 1B. As the tension reinforcement is needed to be in single layer, the strain at

the centroid of the tension reinforcement, ϵ_s , is equal to ϵ_t . To justify the use of $\phi = 0.9$, ϵ_t , must or exceed 0.005 for the final section design.

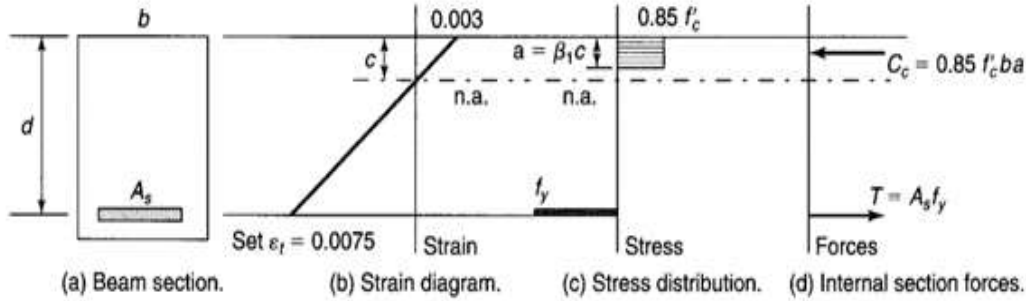


Figure 1B: Assumption for Singly Reinforced Beam Section [10]

Let's $\epsilon_t = 0.0075$,

$$C = \frac{0.003}{0.003+0.0075} d = 0.286d,$$

Using this value of c the expression for concrete compression force C_c is,

$$C_c = 0.85 f_{ck} b \beta_1 c \text{ substituting the value of } c, C_c = 0.24 \beta_1 f_{ck} b d,$$

Equating equilibrium of the forces from figure above:

$$T = C_c, A_s f_y = 0.24 \beta_1 f_{ck} b d, \text{ and steel reinforcement ratio } \rho = \frac{A_s}{b d} = \frac{0.24 \beta_1 f_{ck}}{f_y}$$

approximately $\frac{\beta_1 f_{ck}}{4 f_y}$

with an initial target reinforcement ratio that will be used for the design of singly reinforced rectangular section and an expression for balanced reinforcement ratio where:

$$\rho_b = \frac{0.85 \beta_1 f_{ck}}{f_y} \left(\frac{\epsilon_{cu}}{\epsilon_{cu} + \epsilon_y} \right) \quad (2)$$

The maximum value of $\epsilon_{cu} = 0.003$ and the steel yield strain for f_y -500 steel is 0.0021 thus the equation $\frac{\epsilon_{cu}}{\epsilon_{cu} + \epsilon_y} = 3/5$, using 50% of balanced reinforcement ratio as target value for ρ result. $\rho_{(target)} = 0.75 \rho_b = 0.255 \frac{\beta_1 f_{ck}}{f_y}$ now reinforcements.

$$\text{The reinforcement index } \mathcal{G} = \frac{\rho f_y}{f_{ck}} \quad (3)$$

The nominal flexural strength of singly reinforced rectangular section was given in:

$$M_n = A_s f_y (d - a/2) \quad (4)$$

The expression for depth of compression stress block, a was given and as modified below

$a = A_s f_y / 0.85 f_{ck} b = \omega d * \frac{1}{0.85}$ putting the expression for a , to equation (4) and simplifying the expression one can get the following expression for M_n .

$$M_n = \omega f_{ck} (1 - 0.59\omega) b d^2 \quad (5)$$

The symbol R commonly referred to as the flexural resistance factor $R = \omega f_{ck} (1 - 0.59\omega)$

$$\phi M_n = \phi R (b d^2) \quad (6)$$

$f_{cu} = 30 \text{Mpa}$, $f_{ck} = 24 \text{Mpa}$, $f_{ctd} = 1.165 \text{Mpa}$, $f_{yk} = 499.8 \text{Mpa}$, $f_{yd} = 499.8 \text{Mpa}$

$\beta_1 = 0.85$, $K_1 = 1 + 50 p_{\min} < 2$, $= 1 + 50 * 0.6 / f_{yk} = 1.07143$

$p = \frac{\beta_1 f_{ck}}{4 f_y} = \frac{0.85 * 25}{4 * 499.8} = 0.010625$, $\omega = p \frac{f_y}{f_{ck}} = 0.2125$, $R = \omega f_{ck} (1 - 0.59\omega) = 5.021$,

Because this R factor is for tension controlled beam section and the section required to failed by tension reinforcement yielding, use $\phi = 1$, and calculate for $b d^2$

$b d^2 = \frac{M_u}{\phi R} = \frac{45}{1 * 5.021}$, $b d^2 = 9.68575 * 10^6$, now set b and d are equal and $d = 217 \text{mm}$ and $h = d + 41$, hence $h = 258$ and used $h = 280 \text{mm}$, as $b = 0.8h$ it can be approximated to 200mm .

Now determine the required flexural reinforcement, $A_s = \frac{M_u}{f_y (d - \frac{a}{2})}$, $a = \omega d * \frac{1}{0.85} = 52.25$

$A_s = 592.36 \text{mm}^2$ or $2.95 \phi 16$; but to investigate the flexural behavior of concrete beam with reinforcement connected at mid span with parallel threaded coupler, use **2 ϕ 16** flexural bars.

To conform that the tension steel is yielding, lets calculate the stress block depth a , for the selected steel area a , with new area 402.12mm^2 provided.

Check for minimum moment to prevent sudden failure with little or no warning when the beam cracks in flexure. ACI code section 10.5 requires a minimum amount of flexural reinforcement equal to that shown in the equation below.

$$A_{S_{\min}} = 0.25 \frac{\sqrt{f_{ck}}}{f_y} bwd, > 1.4 \frac{bwd}{f_y} \text{ where } f_y \text{ and } f_{ck} \text{ are in MPa,}$$

From these two values, the governing value is 117.04mm² but A_s provided is already greater than this value hence proceed to next steps.

$$a = \frac{A_s f_y}{0.85 f_{ck} b} = 47.31 \text{ mm, } c = a/\beta_1 = 55.657 \text{ mm}$$

Then to calculate the steel strain at the one level of tension steel $\epsilon_s = \epsilon_t = \left(\frac{d-c}{c} * \epsilon_u\right) = 0.008265$, this value is greater than that of the yield strain of steel ($\epsilon_y = 0.002$) and strain limit of tension controlled strain (0.005), so the assumption is that the steel is yielding and valid.

Finally using equation (6) to calculate the nominal moment strength and including the strength reduction factor ϕ , [$M_n = \phi * R * (bd^2)$], **$M_n = 46.47 \text{ KNm}$** .

DESIGN FOR SHEAR LOAD

According to ACI code section 11.4.6.1, the stirrup is not required if $V_n < V_c/2$ where:

$$V_c = \lambda \frac{\sqrt{f_{ck}}}{6} bwd = 1 * \frac{\sqrt{25}}{6} * 200 * 209 = 34.83 \text{ KN, because } V_u/\phi = 133.3 \text{ KN which exceeds } 17.4 \text{ KN stirrups are required, where } \lambda = 1, \text{ and } \phi = 0.75.$$

Check if the cross section is large enough as per ACI code section 11.4.7.9 which gives that the maximum shear in the stirrup is $V_{s_{\max}} = \frac{2}{3} \sqrt{f_{ck}} bwd$, thus the maximum allowable V_u/ϕ is $\left(\frac{V_u}{\phi}\right)_{\max} = V_c + V_{s_{\max}} = 5V_c = 174.15 \text{ KN}$ because V_u/ϕ is 133.3KN which is less than 174.15KN; the section is large enough.

Check anchorage of stirrups and maximum spacing of two leg stirrups, $f_{yt} = 300 \text{ MPa}$ and $A_v = 100.53 \text{ mm}^2$, for $\phi 8$ two leg stirrups.

- (a) The anchorage of stirrups as per ACI code section 12.13.2.1 allows for smaller stirrups to be anchored by a standard 90 or 135 degree hook stirrup around the longitudinal bars.
- (b) Find the maximum stirrup spacing.

Based on the beam depth, ACI code section 11.4.5.1 requires the smaller of 0.5d or 600mm, ACI code section 11.4.5.3 requires half those spacing is V_s exceeds $1/3 * \sqrt{fck} * b_w d = 69.67\text{KN}$, $V_{s_{\max}} = \frac{2}{3} \sqrt{fck} b_w d = 139.33\text{KN}$, as $V_n (133.3) < 139.33\text{KN}$, provide a stirrup based on governing spacing of the following equations:-

$$\left\{ \begin{array}{l} 0.5d = 104.5 \\ 600 \\ S_{\max} = \frac{3A_v f_{yt}}{b} = 452.4 \\ \frac{16A_v \min F_{yt}}{b \sqrt{fck}} = 482.5 \end{array} \right\}, \text{ hence use } \phi 8 @ 100\text{mm}$$

Compute the stirrup spacing required to resist the shear force for vertical stirrups:

$$S = \frac{A_v * f_{yt} * d}{\frac{V_u}{0.9} - V_c} = \frac{8404308}{76.3} = 110.14\text{mm}, \text{ hence use } f_{yt} > 400\text{MPa } \phi 8 \text{ bar } @ 100,$$

Finally for $M_{sd} = 45.5\text{kNm}$, $V_{sd} = 101.688\text{KN}$.

Finally the length of the beam was decided to be 1500mm and width and depth were computed as shown above as 200mm and 280mm, respectively.

Hence use 2 ϕ 16 bars for diagonal reinforcements and 2 ϕ 12bars for compression reinforcement to ensure there is no damage during handling of the beam. And also use $\phi 8 @ 100$ and staggered stirrup of closed loop.

APPENDIX C

PROCEDURES IN SPECIMEN PREPAREION

1. Sampling of materials - Samples of aggregates for each batch of concrete shall be of the desired grading and shall be in an air-dried condition. The cement samples, on arrival at the laboratory, shall be thoroughly mixed dry either by hand or in a suitable mixer in such a manner as to ensure the greatest possible blending and uniformity in the material.

2. Proportioning - The proportions of the materials, including water, in concrete mixes used for determining the suitability of the materials available, shall be similar in all respects to those to be employed in the work.

3. Weighing - The quantities of cement, each size of aggregate, and water for each batch, shall be determined by weight, to an accuracy of 0.1kg of the total weight of the batch.

4. Mixing concrete - The concrete shall be mixed manually by hand,

5. Formwork - The size of the formworks used was $28 \times 20 \times 150$ cm.

6. Compacting - The test specimens were placed as soon as practicable after mixing, and in such a way as to produce full compaction of the concrete with neither segregation nor excessive laitance.

7. Curing - The test specimens were stored in a place, free from vibration with moist air of a good humidity.

APPENDIX D

MATERIAL STRENGTH OF THE TEST RESULTS

COMPRESSIVE STRENGTH OF THE CONCRETE

This section covers the determination of compressive strength of reinforced concrete specimens. Compression tests were conducted on cubic specimens; it consisted of applying a compressive axial load to molded cubes at a rate which is within a prescribed range until failure occurs. From the test, the ultimate compression resistance of the specified specimen was recorded as shown on Table 1D below.

Table 1D: 14th Day Compressive Strength Result of the Specimen's Samples

Samples	Sections (Lxwxh)	Area (mm ²)	Load (KN)	Compressive Strength (Mpa)
1	150X150X150	22500	674.23	30.8
2	150X150X150	22500	712.07	31.6
3	150X150X150	22500	670.92	29.8
		Average	685.74	30.5



Figure 1D: Cube Strength Test Sampling

The average compressive strength at 14 days was found to be **30.5Mpa**.

TENSILE STRENGTH OF THE STEEL BAR

Tensile tests were conducted on three specimens to determine the tensile strength of the specified bar diameter shows on table 2D below.

Previous studies also show that bars spliced with mechanical connectors fail by bar breakage, given that the splice samples were assembled as required by the manufacturer.

Table 2D: Tensile Strength Test Result of the Bar

Diameter of the Bar (mm)	Yield Strength (Mpa)	Ultimate Strength (Mpa)
16	420	499.8

Table 3D: Experimentally Determined Concrete and Reinforcing Steel Properties

Specimen	Compressive Strength (MPa)		Age at time of Beam Test in Days	Reinforcing Steel Strength
	14	28		
B-1(1pcs)	30.0	33.28	30	Fy = 420Mpa Fu = 499.8Mpa
CIB-1(1pcs)	31.6	35.25	31& 35	
CIB-2(3pcs)	29.8	33.18	31	

The 28th day average concrete compressive strength was **33.9Mpa** and the individual concrete compressive strengths of each of the beam specimens are tabulated above on table 3D.